A SMALL HOLE COULD BECOME REALLY BIG

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SYNOPSIS: Groundwater has been known to be potentially dangerous in underground constructions. Failures of temporary works due to ingress of water are not uncommon. At one of the deep excavations carried out in Taipei, drilling for the replacement of a malfunctioning piezometer damaged the underlying clay blanket and necessitated the flooding of the entire pit for balancing the hydraulic pressure from the groundwater. A total of 70,000 tonnes of water was recharged into the pit. The underground cavities took about 5,400 cubic meters of grout to fill. The construction was delayed by six months and the financial loss was enormous. This paper presents the sequence of events, the remedy and the rehabilitation works.

1 INTRODUCTION

For deep excavations in soft ground, groundwater has been responsible for a great majority of problems encountered. Leakage of retaining walls, for example, frequently results in excessive ground settlements outside the pits as sands are eroded and/or as clays consolidate because of reduction in porewater pressures. If it so happens that a water main runs across the depression, chances are, it will be damaged and the running water will lead to a large sinkhole.

Described herein is a case history in which the replacement of a faulty piezometer without supervision by a geotechnical engineer led to ingress of water at the bottom of a deep excavation. The flow soon became uncontrollable and the pit had to be flooded to prevent the situation from deteriorating. Because the pit is a large one and excavation was nearly completed, as much as 70,000 tonnes of water was recharged to balance the hydrostatic pressure from the groundwater. It took 6 months to mend the damaged clay blanket under the bottom of excavation before the pit was drained and the works resumed. A total of about 3,000 cubic meters of LW (Labile Wasserglas) grout (consisting of cement, sodium silicate and water) and about 2,400 cubic meters of Cement-bentonite grout (consisting of cement, bentonite and water) was injected into the ground to fill up cavities made by the seepage flow.

2 THE SITE

The site is located in central Taipei City. As can be noted from the typical soil profile given in Fig. 1, there exists a thick layer of young sediments, i.e., the so-called Sungshan Formation, from the ground surface to a depth of 48m. The Sungshan Formation comprises 6 sublayers of which Sublayers I, III and V are composed of silty sands (SM/ML) and Sublayers II, IV and VI are composed of silty clays (CL/ML). Underneath the Sungshan Formation is a gravelly layer, i.e., the so-called the Chingmei Gravels. The Chingmei Gravels was in an artesian condition decades ago and was the sole source of water supply for the City of Taipei for nearly half a century. In the mid-70's, the piezometric level in the Chingmei Gravels dropped to, as low as, RL 60m as a result of excessive pumping. It

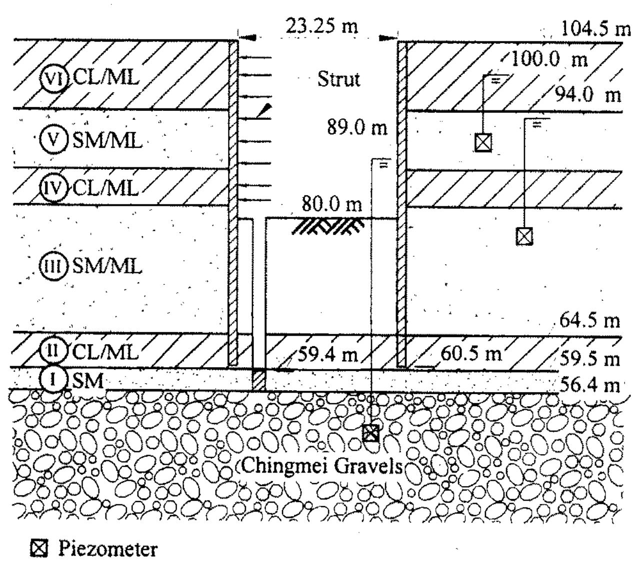

Fig. 1 Soil profile and configuration of excavation

gradually recovered as pumping was banned. The clayey sublayers in the Sungshan Formation are relatively impermeable. They separate the entire subsurface soil stratum into three aquifers with different piezometric levels, i.e., RL 100m in Sublayer V, RL 94m in Sublayer III and RL 89m in Sublayer I and the Chingmei Gravels. However, since the piezometric levels in the Sungshan Formation responded to the lowering and rising of the pressure heads in the Chingmei Gravels, there obviously were flows across neighboring aquifers.

The excavation of interest was for constructing an underground station of the Taipei Rapid Transit Systems. As shown in Fig. 1, it was 23m in width and was carried out to a depth of 24.5m by using the bottom-up method. The pit was retained by diaphragm walls, 1.2m in thickness and 44m in length, and braced by 8 levels of temporary struts. The diaphragm walls toed in Sublayer II to provide a seepage cutoff. Sublayer II, which is an impervious layer consisting mainly of silty clay, essentially served as a seal at the bottom of soil plug which was enclosed by diaphragm walls on its four sides. With a length of soil plug of 20.5m, a factor of safety of 1.3 was obtained against blow-in for a piezometric head of 29.5m at the bottom of the plug. At the time when the incident occurred, the bottom of excavation had been reached and, except at the southern end where the incident occurred, base slab had already been cast.

3 THE EVENTS

The excavation was well instrumented with settlement markers, inclinometers, load cells, etc. Because blow-in was one of the major concerns, the piezometric levels in the Sungshan Formation and the Chingmei Gravels were closely monitored. One of the piezometers became faulty and the contractor attempted to replace it by a new one. At that time, the excavation at the southern end had already been completed and the bottom of excavation was protected by a layer of plain concrete. Drilling was carried out from the bottom of excavation at the location shown in Fig. 2. As drilling reached RL 59.4m, refer to Fig. 1, water started to overflow from the borehole. The contractor promptly extended the casing to the same level as the piezometric level in the Chingmei Gravels, i.e., RL 89m. The flow was stopped and the situation appeared to be under control. However, 3 hours later, water spurted out from a sump, which was 4m in depth below the bottom of excavation, about 5m to the south of the borehole. To avoid the ground from being further disturbed, the contractor did the right thing by removing the extension of the casing and letting the water to overflow from the borehole.

Sand bags were placed on top of the borehole in an attempt to stop the water. However, they were not very effective because water was still able to flow through gaps.

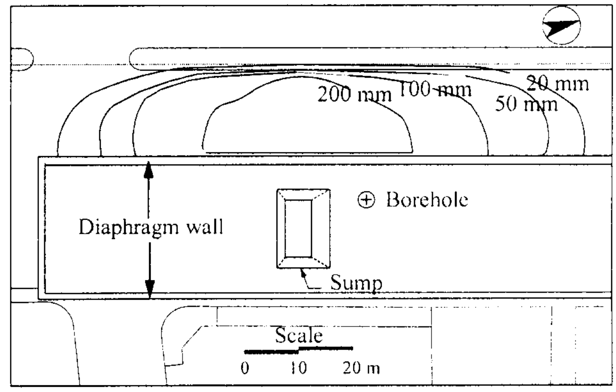

Fig.2 Site plan and settlement contour

Since the casing was 100mm in diameter and the borehole made by percussion drilling was estimated to be 150mm in diameter, much water came out from the annular space between the casing and the wall of the borehole and brought in much soil particles. As the wall of the borehole was eroded, the hole was quickly enlarged.

Three grouting machines were mobilized to inject LW grout into the ground. The grouting did stop the water for a while. However, all of a sudden, a large quantity of water rushed into the pit and brought in a large quantity of soil, estimated to be 300 cubic meters in volume. Since cracks started to develop on the street on the west of the pit and ground settlements became apparent, four of the street lanes were closed to traffic. A total of 9,000 cubic meter of sand and gravels was dumped into the pit in an attempt to stop the flow. The pile of sand and gravels reached a height of 7m. The flow rate was reduced from 5 cubic meter per minute to 2 cubic meters per minute, partly due to the capping of sand pile on top and partly due to the 2.5m increase in water head in the pit (from RL 80m to 82.5m). Since the surrounding ground continued to settle and more and more cracks were developing on the street, it was decided to flood the pit to stop the flow by balancing the differential water head between the inside and the outside of the pit. Fortunately, a water main, 600 mm in diameter, was available right at the site. An opening was made on this water main and a total of 70,000 tonnes of water was discharged into the pit in 18 hours. The water level in the pit rose to RL 89m and became stable at an elapse time of 100 hours since the leakage first started.

As shown in Fig. 2, the 20mm contour of ground settlement extended to a maximum distance of 60m away from where the borehole was. Ground settlements, as shown in Fig. 3, however, were confined by an existing underground structure, which had deep diaphragm walls as retaining structures when it was constructed, and did not go too far toward the west. The maximum ground settlement was estimated to exceed 250mm.

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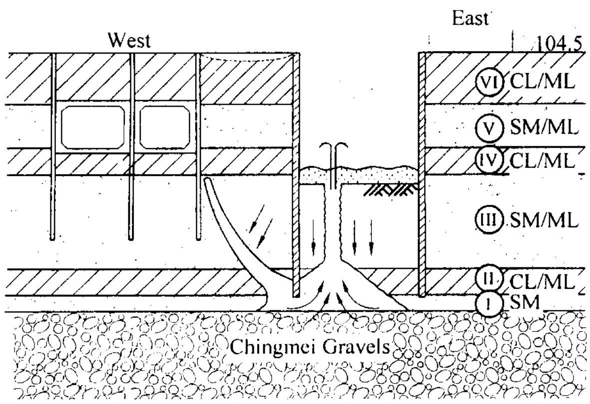

Fig. 3 Scenario of the incident

4 THE SCENARIO

The incident obviously started as the soil at the bottom of the borehole liquefied. As illustrated in Fig. 1, the tip of borehole was entering Sublayer I at the time when water started to overflow. The Chingmei Gravels is extremely permeable. The fine particles in Sublayers I and II were carried away by the fast moving water at the contact with the Chingmei Gravels. At first, water paths would propagate laterally in Sublayer I which is more susceptible to erosion than Sublayer II and the Chingmei Gravels. As Sublayer II caved in, its integrity as a clay blanket was destroyed and the water paths would propagate upward into Sublayer III which had a higher piezometric level than the Chingmei Gravels. These water paths would spread in all directions and connect each other to form large cavities. As arching effects were no longer able to hold up the ground, the roof above these cavities collapsed and ground settlements followed. The scenario shown in Fig. 3 and the extent of disturbance to the ground were confirmed by the subsequent investigations following the incident.

5 THE REMEDY

The safety of the retaining system for the excavation was certainly of primary importance. Some of the struts experienced increases in loads exceeding 20 percent of their capacity and had to be reinforced by adding new members. Struts in the affected area were braced to form a three-dimensional truss. To prevent the diaphragm walls from settling, grouting was immediately carried out along the western wall. A total of 156 cubic meter of cement milk, with a cement consumption of 96 tonnes, was injected in 12 holes at a level of RL 57m. Grouting was carried out till pressure reached a maximum of 650 kPa without specifying the quantity of grout to be taken in each hole.

The area affected was divided into Zones A, B and C as depicted in Fig. 4. Compensation grouting started in Zone C in the day following the incident in order for the street to be open to traffic as soon as possible. A total of 2,230 cubic meters of LW grout, with a consumption of 850 tonnes of cement and 223 cubic meters of sodium silicate, was injected in 68 holes. Most of the grout was taken between RL 60m and RL 84m. Grouting continued till a pressure of 300 kPa was reached without specifying the quantity of grout to be taken in each hole. Ground heaved up by, on an average, 33mm. At a few locations, heaves up to 80mm were observed. The heave, in a sense, was an indication that cavities were satisfactorily filled.

Two curtain walls were installed, one at the northern and the other at the southern end of Zone B, to confine the grout to within the disturbed zone. As shown in Fig. 5, these curtain walls extended to a level of RL 65m. A total of 800 cubic meters of LW grout, with a consumption of 200 tonnes of cement and 200 cubic meters of sodium silicate, was injected in 60 holes for forming the southern wall and in 63 holes for forming the northern wall. In each hole, the take was limited to 220 liters per meter or till a maximum pressure of 800 kPa was reached, whichever occurred first. The pin piles supporting the struts heaved up by, on an average, 10 mm or so and the sequence of grouting had to be adjusted to minimize heaves. The maximum heave of pin piles observed was 38mm.

Grouting in Zone B was carried out in 117 holes to, as shown in Fig. 5, the same depth as the curtain walls, i.e., RL 65m using the double-packer technique. Grouting continued till a take of 1,000 liters per meter or a pressure of 800 kPa was reached, whichever occurred first. A total of 1,580 cubic meters of cement-bentonite grout, with a consumption of 630 tonnes of cement and 71 tonnes of bentonite, was injected. The heaves of pin piles were in general less than 10mm with a maximum of 16mm.

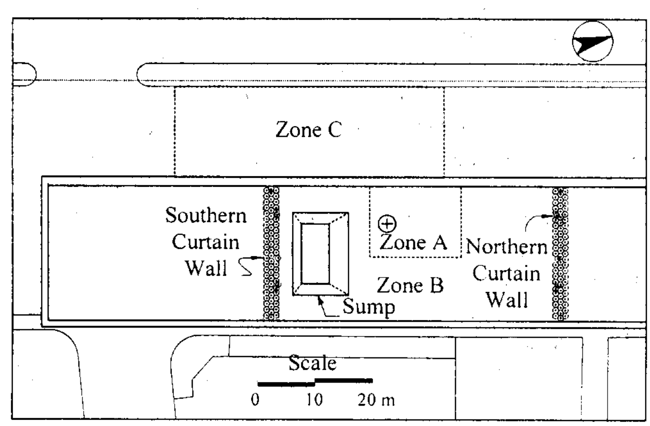

Fig. 4 Curtain walls and zones for compensation grouting

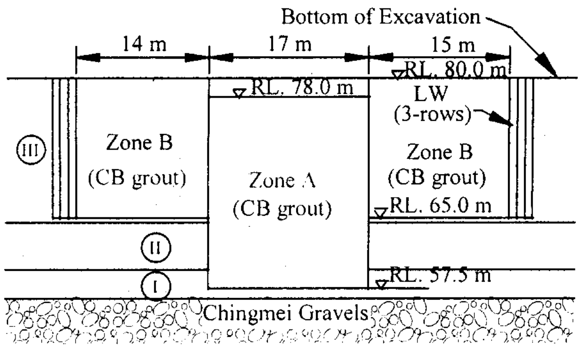

Fig. 5 Section for curtain walls and compensation grouting

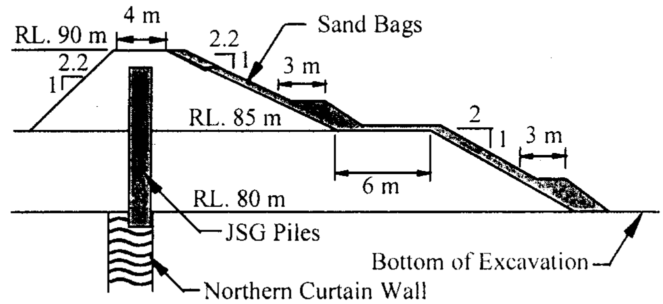

Fig. 6 Cross-section of dike

In Zone A, grouting was carried out to a maximum pressure of 300 kPa without any limit on the quantity between RL 57m and RL 65m, and to a maximum take of 700 liters per meter between RL65 and RL 78m. A total of 688 cubic meters of cement-bentonite grout, with a consumption of 275 tonnes of cement and 31 tonnes of bentonite, was injected in 33 holes. The heaves of pin piles were significant and pressure relieve holes had to be drilled to reduce heaves. Even so, a maximum heave of 52mm was observed.

The workspace was limited by many new struts and steel members bracing the struts. The water in the pond was 9m deep and there was a large pile of sand and gravels at the bottom. Furthermore, in quite a large area, reinforcement bars had already been placed for the base slab to be cast. This added to the difficulty in drilling. It thus took 6 months for the compensation grouting to be completed.

6 THE REHABITATION

As mentioned previously, at the time when the incident occurred, excavation had reached the bottom and, except at the south end, base slab had been cast. Because the remedy of the southern end would be time consuming, in order for the works in the remaining section to resume as quickly as possible, a dike was constructed to separate the pond into two. It was constructed right above the northern curtain wall using the sand and gravels previously dumped as the embankment. The embankment was trimmed to the configuration shown in Fig. 6 The curtain wall was extended by high pressure jet grouting to form an impervious core for cutting off seepage through the dike. The gaps between the grout columns and the diaphragm walls were sealed by LW grout.

The dike also served as a contingency in case the soil plug was not fully mended and a similar problem occurred for the second time. In such a case, the southern pond was able to be filled up quickly while the works to the north of the dike could still continue. Fortunately, the compensation grouting was successful and problem did not occur in the subsequent operations.

7 CONCLUSIONS

This case history illustrates how dangerous the groundwater can be in underground constructions. Any negligence could easily lead to disastrous consequences. Incidents of this nature and of this scale are not uncommon (Construction Today, 1990; Duann, et al., 1997; Tsuji, et al., 1996). To minimize such risks, participation of geotechnical engineers in underground constructions is absolutely essential.

ACKNOWLDEMENT

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