



# Evaluation of the effectiveness of an alternative to control groundwater inflow during a deep excavation into confined aquifers

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## Abstract

This paper presents a case study of groundwater control for a deep excavation into confined aquifers in Fuzhou, China. Field pumping tests were first carried out prior to the excavation to investigate the groundwater conditions at the site and as well as to evaluate the feasibility of previous designed dewatering systems (34 m diaphragm walls together with pumping wells). The results showed that under the condition of previous dewatering systems it would have been extremely difficult to safely lower the water level inside the excavation. A co-working scenario of partially penetrating curtains and horizontal waterproof curtain with jet grouting was then proposed to control groundwater inflows. Additional water-tightness assessment tests (WAT) were then performed that allowed to confirm the efficiency of the proposed method. From the feedbacks of WAT, this proposed scenario not only successfully lowered the water level inside the excavation below the excavation bottom (maximum drawdown up to 14.9 m), but also it minimized the drawdown outside the excavation (less than 0.2 m). Furthermore, an approach was proposed to approximately estimate the hydraulic conductivity of the jet-grouting at the real site from the result of WAT. The hydraulic conductivity of jet-grouted soil is three orders of magnitude lower than that of the original sediments.

**Keywords** Deep excavation · Confined aquifers · Groundwater control · Jet grouting · Water-tightness assessment test

## Introduction

In recent years, with the sustained and rapid development of the national economy in China, urban underground space development and utilization is getting more and more

attention in large and medium-sized cities. The construction of urban railway systems (metros) in urban areas has been recognized as one of the most effective way to relieve the problems of high-density development and traffic congestion. The metro stations are an important part of an urban rail transit network, but during construction they often have to encounter such complicated and difficult situations as poor ground conditions, large groundwater inflows, and densely constructed subsurface urban infrastructure. In general, groundwater resources are abundant in China's coastal regions, such as Tianjin, Shanghai, Ningbo, Fuzhou and Guangzhou. When a deep excavation is conducted in these regions with a high piezometric head in confined aquifers, water intruding may occur. Under these conditions, groundwater will endanger the safety of construction workers and could lead to engineering accidents throughout the construction of metro stations. To prevent water flowing towards the excavations during the construction of metro stations, dewatering in the confined aquifer for deep excavations must be carried out, especially for the subway stations constructed in coastal regions.

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Many studies have been conducted to investigate issues related to dewatering and excavation during deep excavations, such as pumping-induced groundwater drawdown and land subsidence (Burbey 2003; Forth 2004; Roy and Robinson 2009; Shi et al. 2008, 2017; Zhang et al. 2007; Zhou et al. 2010; Shen et al. 2013, 2015, 2017; Wang et al. 2013a, 2017, 2018a, b; Wu et al. 2015, 2017; Pujades et al. 2015, 2017; Serrano-Juan et al. 2017; Zeng et al. 2018; Cao et al. 2019; Liu et al. 2019, 2020). To relieve groundwater drawdown outside the excavation, several waterproof curtain techniques are commonly adopted during dewatering in deep aquifers, such as installing deep cement mixing columns (DCM) (Chen et al. 2013), diaphragm walls (Xu et al. 2014), jet grouting columns (Wang et al. 2013b), steel pipe sheet piles (SPSP) (Kimura et al. 2007) and steel cement–soil mixing walls (SMW) (Ding et al., 2015). The issue of the hydraulic characterization of underground enclosures during excavation dewatering has attracted the attention of many researchers when dewatering-retaining systems are employed in the deep excavation. Pujades et al. (2014) addressed the effectiveness of lengthening enclosures to avoid settlements caused by pumping during shaft excavations. Wu et al. (2016) investigated the mechanism of the blocking effect of retaining wall in the aquifer during dewatering through numerical simulation. Shen et al. (2017) proposed an empirical approach to calculate the head difference at the two sides of a waterproof curtain. However, in such cases, because of considerable difficulties in construction technology and increasing the cost of the excavation, it is unrealistic for oversized lengthening enclosures to completely cut off the groundwater and to reduce pumping effects on groundwater inflows from outside the excavation. Moreover, defects in underground enclosures at the site are frequent (Bruce et al. 1989; Croce and Modoni 2007), which were caused by several factors, such as poor construction techniques, and a poor understanding of local hydrogeological conditions. These factors could lead to the construction of underground enclosures with defects, which in turn could cause inflows of groundwater towards the excavation and unavoidably induce high drawdowns and settlements outside the enclosures. For the assessment of underground enclosures with openings, Vilarrasa et al. (2011) proposed a procedure to determine the presence and location of one opening in a circular underground enclosure. Subsequently, Pujades et al. (2012, 2016) proposed methodologies to assess linear and non-linear underground enclosures with multi-openings by analyzing the groundwater response cause by pumping inside diaphragm walls.

Certainly, some engineering cases also have found that artificial recharge is a feasible and successful method for reducing or eliminating the drawdown during pumping inside and outside an excavation (Wu et al. 2009; Zheng et al. 2018; Zhang et al. 2015). Therefore, this approach

has been recognized as an effective method for controlling land subsidence during pumping for an excavation (Zhang et al. 2017; Shi et al. 2016). Nevertheless, the rate of artificial recharge needs to be adjusted over time based on the actual change of groundwater level to maintain a balance between groundwater extraction and artificial recharge. Not only it is necessary to prevent excessive recharge water from inflowing towards the excavation, but also it avoids that the goal is not achieved due to the lack of recharge water. Therefore, the use of artificial recharge to manage the water balance during construction dewatering is technically feasible, but it is difficult to operate, especially when an excavation undertaken around dense building infrastructure. Perhaps artificial recharge is a more appropriate option for emergency assistance used in the long-term dewatering of excavations, especially when performed near particularly settlement-sensitive building structures or infrastructure. In some cases, given that neither lengthening enclosures nor artificial recharge could be carried out, other groundwater control scenarios should be developed during a deep excavation that is undertaken in deep confined or unconfined aquifers.

The present paper mainly proposes an alternative to groundwater control during a deep excavation that is undertaken in deep aquifers. Due to the local absence of an upper-confining bed in confined aquifers at the site of a metro station in Fuzhou, China, it is difficult to lower groundwater level inside the excavation and to simultaneously minimize the effect of dewatering on surrounding buildings outside the excavation. To achieve this objective, a co-working scenario of partial penetrating curtains (diaphragm walls) and horizontal waterproof curtain made of overlapping jet grouting columns, was employed for cutting off groundwater inflow during the excavation. The method successfully solved the problem of groundwater control for the excavation of this metro station and could be used elsewhere when undertaken construction in deep aquifers.

Thus, the aim of this paper is threefold: (1) to assess the impact of the horizontal waterproof curtain with jet-grouting on controlling the drawdown outside caused by pumping groundwater inside the excavation; (2) to characterize the hydraulic conductivity of the jet-grouting at the actual construction site; and (3) to address the effectiveness of horizontal waterproof curtain with jet-grouting for groundwater control when a deep excavation is undertaken in deep aquifers.

## Materials and methods

### Study area

#### Geology and hydrogeology

Fuzhou, which is the provincial capital of Fujian Province, is located in the southeastern part of China. The deep excavation was undertaken to construct a station of Metro line 2, located at Jin'an District, Fuzhou (Fig. 1). The study area is an alluvial plain on the lower reaches of the Minjiang River. According to the detailed description of several boreholes drilled at the study site, of the site is underlain by Quaternary sediments and late Yanshanian aged granites that form bedrock in the area. The typical geology profile is shown as Fig. 2. The Quaternary deposits are mainly divided into shallow Holocene sediments and deeper Pleistocene sediments at depth. The Holocene sediments consist of backfill (layer ①<sub>2</sub>), silts (layer ①<sub>1</sub>), silts with sands (layer ②<sub>4</sub>) and

fine-medium sands with silts (layer ②<sub>5</sub>). The Pleistocene sediments are composed of silty clays (layer ③<sub>1</sub>), medium-coarse sands (layer ③<sub>3</sub>), silts with sands (layer ③<sub>5</sub>) and gravels (layer ③<sub>8</sub>). The highly-weathered granites underlie the gravels, which are generally located at the depth of 50–60 m below ground surface. The geological section of the metro station is extremely asymmetrical, with great fluctuation. Table 1 presents the physico-mechanical properties of soil at the site.

As presented in hydrogeological investigation results, the study site is underlain by a multi-aquifer-aquitard system (MAAS). Groundwater beneath the site occurs both within a perched unconfined aquifers and deeper confined aquifers (I and II). The depth of the water table in the perched aquifer is 0.8–2.8 m BGS and varies with precipitation. The uppermost confined aquifer I is composed of fine-medium sands with silts (layer ②<sub>4-5</sub>). The piezometric head of confined aquifer I is 4.97 m BGS and changes with seasons. The confined aquifer II is composed of medium-coarse sands (layer ③<sub>3</sub>)

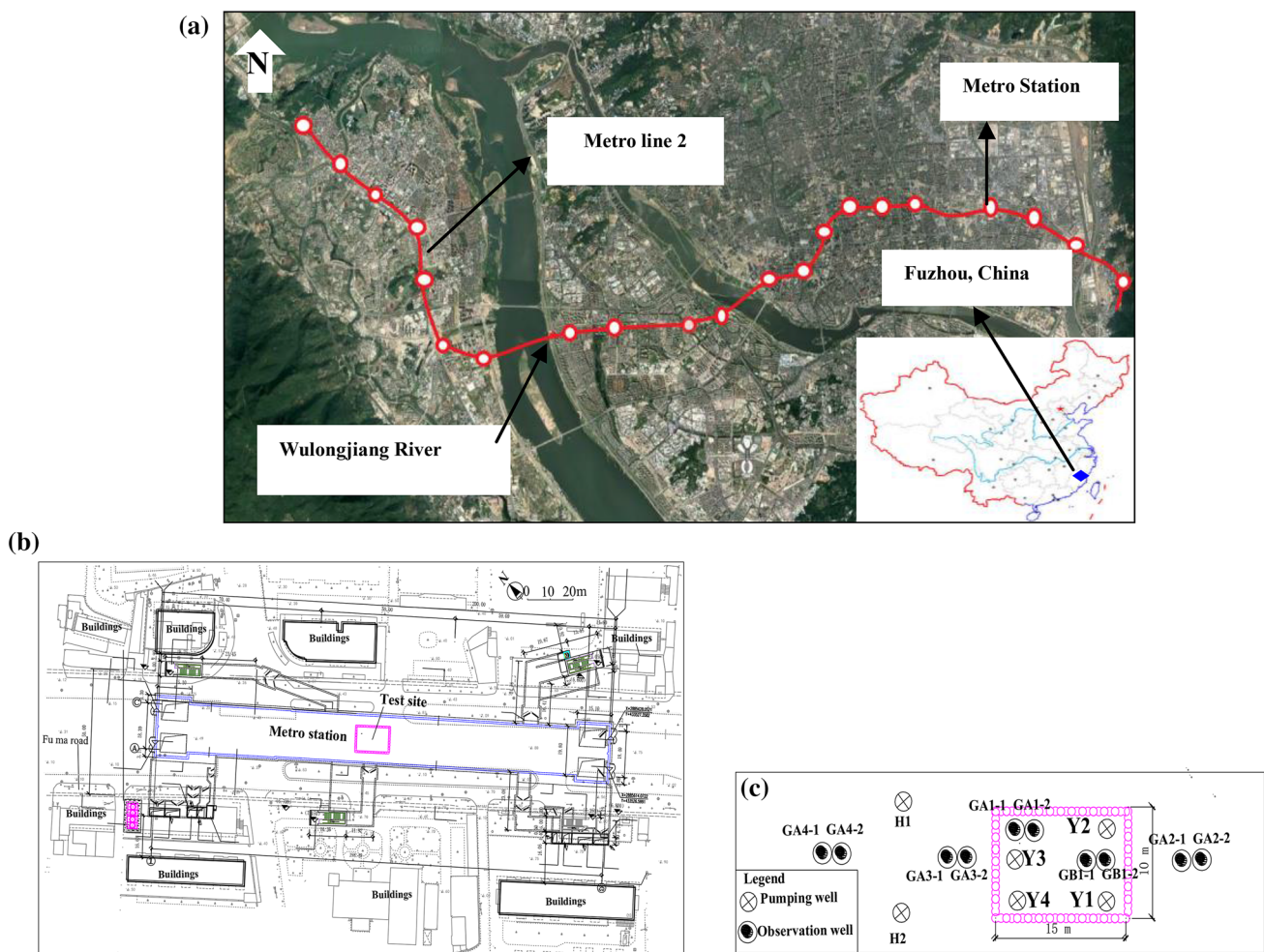


Fig. 1 a Geographical location of the study site. b Plan view of the metro station. c Layout of pumping tests



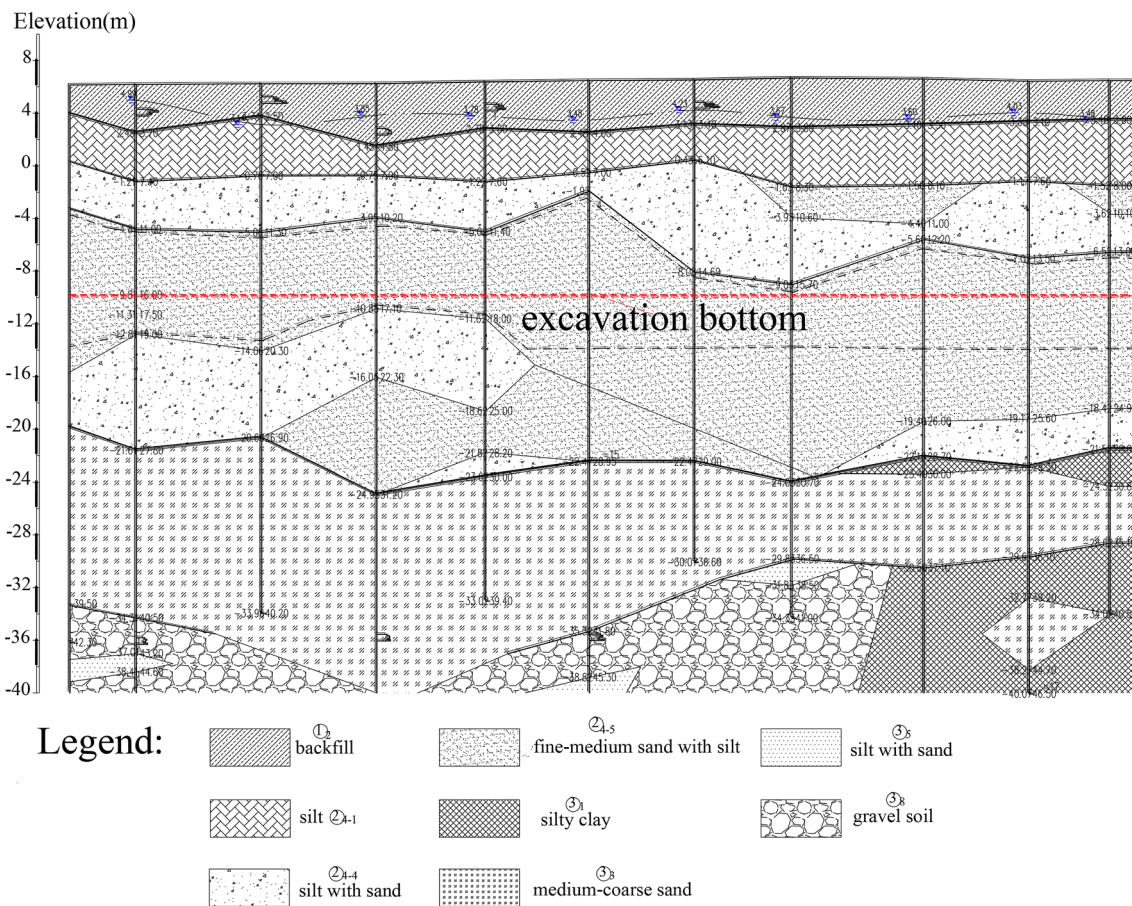


Fig. 2 Typical strata profile

Table 1 Physico-mechanical properties of soil at the site

Soil layer	Thickness (m)	Unit weight $\gamma$ (kN/m <sup>3</sup> )	Cohesion $c$ (kPa)	Friction angle $\varphi$ (°)	Hydraulic conductivity $k$ (m/day)
① <sub>2</sub> backfill	1.3–4.8	18.5	5	15	$8.6 \times 10^{-2}$
② <sub>1</sub> silt	2.2–7.2	15.6	8.6	1.7	$4.6 \times 10^{-3}$
② <sub>4</sub> silt with sand	1.2–11.2	15.9	8.5	2.6	0.13
② <sub>5</sub> fine-medium sand with silt	1.5–20.5	18.5	3	19	8
③ <sub>1</sub> silty clay	0.6–9.9	19.2	34.7	8.4	$4.0 \times 10^{-3}$
③ <sub>3</sub> medium-coarse sand	1.1–17.3	19	1	30	15
③ <sub>5</sub> silt with sand	1.2–2.8	17.4	16	5.2	0.13
③ <sub>8</sub> gravel soil	1.7–3.5	18	0	35	40

and gravels (layer ③<sub>8</sub>), and its piezometric head is 4.00 m BGS and changes with seasons.

**Construction characteristics**

The investigated project is a two-story underground island platform station, which has 5 entrances and 2 ventilation pavilions that are located in both the northern and southern

sides of the metro station. The total length of the metro station is 200 m, the width and depth in the standard segment is 19.7 m and 16.2 m, respectively. The excavation was surrounded by three roads, closely adjacent to several buildings and numerous underground service pipes. The cut and cover method was employed for the excavation of the metro station. Diaphragm walls with a thickness of 80 cm were used to form an enclosure structure for the excavation. The initial

dewatering system that was designed was composed of 34 m deep diaphragm walls and considerable pumping wells to remove groundwater from the enclosed area.

### Pumping tests

A detailed hydrogeological characterization was not performed for the excavation before the construction commenced. The construction company worried about whether the water level inside the diaphragm wall enclosure could be successfully lowered to the target level under the conditions indicated in the initial dewatering system (34 m deep diaphragm walls and many pumping wells). To fully recognize groundwater distribution and hydraulic relation among confined aquifers at the site and to demonstrate the feasibility of this dewatering scenario, field pumping tests were carried out prior to the excavation. Also, it was beneficial to quantitatively describe drawdown characteristics and the effect of excavation dewatering with cut-off walls on the surrounding environment.

The site for the pumping tests was located in the central part of the metro station. Figure 1c displays the layout of pumping tests. Two different types of pumping tests were carried out at the site, namely, single-well and multi-wells pumping tests. Table S1 presents the detailed procedure of single-well and multi-wells pumping tests. The enclosures with 34 m depth were constructed using deep cement-soil mixing piles. The length and width of the enclosure wall at the site of pumping tests were 15 m and 10 m, respectively. Single-well and multi-wells pumping tests were carried out outside and inside the enclosure, respectively.

### Description of the wells

The construction details of the pumping and observation wells are presented in Fig. S1. Two pumping wells (labelled H1 and H2) were installed to the depth of 34 m and 25 m BGS, respectively. The length of screen of pumping well H1 was 3 m in confined aquifer II, and of the length of the screened interval for pumping well H2 was 11 m in confined aquifer I. Pumping wells were partially penetrating wells with an internal and external radius of 273 mm and 600 mm. The pumping well Y1 was installed to the depth of 34 m from the surface, and the length of screens of which was 3 m in confined aquifer II. A mixed pumping well with two screens (labelled Y3) was installed to the depth of 34 m in the confined aquifers, and the length of each screen installed in confined aquifer I and confined aquifer II was 8 m and 3 m, respectively.

Five observation wells (labelled GA1-1, GA2-1, GA3-1, GA4-1 and GB1-1) were installed to the depth of 34 m in confined aquifer II, which has an internal and external radius of 108 mm and 350 mm, respectively. Four observation wells

(labelled GA1-2, GA2-2, GA3-2 and GA4-2) and one observation well (labelled GB1-2) were, respectively, installed to the depth of 23 m and 26 m in confined aquifer I. All of four observation wells (GA1-2, GA2-2, GA3-2 and GA4-2) have an internal and external radius of 108 mm and 350 mm, respectively. The observation well (labelled GB1-2) has an internal and external radius of 273 mm and 600 mm, respectively, which could also be used as a pumping well.

### Single and multi-wells pumping tests

#### (1) Test I: single-well pumping tests

The main aim of the single-well pumping tests was to identify the hydrogeological characteristics of confined aquifers. The single-well pumping test in confined aquifer I outside the enclosure was started on October 15, 2015 and lasted for 13 h. The single-well pumping test in confined aquifer II outside the enclosure was begun on October 16, 2015 and lasted for 22 h. Well H2 and well H1 were, respectively, used as pumping well, and wells GA3-1, GA3-2, GA4-1 and GA4-2 as observation wells.

#### (2) Test II: multi-wells pumping tests

The main aims of multi-well pumping tests were twofold: (1) to evaluate the effect of multi-well pumping inside the enclosure on groundwater drawdown and (2) to demonstrate the feasibility of the initially proposed dewatering system. The multi-well pumping tests started on October 25, 2015 and lasted for 6 d. Wells Y1, Y2, Y3 and GB1-2 were used as pumping wells and wells GA1-1, GA1-2, GA2-1, GA2-2, GA3-1, GA3-2 and GB1-1 as observation wells. The multi-well pumping tests included three stages: In stage I, both wells Y1 and Y3 were activated. In stage II, wells Y1, Y2 and Y3 worked. In stage III, wells Y1, Y2, Y3 and GB1-2 began working together.

### Bottom stability

Since covering depth decreases after excavation, basal heaving often takes place when dewatering is performed above a confined aquifer (Cashman and Preece 2001). In severe cases, this can cause soil failure at the bottom of the excavation, which can lead to groundwater flooding into the excavation zone. As a result, all the dewatering design scenarios need to ensure the stability of the excavation bottom. The governing equation of bottom stability is given by

$$\gamma_s h_{cr} \geq F_s \gamma_w h_w \quad (1)$$

where  $h_{cr}$  is the depth of soil from the excavation bottom to the top of the confined aquifer;  $h_w$  is piezometric head above the top of the confined aquifer;  $\gamma_s$  is the weighted average of

unit weight of soil;  $\gamma_w$  is the specific weight of groundwater;  $F_s$  is the safety factor, defined as 1.10 in this calculation. The safety factor of 1.10 is the value recommended in the Chinese design code (MCPRC 2012).

## Application and discussion

### Pumping test results

#### Single-well pumping test results

Figure 3a and b illustrate the variation of the drawdown measured in observation wells during pumping tests of well H2 and H1 outside the enclosure, respectively.

It can be seen from Fig. 3a, the groundwater drawdown of observation wells GA3-2 and GA4-2 installed in confined aquifer I increased during the first 400–500 min and gradually reached a stable level, whereas there was negligible drawdown of observation wells GA3-1 and GA4-1 that were installed in confined aquifer II. The stabilized drawdown of wells GA3-2 and GA4-2 in confined aquifer I outside the enclosure were 1.96 m and 2.82 m, respectively, whereas the stabilized drawdown of both of observation wells GA3-1 and GA4-1 installed in confined aquifer II were less than 0.15 m.

As shown in Fig. 3b, the groundwater drawdown of all of the observation wells in confined aquifer I and II increased during the first 1000 min and then gradually reached a stable level during the well H1 pumping test. The stabilized drawdown of wells GA3-2 and GA4-2 in confined aquifer I outside the enclosure were 0.82 m and 1.31 m, respectively, whereas the stabilized drawdown of both of observation wells GA3-1 and GA4-1 installed in confined aquifer II were 0.89 m and 0.87 m, respectively. These single-well pumping test results imply that there is some hydraulic connection between confined aquifer II and I. During the pumping tests,

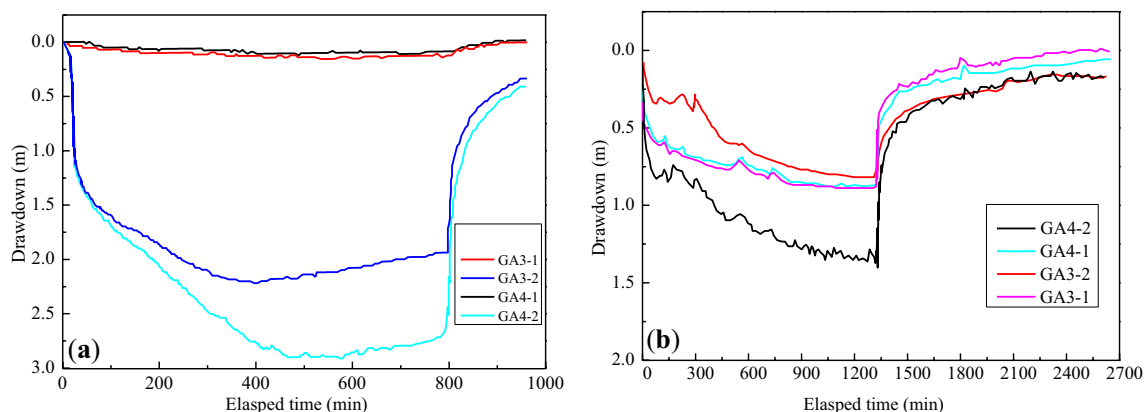
the lower aquifer can be supplied by the upper aquifer due to leakage.

### Multi-wells pumping test results

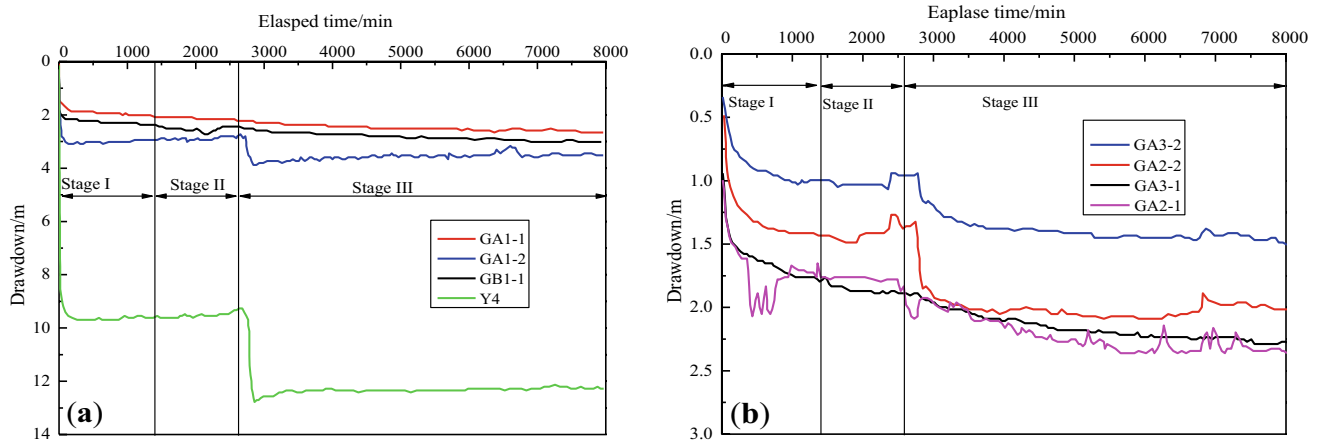
In stage I, both wells Y1 and Y3 were activated, and the pumping rate of Y1 and Y3 were 177.6 m<sup>3</sup>/day and 720 m<sup>3</sup>/day, respectively. In stage II, wells Y1, Y2 and Y3 were pumped at rates of 177.6 m<sup>3</sup>/day, 31.2 m<sup>3</sup>/day and 720 m<sup>3</sup>/day, respectively. In stage III, wells Y1, Y2, Y3 and GB1-2 began working together, and the total pumping rate of four wells was about 1180.8 m<sup>3</sup>/day. Figure 4 illustrates the variation of the drawdown at observation wells during multi-wells pumping tests inside the enclosure.

As shown in Fig. 4a and b, at the beginning of the pumping test (stage 1), both wells Y1 and Y3 were pumped. The pattern of groundwater head decreases at all observation wells (except for well GA2-1) was very similar. The groundwater head in each well decreased rapidly at first and then decreased at a slower rate until it reached a steady level. The steady drawdown in well GA1-1 and GB1-1 installed in confined aquifer II inside the enclosure were about 2.38 m and 2.04 m, respectively, and that in well GA2-1 and GB3-1 outside the enclosure were 1.83 m and 1.69 m. Then, well Y2 was activated, namely, wells Y1, Y2 and Y3 worked together (stage 2), and the drawdown inside and outside the enclosure increased at a slower rate again. As wells Y1, Y2, Y3 and GB1-2 began working together (stage 3), the drawdown inside and outside the enclosure rose further. Finally, the steady drawdown in wells GA1-1, GB1-1, GA2-1 and GA3-1 installed in confined aquifer II inside the enclosure were 2.91 m, 2.57 m, 2.31 m and 2.19 m, respectively, and that in wells GA1-2, Y4, GA2-2 and GA3-2 installed in confined aquifer I outside the enclosure were 3.79 m, 12.35 m, 2.03 m and 1.55 m, respectively.

Compared with the drawdown in wells Y4 and GA2-2 installed in confined aquifer I inside and outside the



**Fig. 3** a, b Display drawdown variation in observation wells during well H2 and H1 pumping test outside the enclosure, respectively



**Fig. 4** Variation of the drawdown in observation wells during multi-wells pumping test. **a** Inside the enclosure; **(b)** outside the enclosure

enclosure, it can be seen that there is a large difference in the drawdown due to confined aquifer I completely cut off by the enclosure. Nevertheless, the final drawdown in well GA1-2 installed in confined aquifer I inside the enclosure was only 3.79 m, which is much smaller than that in well Y4 that was installed at the same aquifer inside the enclosure. This illustrates that there may be a “skylight” where the upper confining bed was locally absent in the area where well GA1-2 is located. There is a close hydraulic relation between confined aquifer I and II in the area, which made it difficult to lower the potentiometric level of groundwater in the confined aquifers inside the excavation.

Due to the enclosure of 34 m only partially cut off confined aquifer II, the drawdown difference between inside and outside the enclosure was limited. This also illustrates that under this condition there is great difficulty in dewatering to lower the potentiometric level of confined aquifer II inside the excavation and simultaneously reducing the risks of adverse effects on adjacent buildings outside the enclosure. Thus, the dewatering system needs to be redesigned because the purpose of lowering the groundwater level inside the excavation for improving the safety of the construction cannot be achieved.

**Dewatering scenarios design**

**Proposed scenario to groundwater control**

In terms of the geological investigation at the site of the metro station, the results showed that there was a discontinuous aquitard between confined aquifer I and II below the bottom of the excavation in the middle part of the excavation area. Pumping test results also indicated that there is a close hydraulic relation between confined aquifer I and II in the middle part of the excavation area due to the localized absences in the aquitard between these two confined

aquifers. Under such conditions, it would be very difficult for pumping inside the excavation to lower the confined water head sufficiently to ensure stable and workable conditions at the base of the excavation in the area contained by the cut-off wall.

For this specific hydrogeological condition, it was proposed that a horizontal waterproof curtain constructed using high-pressure jet grouting technique would be installed at the base of the excavation. Massive constructed horizontal barriers that are installed by high-pressure jet-grouting have the potential to improve the sealing in a dewatering enclosure and may be beneficial for groundwater control during the excavation. This is particularly the case if such a horizontal barrier is coupled with conventional vertical jet-grouted barriers (or diaphragm walls).

**Design of an horizontal waterproof curtain with jet grouting**

Generally, the total vertical stress of soil can be less than the pressure of groundwater that underlies an excavation when the constructed horizontal barrier is not deep enough. At this point, pumping water is still needed to reduce the piezometric head so as to ensure the stability of the horizontal barrier. Conversely, groundwater below the horizontal barrier may not need to be pumped when the constructed horizontal barrier is deep enough. Under the condition, only the groundwater in the enclosed area would need to be removed to be below the bottom of the excavation. Thus, the position of the horizontal waterproof curtain is quite crucial for ensuring the stability of the excavation and of nearby subsurface infrastructure. In this context, Eq. (1) is able to be used for the design of the position of the constructed horizontal barriers with jet grouting.

Massive horizontal barriers are usually required to be located sufficiently deep below the base of the excavation



to prevent the need for groundwater pumping and the potential adverse structural impacts caused by excessive pumping. As the excavation depth at the construction site was 16.2 m, the minimum depth of soil from the excavation bottom to the bottom of artificially horizontal barriers was calculated to be 18.5–19.6 m when a safety factor of 1.10 was considered in Eq. (1). Nevertheless, considering a large number of uncertainties in the field conditions at the site, such as soil properties and piezometric heads, the designed depth of the artificially horizontal barrier was increased over the calculated value to provide an increased margin of safety.

The designed depths at the bottom of the horizontal waterproof curtain at the west shaft excavation and other excavation segments were 20.5 m and 19 m below the excavation bottom, respectively (shown in Fig. 5). The thicknesses of the horizontal waterproof curtain with jet grouting at the west shaft excavation and other excavation segments were 6.5 m and 5.0 m, respectively. The design requirements of jet-grouted columns are summarized as follows: (1) designed depth: from 30 to 36.5 m BGS at the west shaft excavation and from 30 to 35 m BGS at other excavation segments; (2) target diameter and center spacing of the jet

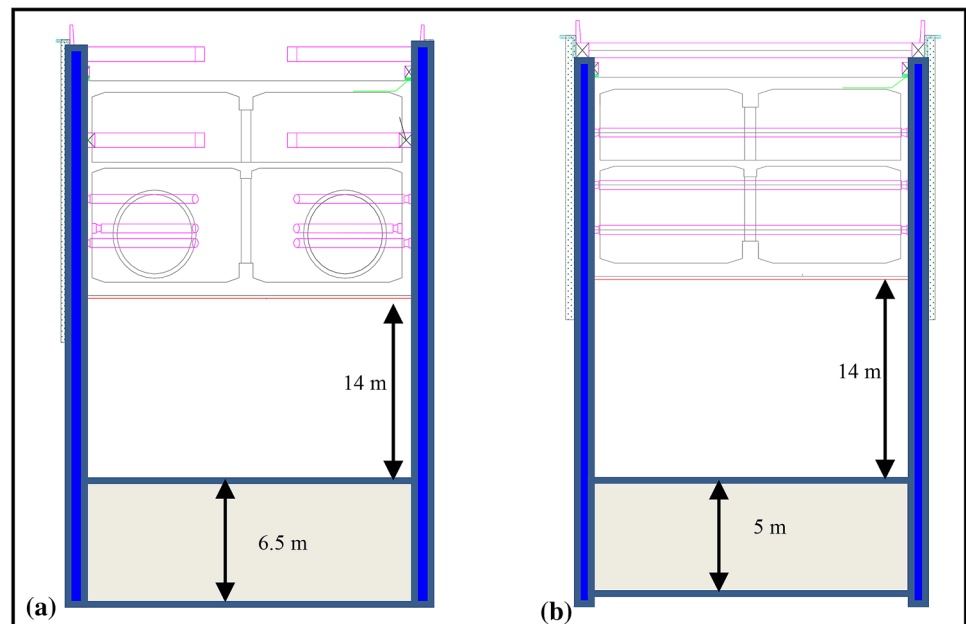
grouting columns: 1.1 m and 0.75 m, respectively; (3) average 28-day UCS: larger than 1.0 MPa.

However, due to the large depth of the jet-grouted barriers below the ground surface at the construction site under conditions where there is an elevated piezometric head, conventional high-pressure jet grouting method may not be suitable for enabling construction at the site. To solve this problem, the double high-pressure jet grouting method was used for the construction of a horizontal waterproof curtain. This technique was developed based on the traditional triple-rod method. The major difference between them is that the grout pressure is increased, almost to a level at least equal to that of the hydrostatic groundwater pressure. Table 2 lists the construction parameters used for jet grouting the columns.

### Water-tightness assessment test (WAT)

In practice, due to the inherent uncertainties of carrying out jet grouting and the heterogeneous nature of aquifer sediments, some un-grouted portions may occur in patches of coarse sand. These patches can concentrate groundwater flow and pose a serious threat to the stability of the excavation zone. Thus, additional water-tightness assessment tests

**Fig. 5** Horizontal waterproof curtain design: (a) shaft excavation; (b) standard segment



**Table 2** Construction parameters of jet grouting columns

Properties	Grout			Water		Air		Rod lifting rate (cm/min)	Rod rotation rate (rpm)
	Pressure (MPa)	Flow rate (L/min)	Water-cement ratio by weight	Pressure (MPa)	Flow rate (L/min)	Pressure (MPa)	Flow rate (m <sup>3</sup> /min)		
Range value	35	100	1:1	25–35	70	0.8	6	12	10



(WAT) should be carried out prior to the excavation to verify that the dewatering system is likely to operate in an effective manner during the actual construction. Such testing would indicate whether there is likely to be any difficulty in lowering the groundwater level inside the excavation or producing a serious risk to buildings nearby caused by pumping groundwater. If this is found to be the case, immediate remedial action should be taken.

To assess the feasibility of this scenario and to predict the influence of pumping inside the excavation on the surrounding environment in advance, additional water-tightness assessment tests (WAT) were carried out before the excavation. Figure 6 presents the layout of the pumping wells and observation wells inside and outside the excavation. A total of 23 pumping wells (labelled P-1 to P-23) were installed to the depth of 24–25 m BGS, and 6 pumping wells (labelled O-1 to O-6) were installed to the depth of 12 m BGS in three excavation zones (I, II and III). A total of 21 observation wells (labelled Q-1 to Q-14 and S-1 to S-7) were arranged outside the three subsections of the excavation. The construction details of the pumping and observation wells are presented in Fig. S2.

Table S2 illustrates the detailed procedure that was used in the water-tightness assessment tests. The multi-wells pumping test in Subsection I of the excavation was started on 8:30 am, August 27, 2016 and lasted for about 5 days. Subsequently, well P-2 was used as a pumping well and well P-3 was changed to an observation well on 8:00, September 1, 2016. The multi-wells pumping test in Subsection II of the excavation was started on 8:00 am, September 3 and completed on 20:00 pm, September 7, 2016, and, therefore, lasted for about 4.5 day. The multi-wells pumping test in Subsection III of the excavation was started on 12:00, September 9, 2016, but the pumping test was temporarily interrupted on 16:00, September 9, 2016. The pumping test was not resumed until 10:00 am, September 10, 2016, and completed on 8:00 am, September 14, 2016, lasted for about 4 day.

### Pumping rate of wells

Due to the adjustment of pumping well during multi-wells pumping test in subsection I, the results of the first 5 days of the pumping test were analyzed. In the first 5-day pumping test, wells P-1, P-3, P-4, P-5, P-6, P-7 and P-8 were used as pumping wells. Wells P-9, Q-2, Q-4, Q-6, S-2, S-4 and P-10 are installed inside and outside the subsection I of the excavation for the observation of water level variations. The results of the multi-wells pumping test in Subsection III starting from 10:00 am, September 10, 2016 were analyzed. During the multi-wells pumping tests in subsection I, II and III, the time-history curves of pumping rate of wells are shown in Fig. 7, respectively. From these results, we can see that.

The evolution of the pumping rate of wells in all the subsections of the excavation was consistent during the whole water-tightness assessment tests. At the initial stage of the pumping test, the pumping rate of each well was relatively high. With the duration of pumping, the pumping rate of wells decreased gradually. During the initial stage of the pumping test in Subsection I, the total

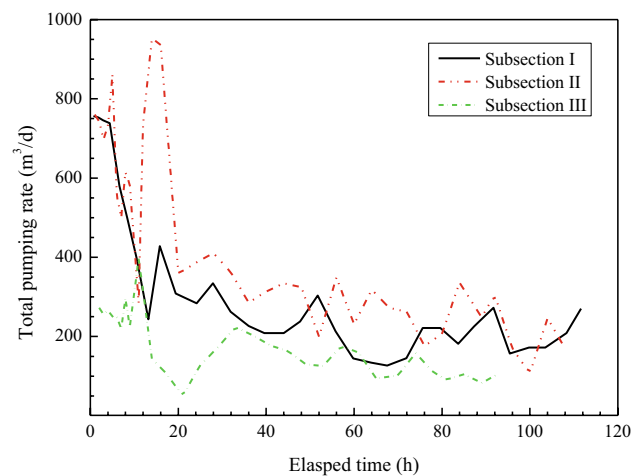


Fig. 7 Time-history curves of pumping rate of wells in different subsections

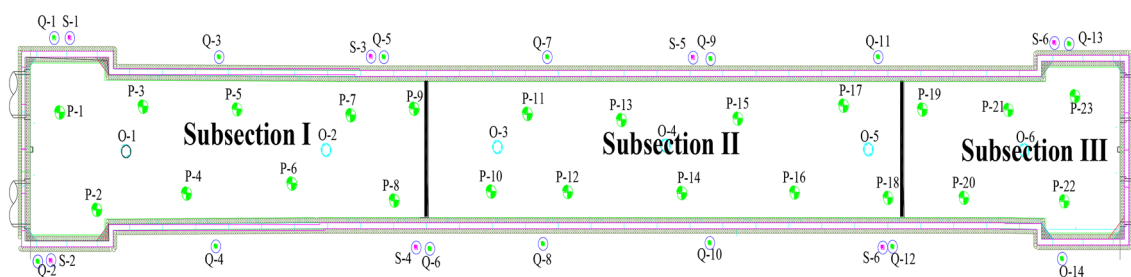


Fig. 6 The layout of the pumping and observation wells

pumping rate of all pumping wells was 756.72 m<sup>3</sup>/day. In the late period, the total pumping rate of all working wells inside was 270.24 m<sup>3</sup>/day, which was only 36 percent of that at the beginning period of the pumping test.

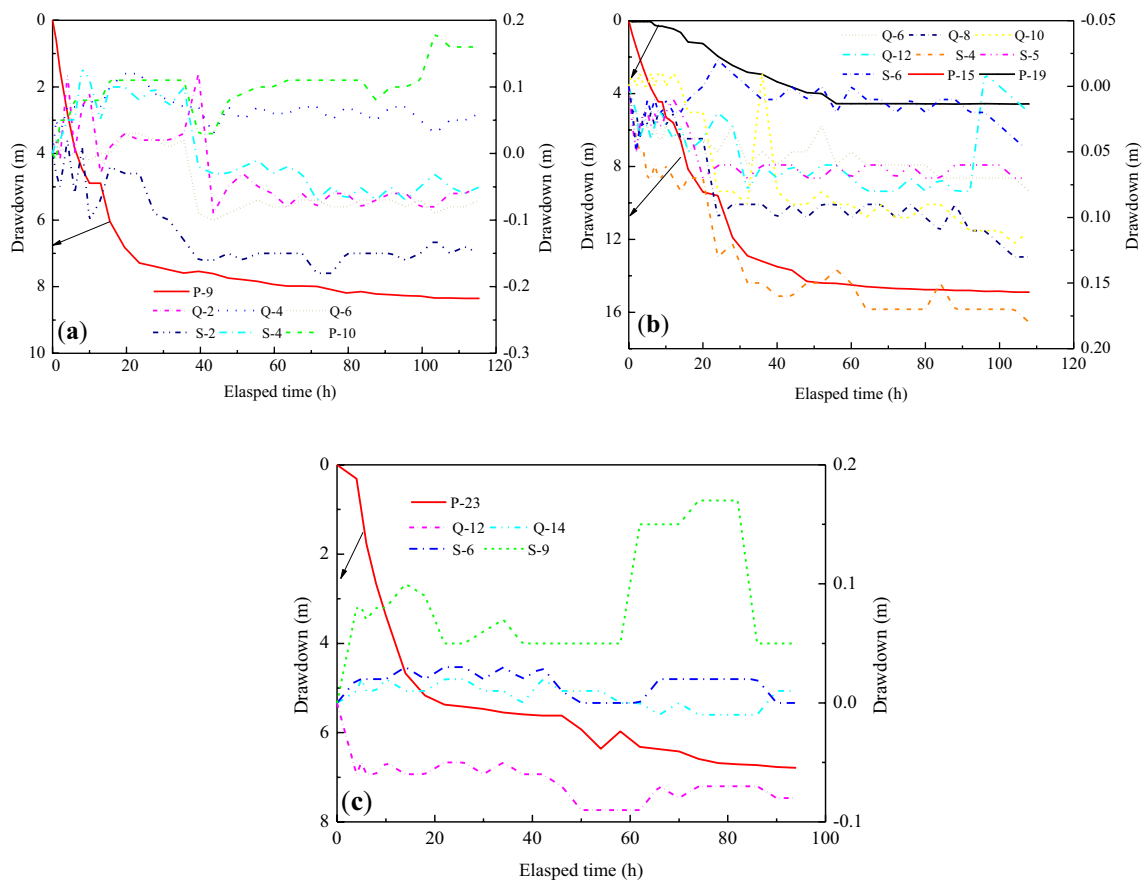
As shown in Fig. 7, during the beginning period of the pumping test in Subsection II, the total pumping rate of wells was about 764.16 m<sup>3</sup>/day. After 20 h pumping, the pumping rate of wells decreased gradually. In the late period, the total pumping rate of all operative wells was 169.92 m<sup>3</sup>/day, or a decrease of 78% of that in the initial period of the pumping test. The results of the pumping test in Subsection III were consistent with those of the other two subsections of the excavation. Water was pumped quickly from the wells at the initial stage of the pumping test (Fig. 7). As the pumping test progressed, the amount of water pumped from the wells decreased gradually. The total pumping rate of all operative wells was about 109.68 m<sup>3</sup>/day when the water level inside the Subsection III stabilized. This value was less than half of that at the initial period of the pumping test.

## Groundwater drawdown

Figure 8a–c show the variation of drawdown measured in different observation wells during the multi-well pumping test in Subsection I, II and III. Several conclusions can be drawn from this.

Firstly, as shown in Fig. 8a, the water level inside subsection I began to stabilize after 22–24 h pumping, and the stabilized drawdown of well P-9 inside the excavation was about 8.35 m. Meanwhile, the drawdown outside the excavation zone I was less than 0.16 m, which indicated that pumping inside the excavation zone I had little impact on the surrounding environment.

According to the results of groundwater drawdown measured in observation wells (shown in Fig. 8b), after 48 h pumping the water level inside the Subsection II stabilized gradually. The stabilized drawdown of well P-15 inside the excavation was about 14.9 m. From the observation of the drawdown of well P-19 inside Subsection III, it found that the water level varied significantly during the test. The stabilized drawdown of well P-19 inside Subsection III was 6.52 m, as shown in Fig. 8b. This implied that there was some hydraulic connection between Subsections II and III.



**Fig. 8** Variation of the drawdown measured in observation wells

The possible reason was that there were defects in the wall between Subsections II and III. The drawdown outside the excavation during the pumping test was steady with little fluctuation and varied from 0.02 to 0.18 m. These also showed that pumping inside the excavation zone II had little impact on the adjacent facilities outside the excavation.

As presented in Fig. 8c, after 80 h pumping test the drawdown inside the Subsection III stabilized gradually. The groundwater drawdown outside the excavation was limited and was less than 0.1 m in all of the observation wells. That implied the combined effects of the partial penetrating curtains (diaphragm walls) and horizontal waterproof curtain with jet-grouting were successful in cutting off the hydraulic connection between groundwater inside and outside of the excavation zone.

### Discussion

The horizontal waterproof curtain with jet-grouting can be employed for groundwater control when a deep excavation is undertaken in confined aquifers. In this case, it's very important to quantify the hydraulic conductivity of jet-grouting at the actual site. Although some laboratory tests suggested that jet-grouted soil had a very low hydraulic conductivity, ranging from  $8 \times 10^{-6}$  to  $8 \times 10^{-8}$  m/day (Allan and Kukacka 1995), there was a huge difference between values measured in the laboratory and the field owing to the fact that laboratory tests are usually carried out under ideal conditions that do not necessarily occur. For example, Vilarrasa et al. (2011) quantified the hydraulic conductivity of an enclosure created with jet grouting in the field, and it was concluded that the hydraulic conductivity of jet-grouted soil was typically about two orders of magnitude lower than that of the original sediments. Despite this, due to site-specific factors, such as the characteristics of sediments and their heterogeneity, the hydraulic conductivity of jet-grouting has to be calibrated for a specific site from the results of a water tightness assessment test (WAT).

When the potentiometric surface inside the excavation reaches a stable level, the total pumping rate of all operative pumping wells will be approximately equal to the

leakage rate of the installed horizontal waterproof curtain with jet grouting if it is assumed that there is no leakage through the diaphragm walls. Hence, the effective hydraulic conductivity of the jet grouted curtain can be estimated by applying Darcy's law as

$$k_{\text{eff}} = \frac{Q_p}{AJ} = \frac{Q_p b}{A(h_1 - h_s)} = \frac{Q_p b}{A\Delta h}, \tag{2}$$

where  $k_{\text{eff}}$  is the effective hydraulic conductivity of jet grouting,  $Q_p$  is the total pumping rate of all operative pumping wells,  $A$  is the area of the excavation and  $J$  is hydraulic gradient, defined as  $J = (h_1 - h_s)/b$ , where  $b$  is the thickness of the horizontal waterproof curtain with jet grouting,  $h_s$  and  $h_1$  is hydraulic head at the top and bottom of massive jet grouting, respectively, and the head difference ( $\Delta h = h_1 - h_s$ ) between at the bottom and top of massive jet grouting can be estimated from the measured groundwater level.

Table 3 illustrates the values of the hydraulic conductivity of jet-grouting estimated by applying Eq. (2). The hydraulic conductivity values of jet grouting in subsections I, II and III were calculated to be 0.139 m/day, 0.044 m/day and 0.051 m/day, respectively. It can be seen that the hydraulic conductivity of jet-grouting in subsection I is about 2–3 times that of Subsection II and III. The average hydraulic conductivity of jet grouting performed for the deep excavation is 0.0782 m/day, accounting for 0.52% of hydraulic conductivity of original sand material. Note that the hydraulic conductivity of jet grouting may be overestimated when the leakage of diaphragm walls or partition walls inside the excavation above the massive jet grouting occurs.

The reduction in permeability is significant because the hydraulic conductivity of jet-grouted soil is three orders of magnitude lower than that of the original sediments. The proposed scenario of jet-grouting has made important contributions to the reduction of pumping time and pumping rate. Thus the proposed co-working scenario of partial penetrating curtains (diaphragm walls) and horizontal waterproof curtain with jet grouting successfully achieved the goal of groundwater control during the excavation.

**Table 3** Estimation of hydraulic conductivity of jet-grouting

Subsection	$Q_p$ (m <sup>3</sup> /day)	$A$ (m <sup>2</sup> )	$h_s$ (m)	$h_1$ (m)	$k_{\text{eff}}$ (m/day)	Average $k_{\text{eff}}$ (m/day)
I	270.24	1570.6	- 13.65	- 5.65	$1.39 \times 10^{-1}$	$7.82 \times 10^{-2}$
II	169.92	1713.9	- 17.1	- 5.78	$4.4 \times 10^{-2}$	
III	109.68	852.5	- 17.0	- 4.4	$5.1 \times 10^{-2}$	

$k_{\text{eff}}$  is the hydraulic conductivity of jet grouting,  $Q_p$  is the total pumping rate of all operative pumping wells,  $A$  is the area of the excavation,  $h_s$  and  $h_1$  is average hydraulic head at the top and bottom of jet grouting, respectively

## Conclusions and lessons learned

This paper presents a case history of groundwater control for a deep excavation when undertaken in confined aquifers. Through analytical analyses and field measurements, a number of conclusions and lessons learned can be drawn in this study.

To assess the effect of previously designed dewatering systems (34 m deep enclosure and pumping wells) on groundwater control for the excavation, field pumping tests were performed under the condition of 34 m deep enclosure. Results showed that (1) the potentiometric level of groundwater was not lowered below the excavation bottom, and (2) groundwater drawdown outside the excavation was excessive. Thus, the dewatering systems had to be redesigned to ensure the safety of the excavation. An approach was then proposed using both partially penetrating curtains (diaphragm walls) and a horizontal waterproof curtain (overlapping jet-grouted piles) to control groundwater levels during the excavation process. The water-tightness assessment tests (WAT) were performed before the excavation, and based on the results of WAT, it was known that the coupled use of partial penetrating curtains (diaphragm walls) and horizontal waterproof curtain (jet-grouting) was successful for groundwater control during a deep excavation that was undertaken in the confined aquifers. The maximum drawdown inside the excavation was up to 14.9 m, whereas the drawdown outside the excavation was less than 0.2 m. Also, an approach to approximately characterizes the hydraulic conductivity of the jet grouting was proposed based on the result of WAT. The average hydraulic conductivity of jet grouting at the site was determined to be 0.0782 m/day, accounting for 0.52% of hydraulic conductivity of original sand material. The hydraulic conductivity of jet-grouted soil is three orders of magnitude lower than that of the original sediments.

Overall, an ideal procedure for groundwater control is followed during a deep excavation, including a deep hydrogeological characterization, dewatering scenario design via numerical modelling, (WAT) and monitoring of the dewatering (Pujades et al. 2014). It is worth highlighting that these ideal sequences of steps should be followed before the excavation when a deep excavation is undertaken in confined aquifers. However, a mistake was made for excavation outlined in this paper as a detailed hydrogeological characterization was not carried out before the construction commenced due to such various factors as cost reduction or meeting tight deadlines. Although the proposed co-working scenario of partial penetrating curtains (diaphragm walls) and horizontal waterproof curtain with jet grouting successfully achieved the goal of groundwater control, the cost of the excavation increased by about 3.21 million dollars.

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