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Lessons from the collapse during construction of an inlet pumping station : Geotechnical instrumentation aspects

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Lessons from the collapse during construction of an inlet pumping station : Geotechnical instrumentation aspect

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ABSTRACT: In the wastewater treatment plant, an Inlet Pumping Station (IPS) is one of the most important structures to collect and pump all wastewater to the treatment plant. The IPS presented in this paper consists of 20m deep underground chambers. A reinforced concrete diaphragm wall system 1.0m thick and 25m. deep was used in construction of the IPS. Four Inclinometer tubes were installed in the diaphragm wall to monitor the lateral wall movements during excavation work for basement construction. When excavation approached the final depth, excessive lateral wall movements were observed, but they were not taken seriously. Later, some cracks on the capping beam and noise from the bracing system were noticed. Then the IPS collapsed and was buried under sliding soil mass. At the time IPS collapsed five main bracing layers and only two of the five intermediate bracing layers had been installed. This paper emphasizes the importance of instrumentation. Additionally it presents the results of investigation into the causes of the collapse, as well as the reconstruction work.

1. INTRODUCTION

Bangkok metropolis is sectioned into four parts for the implimentation of four mega projects, or "phases" that comprise a wastewater treatment system to serve the rapidly growing industrial and domestic needs. Three of four phases are under construction. All projects are a turnkey system where the main contractors are responsible for both design and construction. The engineers will be the consultants as representatives of the owner to review the conceptual and detailed design and to supervise the construction. The aim of this paper is to highlight the importance of instrumentation in deep underground construction, as proved by the collapse of the Inlet Pumping Station (IPS) at the second phase. This paper also briefly describes its reconstruction.

2. PROJECT DESCRIPTIONS

The second phase Bangkok wastewater project presented herein was started in 1995 to serve the central area of Bangkok City. The project was planned to be completed by 1999 (ie. within 4 years). In the sewage treatment plant (STP), the IPS is one of the most important structures to collect and pump all wastewater to the sewage treatment plant. Figure 1 presents the location of IPS on the sewage treatment plant (STP) site.

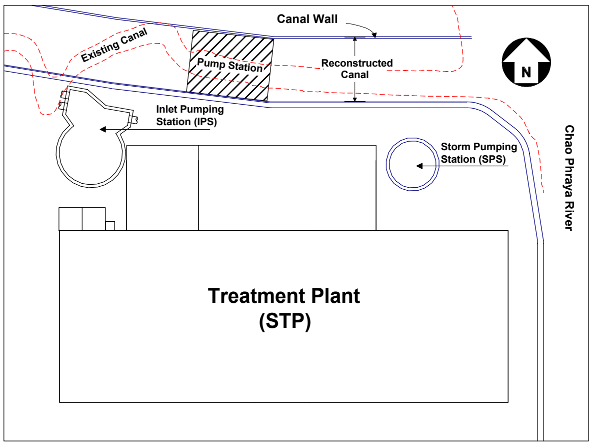

Figure 1. Layout of Inlet Pumping Station

The IPS has two distinct chambers, a pumping chamber and an inlet/bypass chamber, all built underground. The pumping chamber is a circular shaft while the inlet/bypass chamber forms a box on the north side of the pumping chamber. The pumping chamber is 20.3m in diameter and 20.2m deep. It consists of a central shaft 8.75m. in diameter and an annulus 4.875m. wide. The inlet chamber is essentially a large box receiving the inlet sewers and also housing screens for removal of debris. The inlet chamber is 11.5m x 9.0m approximately in plan. The bypass chamber is adjacent to an inlet chamber and divided by a concrete wall from base level at

19.5m below ground to roof level. The inlet/bypass chamber has three levels. The base level is at 19.50m below the ground. Levels 1 and 2 are at 10.5m and 5.5m depth respectively. The roof slab is at 1.0m depth.

The IPS was located at the northwest corner of the STP site, approximately four to ten meters from a canal. The canal is protected by concrete walls supported by battered concrete piles. Private residences consisting of wooden house were located close to the property line in the west.

Space limitation necessitated reinforced concrete diaphragm walls 1.0m thick and about 25.0m deep as permanent structures of the IPS. The diaphragm walls were also used as temporary structures during excavations up to 22.0m deep. The one-meter-thick wall was cast in panels with lengths of around 2.5 - 3.0m using the bentonite slurry trench construction method. The connection between adjoining panels was constructed by using U-shaped stop-ends with water bars.

For excavation work, five levels of temporary steel bracing were designed to support the straight wall portion and an additional five intermediate bracing layers was planned to frame the edges where the straight wall meets the circular enclosure (Fig. 2). Four inclinometer tubes were installed at critical points of the diaphragm wall to observe lateral movements during excavation for underground construction.

The bracing system was classified as temporary and thus no submission was made for review. The final temporary bracing arrangement (Fig. 3) was changed from the initial design in Figure 2. In the initial design, two lacing beams shown as a dashe line in Figure 2 were planned to strengthen the strutting system. However these were cancelled in the final temporary bracing system (Fig. 3).

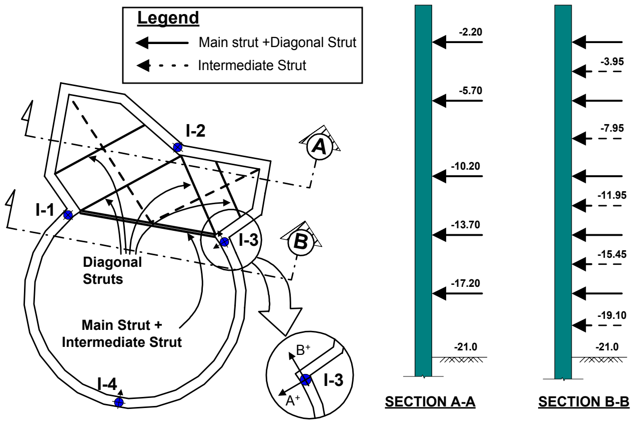

Figure 2. Temporary bracing system (Initial design)

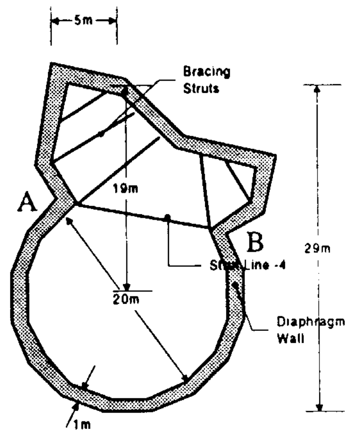

Figure 3. Final Temporary bracing system (after Kanok-Nukulchai et al., 1998).

Table 1. Summary of soil properties

Depth from	m to	Soil Description	γ_t kN/m^3	Su kN/m^2
0.0	3.0	Fill	18.0	-
3.0	16.0	Soft Clay	16.0	12-30
16.0	20.0	Stiff Silty Clay	19.0	100-150
20.0	30.0	Very Stiff Silty Clay	20.0	175-250
>30		Dense Silty Sand	20.0	-

3. SOIL CONDITIONS

At the start of the project, the site consisted of very soft clay (mud) about 2 - 3m deep. The soft clay apparently extended westward, into the residential area of the wooden houses. When work on the project commenced, mud was removed from the site up to the property boundaries. Approximately a 3.0m thick of sand backfill was placed on the site. Then the soil conditions consisted of 3.0m thick sand backfill, followed by soft to medium dark grey clay approximately 16.0m deep. Then stiff to very stiff silty clay was encountered at about 30.0m depth, overlaying the first dense silty sand layer. The sensitivity of the soft to medium clay was in the order of 2.5 to 6.0. A piezometric level was found at about 21.0m below ground level, as a result of deep well pumping. A summary of general soil properties and soil stiffness is presented in Table 1.

4. COLLAPSE OF INLET PUMPING STATION

The reinforced concrete diaphragm wall and installation of four inclinometers were completed on 15th January 1997 by the diaphragm wall sub-contractor. After completion of the diaphragm wall, the same sub-contractor pointed out to the main contractor the importance of an adequate bracing system and staged construction sequences. The excavation and strutting was carried out by another subcontractor, while the other permanent structures such as capping ring beam were constructed by the main contractor. The capping ring beam had been casted to two thirds of its height by the main contractor on 8 June 1997. Two inclinometer tubes (I-1 and I-2) were found damaged and this was reported. But the damaged tubes were left unrepaired and could not be used for monitoring.

The excavation began in mid-June, 1997. Backhoes were used for excavating inside the shaft and all materials were collected with a bucket. The bucket was hung from a service crane located behind the diaphragm wall perimeter near I-1 location (collapsed zone) on the ground. The excavation and installation of struts was carried out step by step. However, only 5 diagonal bracing layers, 5 main struts and two out of five intermediate struts were installed without instrumentation.

Lateral wall movements were recorded by I-3, located at one edge of circular enclosure and I-4, located in the intact zone (Fig.2). Wall movements in A-axis direction of the inclinometer access tube I-3 were indicative of the wall in cantilever/unsupported condition while those in B-axis direction were indicative of the wall being supported (Fig. 4a).

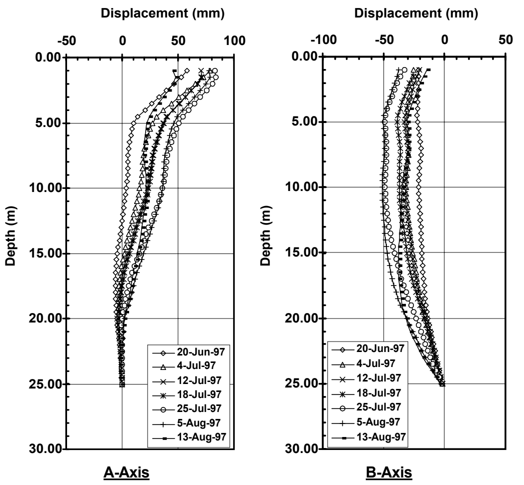

Figure 4a. Inclinometer reading of I-3

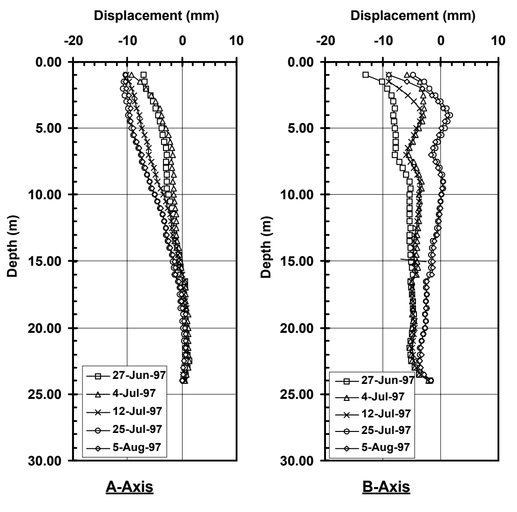

Figure 4b. Inclinometer reading of I-4

However, the wall movements in A-axis direction were considerably large and thus the main beams supporting the edges of the circular enclosure seemed not effective. The two differential movements indicated that the edges of the wall between the straight wall and the circular enclosure had been under torsional stress. It was reported that tension cracks on the capping beam were observed in late July 1997. I-4 recorded wall movements with a maximum of 13mm (Fig. 4b). Movements of the wall toe could not be measured due to lack of the inclinometer tube penetration into the soil below the wall toe level.

After observation of tension cracks on the capping beam, immediate installation of the remaining intermediate struts to support the edges of the circular enclosure and straight wall area were recommended. Occurence of more cracks on the capping ring beam and noise from the bracing system was noticed. At this stage excavation depth inside the circular enclosure had been reached the final depth. However, only two of the five intermediate bracing layers had been installed at the time the IPS collapsed. Figure. 5 shows the photo of the bracing system taken one day before collapse. The IPS collapsed in the morning on 17 August 1997.

After the collapse, soil mass caved in. Several adjacent wooden houses were also damaged and slipped down (Fig. 6a). The soil mass buried the collapsed portions of the wall, the steel bracing, and the excavation equipment (Fig. 6b). Some sections of the wall on the western side collapsed while the remaining wall sections stood in place. The collapse also sheared off the capping beam at the collapse location, causing severe cracking in the capping beam section, and opening panel joints on remaining sections of the wall. No human casualties were reported in the incident as the loud noise prior to the

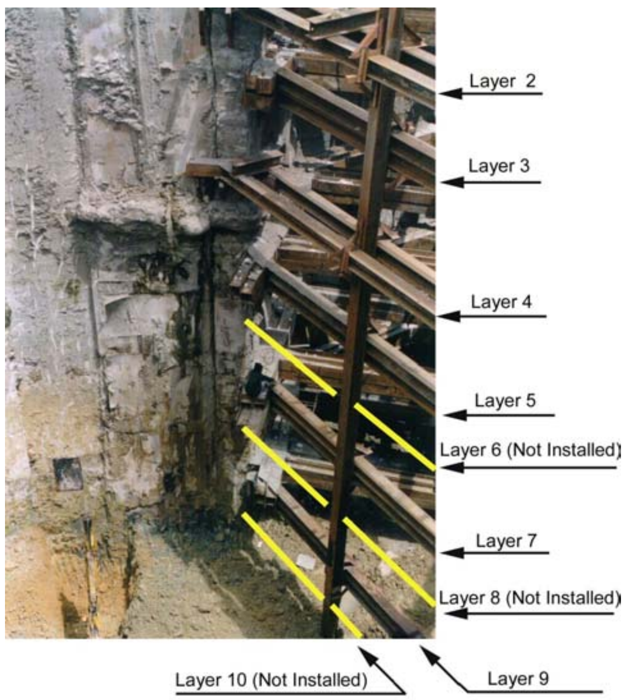

Figure 5. Bracing system seen on 16, Aug. 1997 (after Thasnanipan, 1997)

Figure 6a. After collapse soil mass moved in (after Thasnanipan, 1997)

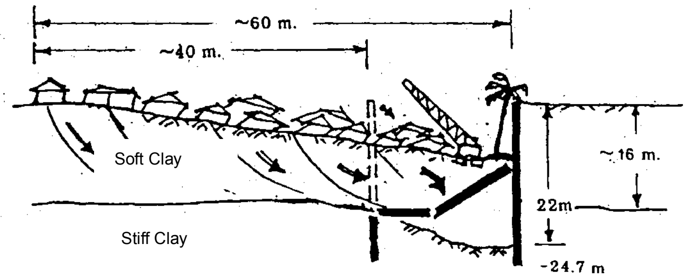

Figure 6b. Soil mass buried everything inside the pit. (after Phien-Wej et al. 1999)

collapse provided warning to the people working inside.

5. CAUSE OF COLLAPSE

The cause of collapse has been investigated by various parties involved. The preliminary causes of the collapse were considered to be:

- Improper shape of IPS permanent structure
- Failure of diaphragm wall
- Failure of internal temporary bracing during excavation
- Failure due to improper sequence of excavation
- Failure due to poor workmanship in temporary works

Three main causes of collapse are identified; the improper shape of the IPS, failure of the diaphragm wall, and improper temporary bracing.

In the case of improper shape of the IPS, similar underground structure had been completed in Frankfurt, Germany (Katzenbach et al. 1998) as shown in Figure 7. Pit excavation was about 20.0m deep and a similar temporary strutting system was also used.

Evidence also indicated that collapse of the IPS was not caused by the diaphragm wall. It was found that the results of analysis of the wall designed by both the designer and diaphragm wall sub-contractor were in conformity with completed projects in Bangkok. The completed projects have been reported by Teparaksa et al. (1998) and Thasnanipan (1998). No defect or leakage was found on the exposed walls. Phien-Wej & Sriruanthong (1999) reported that at the final excavation depth in IPS reconstruction, the toe of the diaphragm wall in the collapsed zone was found in original position and there was no evidence of basal heave.

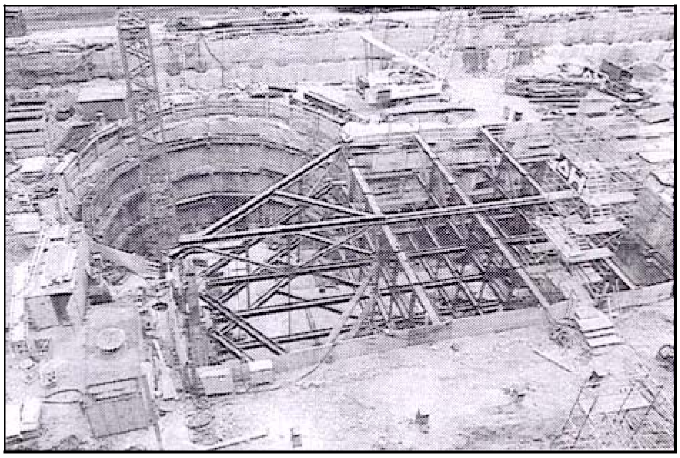

Figure 7. Underground construction of similar shape to IPS (after Katzenbach et al. 1998)

The importance of temporary bracing has been emphasized in various design guides, manuals and reports by Padfied & Mair (1984), William & Waite (1993) and Feld & Carper (1996). In the case of improper temporary bracing, this issue was identified as the main cause of collapse. Results of the analysis by means of FEM (Kanok-Nukulchai, et al. 1998) concluded that the main strut supporting the edges of the circular enclosure was grossly overstressed. The report also stated that these struts would have progressively reached their capacity at the initiation of the collapse and may have become totally ineffective. It was clearly indicated by the photographs taken just one day before the collapse of the IPS (Fig. 8).

Figure 8 shows separations of the built-up walling beams and struts, which would reduce the capacity of the monolithic section by 30%. The three lower intermediate struts required at the edges of the circular enclosure were not present eventhough the excavation works reached the final depth (Fig. 5). This improper bracing also contributed to the cracks observed on the capping beam. Additionally, lateral movements of the walls, shown by inclinometer

Figure 8. Separating of the built-up walling beams and struts in layer 3 (after Thasnanipan, 1997)

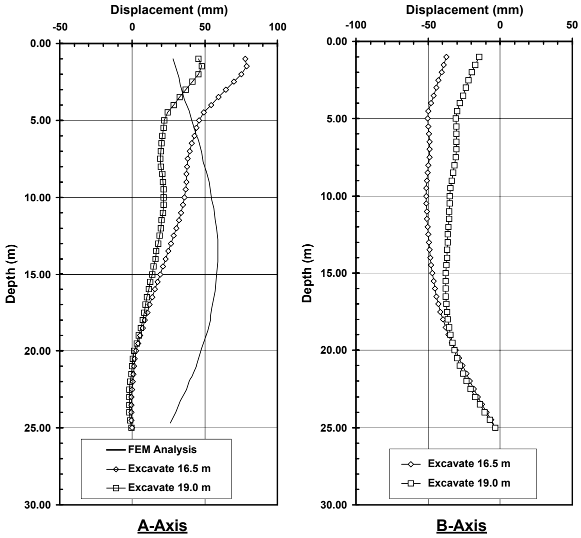

Figure 9. Prediction vs. actual wall behavior (I-3)

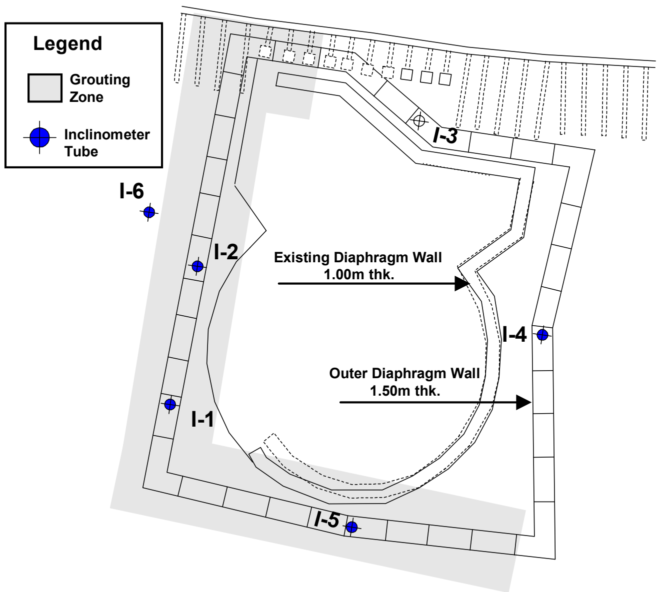

Figure 10. Location of inclinometers and treated area by Jet grouting (after EIT 1999)

monitoring from I-3 indicated that the walls were in cantilever mode. The wall behavior did not reflect predicted behavior indicating inefficiency of the bracing system (Fig. 9). The cracks occurred in the capping beams confirmed cantilever wall movements.

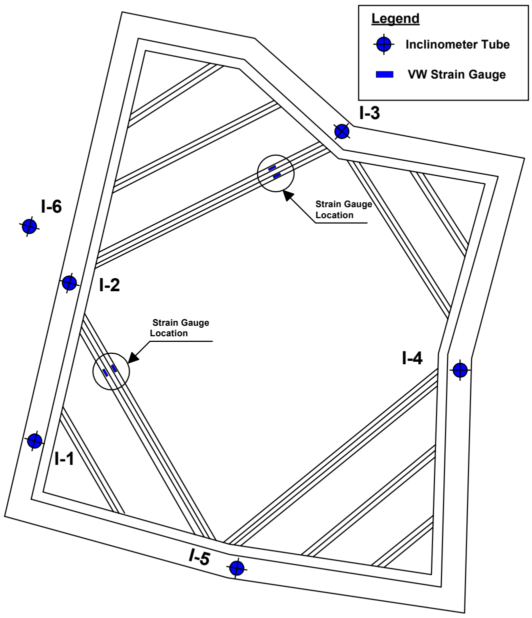

Figure 11. Arrangement of strutting system with stain gauges (after EIT 1999)

6. REMEDIAL ACTION / RECONSTRUCTION

Reconstruction of the IPS was started at the end of 1997. The process involved construction of an outer diaphragm wall perimeter to enclose the collapsed IPS location. The IPS could not be relocated due to presence of affiliated structures. Before starting the outer loop diaphragm wall, Jet Grouting techniques were used to stabilize the soil in the surrounding area for purposes of diaphragm wall construction as well as for excavation works (Fig. 10). Jet grouting and diaphragm wall construction was undertaken by another subcontractor. Jet grouting stabilized only soft clay up to about 16.0m deep. The outer loop diaphragm wall which was designed as a temporary structure was 1.5m thick and about 32.0 m. deep.

During reconstruction, all parties realized the importance of instrumentation. Six inclinometers were installed, five numbers in the wall panels, and one in the jet grouted area in the collapsed zone (Fig. 10).

During excavation, two bracing layers with heavily built-up steel sections were used as diagonal struts at depths of 2.0m and 11.5m (Fig. 11). In both layers, strain gauges were also installed on two main struts and readings were recorded each time a depth of every 2.0m. had been excavated or every 2 days.

Installation and monitoring of (VWSG) on struts provided information on their performance during construction so that necessary action against instability could be taken.

Figure 12. Excavation in progress for IPS reconstruction

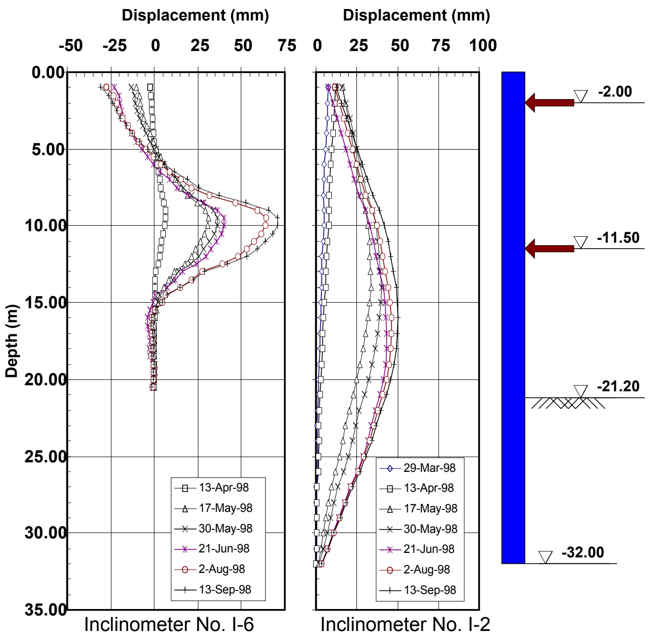

Figure 13. Deflections recorded by I-2 & I-6

Figure 12 shows the status of excavation and strutting before casting the new IPS structures inside the temporary diaphragm wall. The lateral wall movements were recorded by I-2 and I-6 with maximum wall deflections of about 50.0mm 70.0 mm respectively (Fig. 13). The reconstruction work was thus successfully completed with proper instrumentation.

7. CONCLUSIONS & RECOMMENDATIONS

It was learnt that ignorance of instrumentation for deep underground construction can cause a serious disaster.

Adequate instrumentation should be provided for deep excavation to avoid remedial work.

Monitoring and maintenance of instrumentation should be carried out throughout the work.

Parties involved in deep underground construction should have enough knowledge in geotechical works and instrumentation.

Proper instrumentation enabled the successful reconstruction of IPS.

8. ACKNOWLEDGEMENT

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