

PIPING FAILURE OF A COFFERDAM IN SOUTHERN TAIWAN

by

S. W. Duann, C. H. Wang, R. F. Wang and G. J. Liou

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Piping failure of a cofferdam in southern Taiwan

Shaw-Wei Duann

Moh and Associates, Inc., Taiwan, ROC

Rei-Fung Wang

Taiwan Power Company, Taiwan, ROC

Chien-Hong Wang

Moh and Associates, Inc., Taiwan, ROC

Gwo-Jenn Liou

Moh and Associates, Inc., Taiwan, ROC

SYNOPSIS: Piping failure occurred in an open excavation in southern Taiwan despite the fact that a satisfactory factor of safety was obtained by analyses. The problem appears to be associated with the presence of a thin clay blanket below the bottom of excavation, together with the rising of groundwater table due to waves and heavy rains brought by a typhoon. This paper describes the sequence of the events, the failure mechanism and the remedial measures taken.

1. INTRODUCTION

Piping failure occurred in a braced excavation for constructing an inlet house in southern Taiwan. The Inlet House was to be constructed for expanding the circulation water system for a steam power plant. After the passing of a typhoon which brought the largest precipitation in the last 30 years, piping failure occurred and a total of 3,000m³ of sand and 14,000m³ of water rushed into the pit, resulting in serious ground loss outside the excavation.

2. THE SITE

As shown in Fig. 1, there were two excavations in this L-shape construction site, one for the Inlet House and one for the Pump House. The excavation for the Inlet House, which is the subject for discussion, was 78.4m in length, 19.5m in width and 13.75m in depth. It is separated from the Main Channel of Kaohsiung Harbor on its west by a pier of 2m in width and from Channel No. 5 on its south by a pier of 8m in width. On its east, an existing factory was in the vicinity and the excavation for the Pump House was only 6m away.

3. SUBSOIL CONDITION

Based on the logs of two boreholes located in the middle of the excavation, the subsoils

within a depth of 33m below the ground surface mainly consist of silty medium fine sand with occasional traces of sandy silty. The results of standard penetration tests were:

<u>Depth</u>	<u>N-values</u>	<u>Average</u>
0 to 5m	3 to 4	
5 to 23m	10 to 20	14
23 to 33m	> 20	23

Below the depth of 33m, the subsoils consist of stiff to very stiff silty clay layer with occasional silty sand and sandy silt seams.

4. RETAINING SYSTEM AND INSTRUMENTATION

The layout of retaining walls is shown in Fig. 1 and a cross section of the west pier is shown in Fig. 2. The entire construction site is enclosed by soil mixing walls (SMW) of 0.6m in thickness and 32m in depth. In addition, a row of sheet piles (YSP IV) of 17m in length was installed to enclose the two individual pits. The SMW walls were reinforced by H-piles of 19m in length. The H-piles were spaced at 0.45m intervals on the west and the south sides of the excavation for the Inlet House and at 0.9m intervals on the north and the east sides.

As depicted in Fig. 2 the piers along the two

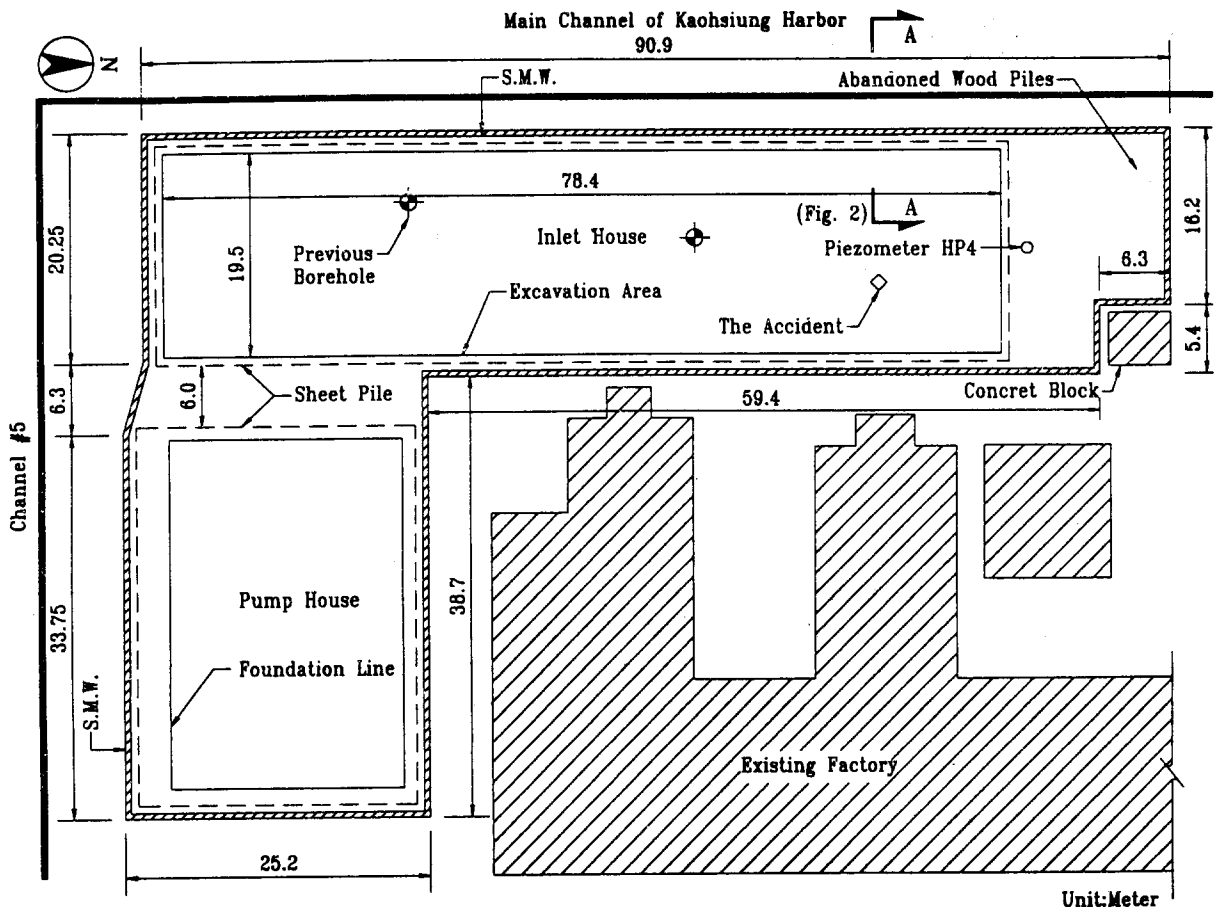


Fig. 1 General layout of site

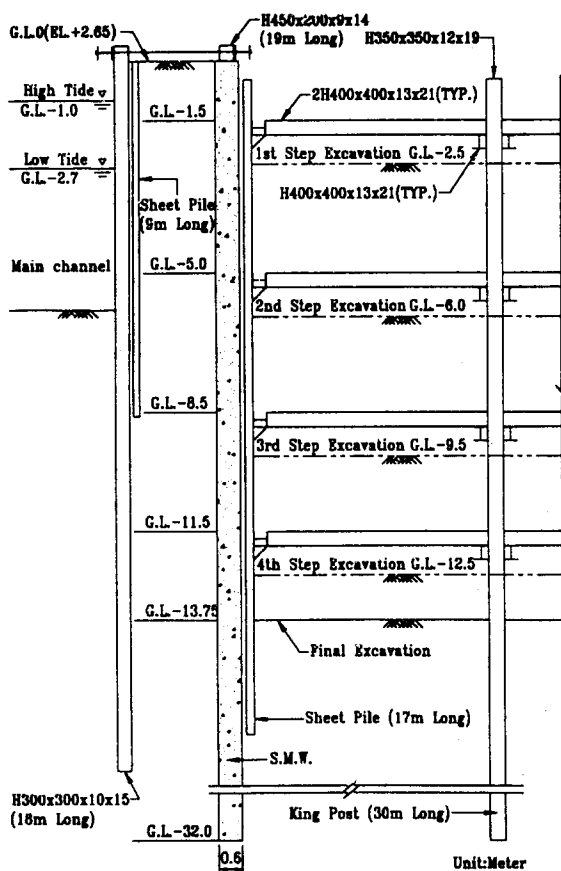


Fig. 2 Cross section A-A

Channels were protected by sheet pile walls, 9m in length. To ensure safety during construction, steel H-piles, 18m in length, were driven at spacings of 2m alongside these sheet piles. They were tied to the SMW by steel rods at the top and the piers essentially became a double-wall cofferdam. The excavation for the Inlet House was braced by four levels of struts. The king posts supporting the struts were 30m in length and the piles supporting the platform, refer to Fig. 3 for locations, were 36m in length.

The instrumentation system consisted of settlement points for ground settlements, heave indicators for vertical movements of king posts, pneumatic piezometers for groundwater levels, inclinometers for lateral deflection of retaining walls, and strain gauges for strut loads.

5. THE EVENTS

The excavation had already been completed prior to the incident. Based on field records, water started to run at a location, refer to Fig. 3, near a platform support shortly after a typhoon

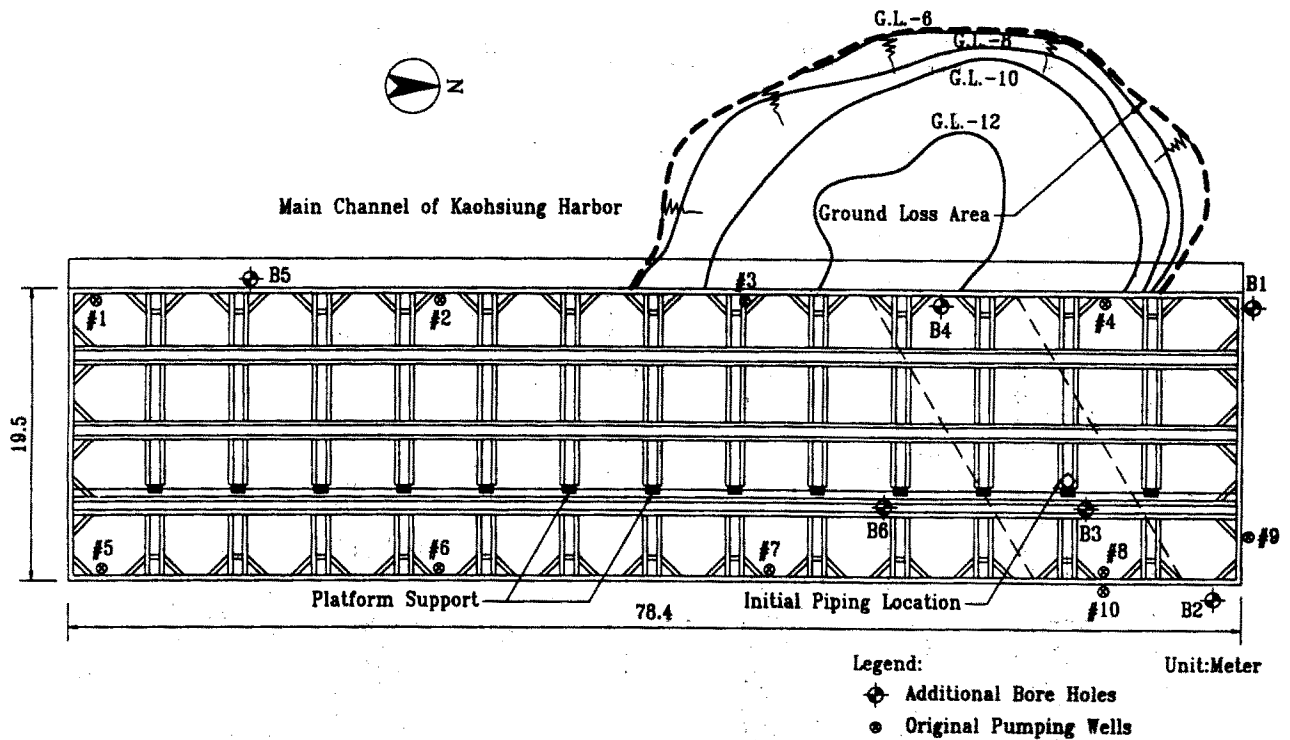


Fig. 3. Extent of sinkhole and locations of additional boreholes

passed. A volume of 40 to 50m³ of sand was found to have come out with the water. The excavation had already been completed and the formation level reached. On the day when this occurred, the rain was the heaviest in the past thirty years and heavy rain continued for four more days. The total rainfall in the 5-day period accumulated to 896mm. Immediately after the rain stopped, three pumping wells were drilled in the immediate vicinity of the location where piping occurred, attempting in vain to lower the ground water table and to stop the piping. Piping was finally stopped by pumping from a new well, 40m in depth, at the northeast corner outside the pit (Location #10 in Fig. 3).

After the plain concrete layer was cast and 80% of reinforcing bars for the mat foundation were laid, a blasting sound was heard in a midnight, approximately half a month after the problem first occurred. A portion of the west pier, together with H-piles and sheet piles, collapsed and fell into the Main Channel. About 15 minutes later, sea water rushed into the pit, and the water in the pit quickly rose to G.L. -4.5m.

A reconnaissance was made in the following morning. A 44-m section of the west pier was found to have collapsed and more than 3000m³ of sand had intruded into the excavation piling to a maximum height of 3.8m above the bottom. Sounding revealed a large sinkhole outside the pier as shown in Fig. 3. Fortunately, no one was hurt and there was no structural damage to the retaining system or to the adjacent factory. All the instruments were still functional and the readings were within the pre-set tolerances.

6. THE CAUSES

The SMW retaining walls were 32m in depth, and the safety factor against piping, according to NAVFAC DM-7.1(1982), was greater than 2. Therefore, the accident should not be attributed to insufficient penetration of the retaining walls. Yet, failure still occurred. The failure mechanism deserves a thorough investigation.

For exploring the ground conditions and figuring out the causes of failure, six boreholes, B1 to B6, were drilled. Their locations are shown in Fig. 3 and boring logs are available in Fig. 4. As can be noted, the

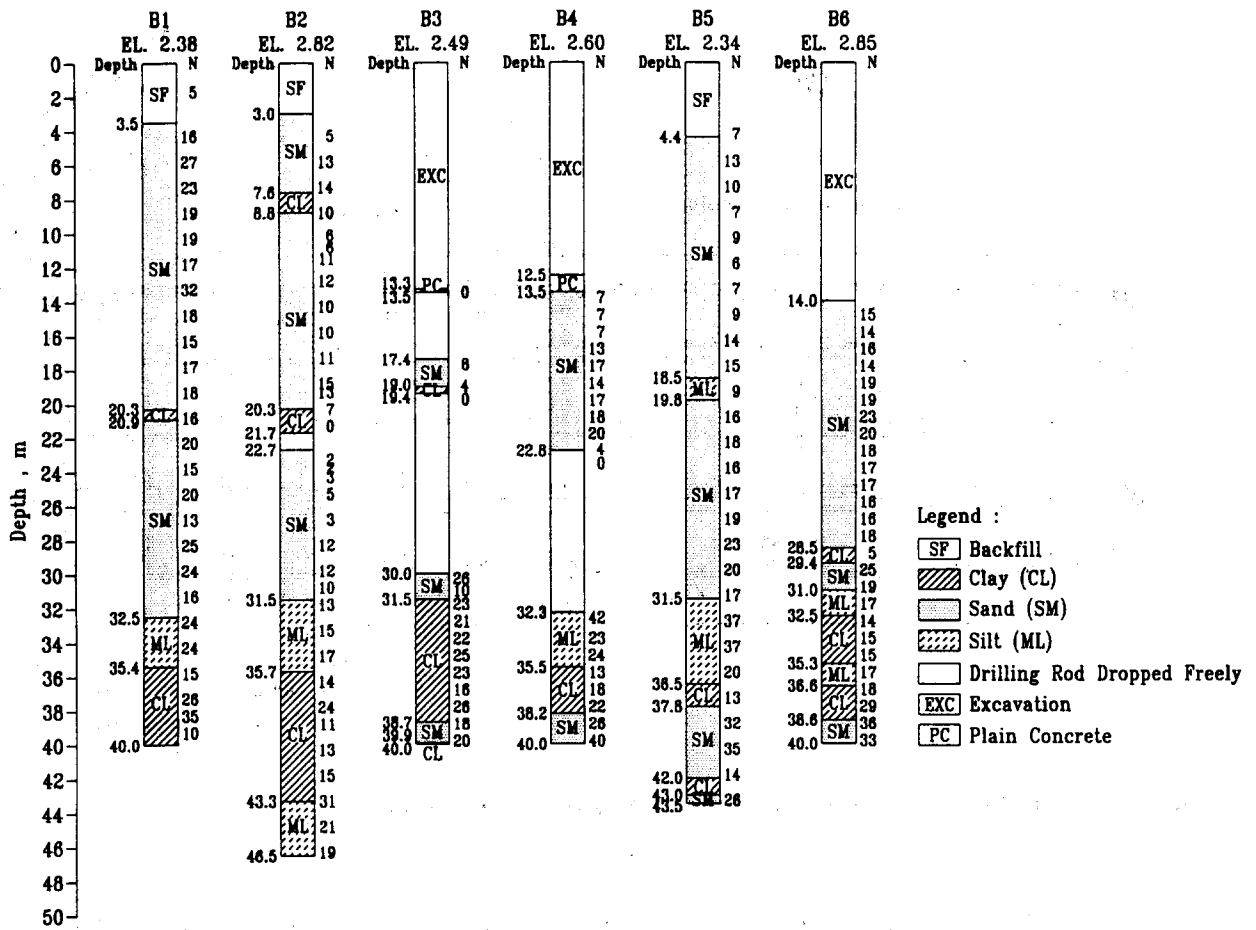


Fig. 4. Boring logs

SPT N-values were zero at some depths in Boreholes B2, B3 and B4, indicating that the subsoils were seriously disturbed. A schematic view of the failure is given in Fig. 5.

In Borehole B4, which is the nearest to the center of the sinkhole, there appeared to be a 0.3m section of disturbed zone below the bottom of the SMW, the toe of which was at a depth of 32m. This indicates that a tunnel had been formed and become a water path leading to the failure. Since this borehole is not necessarily right at the middle of the course, it is conceivable that the size of the tunnel could be much larger, say, of an order of 1m to 2m.

The groundwater table below the bottom of excavation was regulated by pumping at 9 wells of which the locations are shown in Fig. 3 (Locations #1 to #9). The wells extended to a depth of 28m and were open along their depths. The boring logs of Boreholes B1 through B3 revealed a clayey

layer, 0.4 to 1.4m thick, at a depth of 20m or so. Unfortunately, the continuity of this thin clay layer could not be verified in Boreholes B4 through B6. This clay layer practically became an impervious blanket separating the subsoils into two regions. As illustrated in

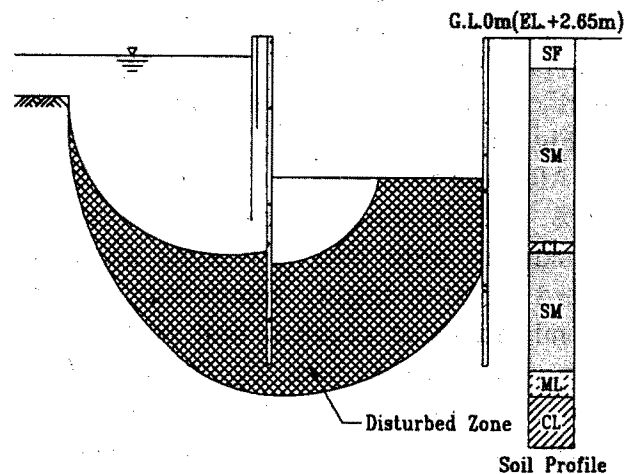


Fig. 5. Schematic drawing of the failure

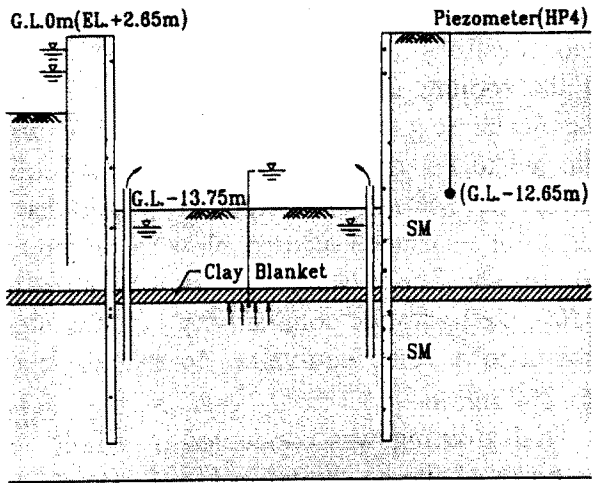


Fig. 6 Water head below the clay blanket

Fig. 6, in the upper region, water was free to flow into the wells by gravity. The lower region was concealed at the top by the clay blanket and was pressurized by the seepage flow from the bottom. The pressures were released at wells on the two sides. It is not difficult to realize that the water head at the mid-section was greater than the water heads near the walls.

Two-dimensional finite element seepage analyses showed that, although the water levels in the wells were maintained at a level of 2m below the bottom of excavation, the pressures acting on the underside of the clay blanket would be of an order of 100 kPa at the mid-section. The weight of soil above the blanket was roughly 114 kPa for a unit weight of 19 kN/m³ and a thickness of 6m (i.e., 19.5m-13.5m). Therefore, the factor of safety against blow-in at mid-section is very marginal. In fact, the problem is three dimensional, rather than two-dimensional. The wells were point sinks rather than drainage trenches as implied in the two-dimensional analyses. The pressures at the two ends and at the center of the pit would have been much greater than what was computed. Bauer *et al.* (1980) had emphasized such a phenomenon which should properly be considered during the design of cofferdams.

In addition to blow-in, at locations where the blanket is thin or is entirely missing,

piping is likely to occur. Piping might also occur through fissures in the blanket created as a result of driving of king posts and platform supports.

The problem was aggravated by the waves and the heavy rain accompanying the typhoon. It is conceivable that the waves would be a meter or two above the high tides which were the basis of design. The readings of Piezometer HP4, the location of which is shown in Fig. 1, installed in the compartment bound by SMW and sheet piles to the north of the excavation are shown in Fig. 7. As can be noted, the water level was 1m higher than the previous elevation before piping first occurred. Unfortunately, HP4 was installed above the clay blanket (at G.L. -12.65m) and the readings do not represent the water head in the lower region. It is even more pitiful that there were no piezometers available inside the pit. It is quite possible that the pressures acting on the underside of the clay blanket increased as the groundwater level rose. It is also possible that the pumps were incapable of coping with the additional surface water during the heavy rain and the water levels in the wells could no longer be maintained at the desired level (i.e., 2m below the bottom of excavation).

7. REMEDIAL ACTIONS

It was found that the two rows of 17m sheet piles between the two excavations, referring

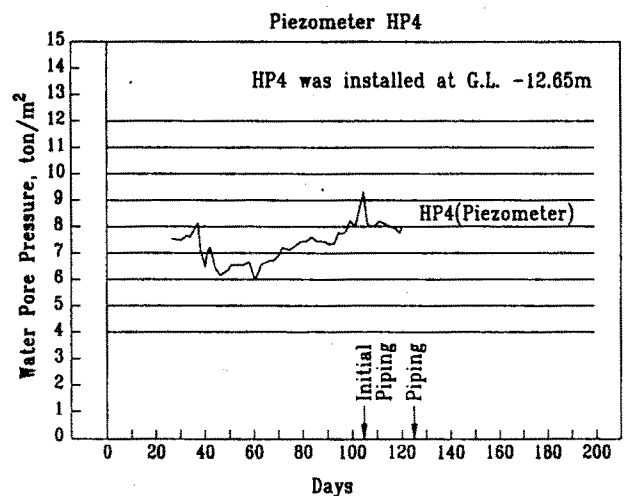


Fig. 7. Ground water levels

0.3 MPa or ground heave was observed. It was found that the initial grouting pressure was consistently low (0-0.2 MPa) in the disturbed zones. On the other hand, the grouting pressure increased rapidly beyond 0.3 MPa in the undisturbed zones.

A total of 1,317m³ of grout was injected in these 28 holes. Hole A17 alone took 292m³ while the three neighboring holes, A13, A19 and A20, took 86m³, 83m³ and 85m³ respectively, although they were grouted prior to A17. It is therefore evident that Location A17 must be where piping was the initiated

and where the cavity was deepest.

7.1.2 Stage 2-ground improvement

Grouting was carried out in 50 holes in the failure zone, and 100 holes along the west side. The locations of these grouting holes are shown in Fig. 8. A fixed quantity of grout was injected at each depth at a rate of 10 liters per minute using the double-packer grouting method. In each hole, grouting was conducted in two steps as follows:

(1)CB grouting to 15% of the volume of the ground to be treated with each cubic meter of grout containing 918 liters of water, 200kg of cement and 45kg of bentonite.

(2)SL grouting to 25% of the volume of the ground to be treated with each cubic meter of grout containing 714 liters of water, 51 liters of diluted sulfuric acid and 235 liters of sodium silicate.

A total of 31 in-situ tests were performed and the average of the hydraulic conductivities obtained was $(4.6 \pm 3.0) \times 10^{-8}$ m/sec. In two holes, where the grouting pressures did not reach 0.3 MPa, the values obtained were of the order of 1.3×10^{-7} m/sec which was slightly greater than the design criteria of 1.0×10^{-7} m/sec. Additional chemical grout was injected locally and the hydraulic conductivity was reduced to 3.2×10^{-8} m/sec.

7.2 Groundwater control

Only one (#2) of the 9 original pumps could still operate normally. Therefore, 22 wells of 300mm in diameter and 35m in depth was drilled and equipped with a pump of 5hp in each well. The locations of these wells are shown in Fig. 8 (W1 to W22).

Four electrical type piezometers, SP1 to SP4, were installed to a depth of 23.85m to monitor the groundwater level inside the excavation. It was observed that the groundwater level was maintained at 0.8 and 2.0m below the excavation level.

8. CONCLUSIONS

In summary, the failure could have been a result of one of, or a combination of, the following:

1. The clay blanket blocked the seepage

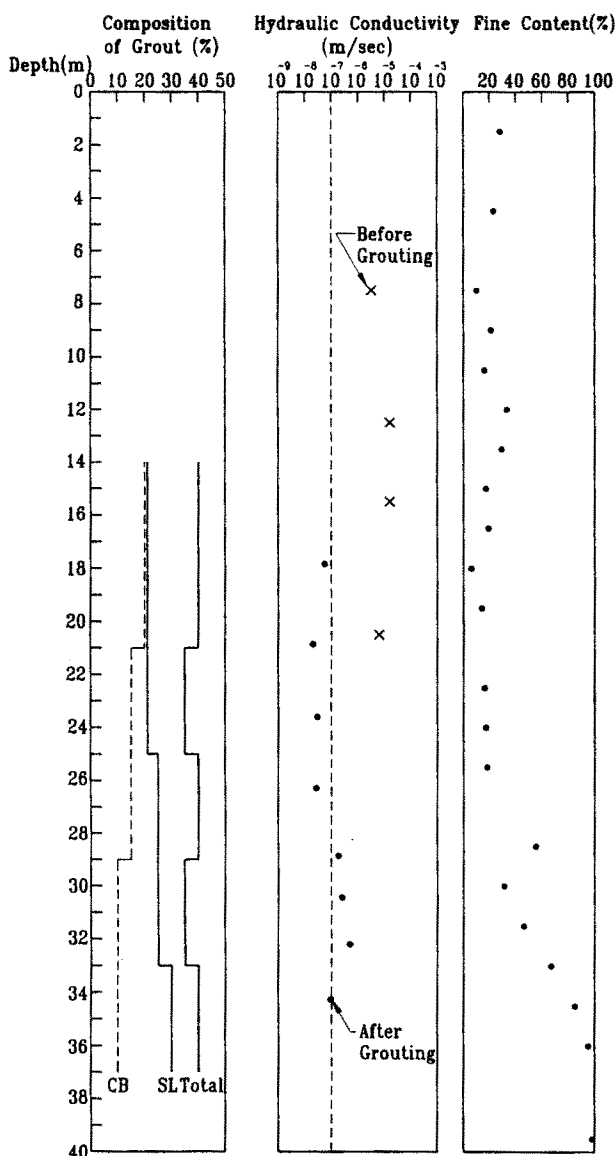


Fig. 9 Hydraulic conductivity, quantities of grout injected and fine contents of soil

1. The clay blanket blocked the seepage flows and built up pressures on its underside. Piping might have occurred at where the blanket was thin or was missing as a result of hydraulic fracturing.

2. Driving of king posts and platform supports damaged the clay blanket and the fissures, if any, would be water paths leading to piping and erosion.

3. The problem was aggravated by the rising groundwater table due to heavy rain.

4. Insufficient pumping capacity led to the rise of groundwater level below the bottom of excavation.

This case history provided an excellent example illustrating the risks involved in deep excavations and the importance of geotechnical engineering and instrumentation.

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