

Collapse of Braced Excavation in Singapore



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ABSTRACT: After the collapse of the subway excavation in the Nicoll Highway, extensive in-situ investigations were carried out to identify causes of the accident. The results of detailed studies disclosed that lack of toe-in depth of the diaphragm wall into the stiff competent strata could be one of the causes of the incident. This aspect of the studies is highlighted in this paper.

COLLAPSE OF BRACED EXCAVATION IN SINGAPORE

Outline

At a site of braced excavation for a cut-and-cover tunnel construction in Singapore, a large scale collapse occurred on April 20, 2004 over the length of 100m section of a carriage way called Nicoll Highway which is located in the southern coast of the Singapore Island as shown in Fig. 1. A closer map of the location is shown in Fig. 2. The construction south of Nicoll Highway Station was underway involving deep braced excavation flanked by diaphragm walls. The depth of excavation at the time of the failure was 30.8m. The diaphragm wall 0.8m thick was braced by the steel struts with the help of walers and splays. The collapse took place when excavation was underway below the 9th step of struts already in place.

As a result, the soil mass moved towards the excavated space involving large slides on both sides accompanied by the subsidence in the surrounding area. The site is underlain by thick deposits of soft marine clay to a depth of about 35m. Following the incident, detailed studies have been made including borings, sounding, sampling, lab. testing and numerical analyses simulating conditions at the time of the failure. The outcome of the studies was reported by Tan (2006), Yong (2006) and Yong and Lee (2007). Although the probable causes of

the incident were pointed out, the present paper will focus on one of the likely causes which are associated with poor understanding of soil conditions in the extremely soft soil area in Singapore.

Collapse of the strut-supported excavation

A section of temporary retaining wall structure adjacent to Nicoll Highway collapsed almost suddenly within a matter of a few minutes at 3:30 p.m. on April 20, 2004. An overview of the site of failure is shown in Fig. 3. This is located at the seaward side of the land reclaimed around 1970. The features of the braced excavation and collapse are roughly shown in the plane view of Fig. 4 where it is noted that a large tunnel access shaft (TSA) 17m in radius had been constructed to facilitate transport of structure elements and materials down to the floor of the excavation.

Method of construction

The section of the cut-and-cover tunnels were under construction by means of the bottom up method with two lines of diaphragm walls supported by strut-waler-splay system. The thickness of the diaphragm wall was 1.0m at the location of cable crossing and at the end section next to the access shaft. Otherwise the thickness was 0.8m. One element of the wall each 3 to 6m wide was excavated in slurry by means of a basket-type digging machine in two or three bites. After lowering the rein-

forcement cages, concrete was poured in by Tremic pipes. Joints of neighboring elements of the wall was sealed against water penetration, but not rigidly connected as structures. The diaphragm walls, over the section Type M3, were for the curved stretch closest to the access shaft and constructed in May 2003. The walls were designed to be extended 3m into the stiff base layer called Old Alluvium (OA-layer). A detailed plan view of the M3 section is shown in Fig. 5 and a typical side views of the strutting system for the cross section A-A' is displayed in Fig. 6.

The diaphragm walls were supported by the 10 levels of steel struts with discontinuous walers except at the top strut. To reinforce the strutting action, two soil layers at the depths of 28 to 29.5m (sacrificial strut) and of 33.5m to 36.5m were stabilized as shown in Fig. 6 by means of the jet grouting which had been installed from the ground surface before the excavation began. At the same time, with an aim of providing support for the railway tunnel structures to be placed later, four lines of bored piles each 1.4 to 1.8m in diameter were installed from the ground surface at a spacing of 4m or 6m as shown in Fig. 6. The depth of the pile embedment was 34.2m on the north side counted downwards from the elevation of 74.2m RL and, it was 29.6m RL on the south side from the elevation of 69.6m RL as shown in Fig. 6, where RL (Reduced Level) means a local measure of elevation with a zero point taken at a depth 100m from the mean sea level.

Situations around the time of the collapse

Excavation with strutting had gone without any hitch until the 9th strut was installed. It was envisioned at this stage that the entire lateral load near the bottom of the excavation was surely transferred to the 9th strut. Then, the excavation to the 10th level of strut was started, thereby taking out the jet-grouted portion 1.5m thick. At this stage, the struts located in this stretch were S333 to S340 with a horizontal spacing of about 4m. The width of the excavation in plan was about 20m and the depth was 30.8m. When the excavation had progressed to the location below the strut No. S332 as indicated in Fig. 5, the collapse started to occur. That was at 3:33 p.m. on April 20, 2004. The pictures after the collapse, at 3:34 p.m. and at 3:41 p.m. are displayed in Fig. 7. These photos looking over to the east were taken from the window of the apartment building just north of the Nicoll Highway.

In the morning of April 20, from 9:00 a.m. to 10:00 a.m. engineers on the spot is said to have heard sounds of metallic collision near the north side of the strut S338 and observed the inner flange of H-beam waler at the 9th strut having buckled and shifted about 5 to 6cm downward. This shift aggravated to about 20cm by 2:30 to 2:45 p.m. It was also witnessed around this time that the inner flange of the waler at the 9th strut S335 on the south side had also buckled. Between 2:00 p.m. and 3:00 p.m. the sounds of metallic collision were heard more frequently and eventually the collapse took place at 3:33 p.m.

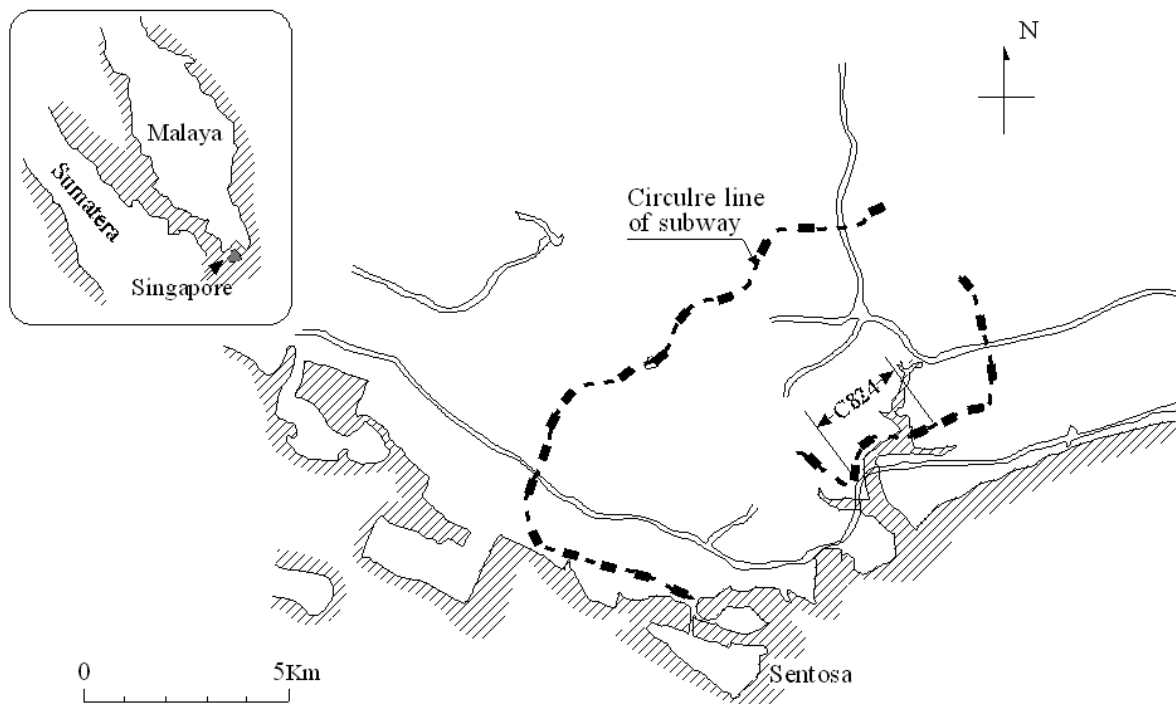


Fig.1 South part of Singapore.

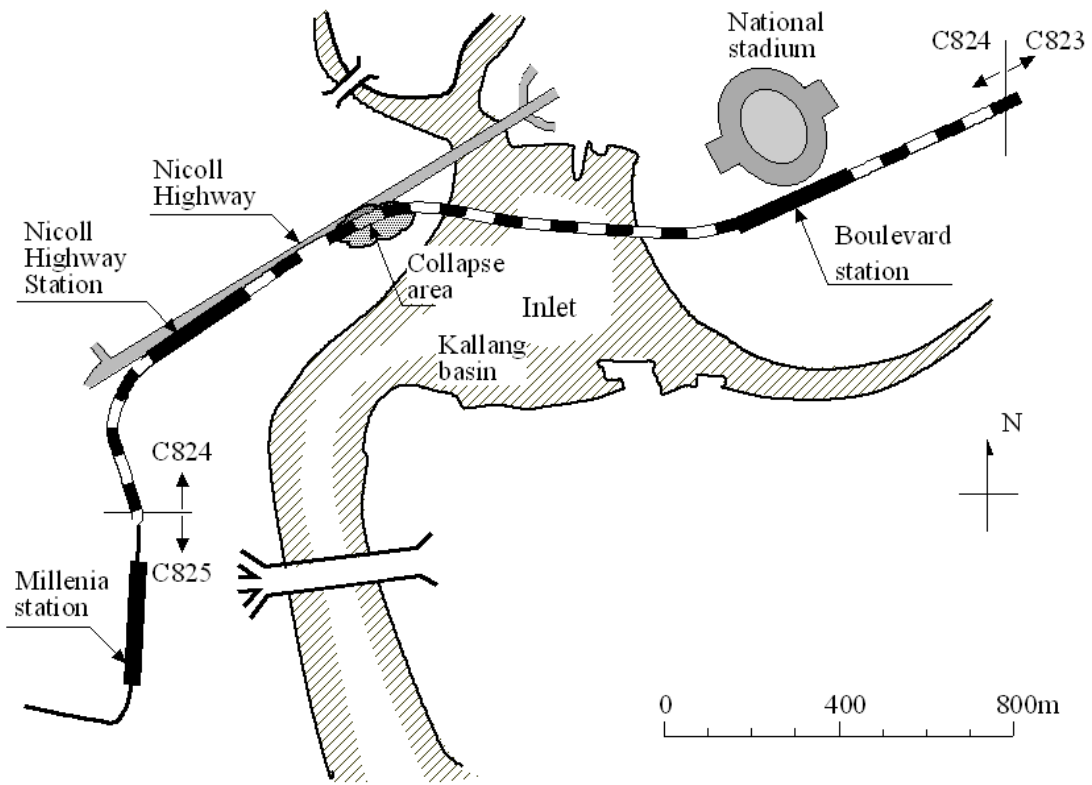


Fig. 2 Place of the collapse in the subway construction line.



Fig. 3 Oblique view of the site to the east after the collapse.

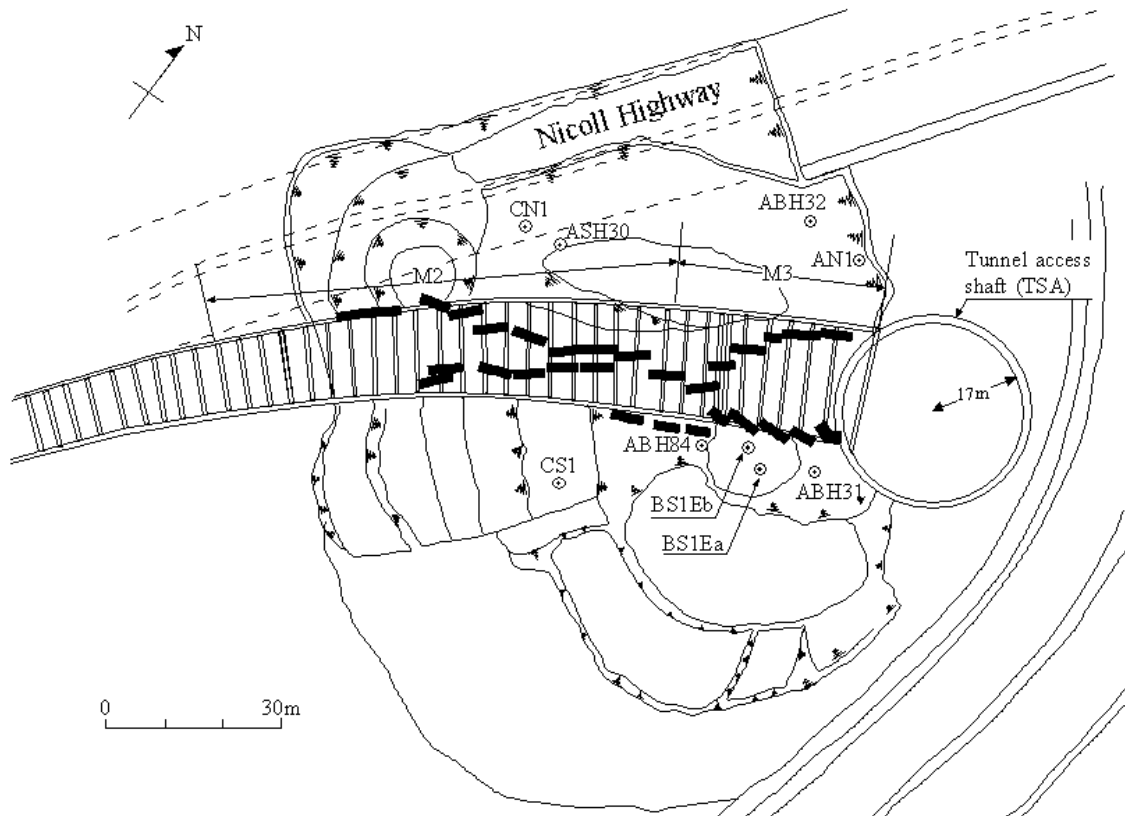


Fig.4 Plan view of the area of the failure and braced wall system.

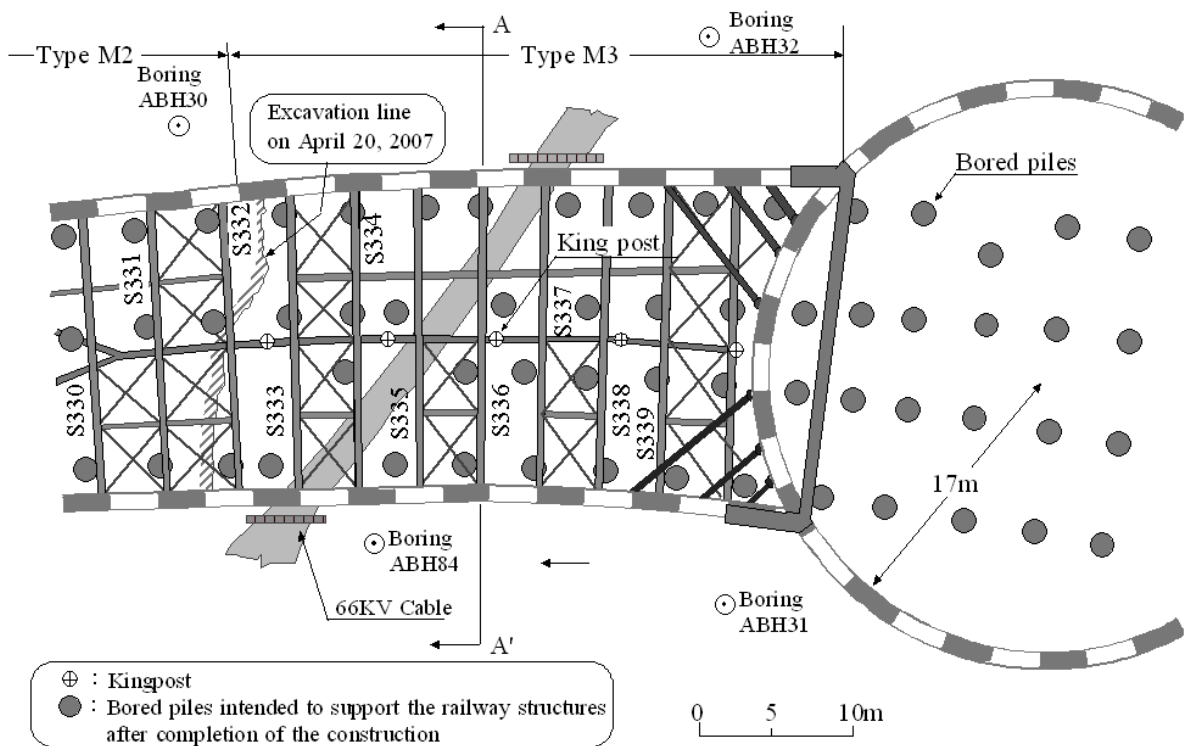


Fig. 5 Plan view of the Type M3 section due west of the access shaft.

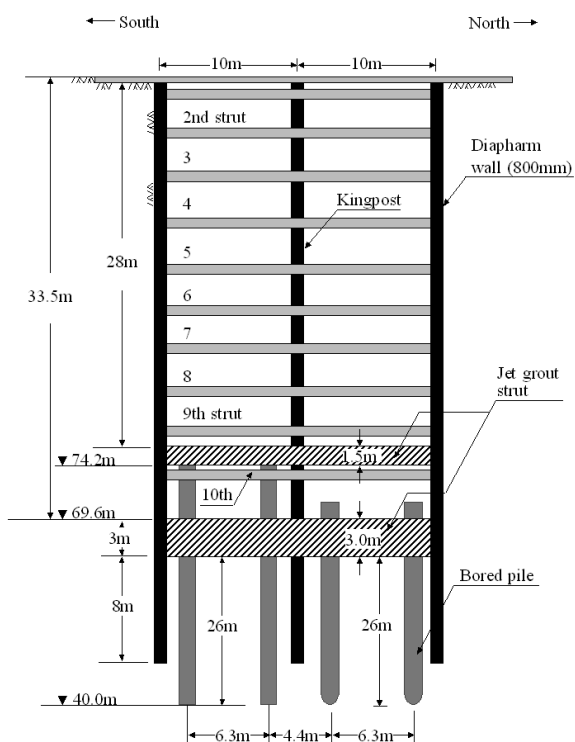


Fig. 6 A typical side view of the struted system for the A-A' cross section.



Fig. 7 Photos showing the incident at 3:34 p.m. and 3:41 p.m.

Conceived causes of the failure

After the occurrence of the collapse, the Committee of Inquiry (COI) was established and probable causes were investigated and pointed out. In May 2005 the COI report was released, thereby indicating the main reasons as described below.

- (1) Under-design of the diaphragm wall using an inappropriate method of analysis.
- (2) Under-design of strut-waler connection, particularly in the curved section due west of the SA shaft.
- (3) Incorrect back analysis and problems with instrumentation and monitoring for conducting the observation-led construction control.

These would have been combined to induce the failure and detailed account is described for each of the above items in the papers by Tan (2006), Yong (2006), and Yong and Lee (2007). It is not the intention of this paper to debate on these items. The purpose herein would rather be to focus on features of soil conditions which have not been thoroughly discussed, but unearthed more recently through further investigations in details.

Soil conditions at the site of the incident

In the design stage before the incident, soil conditions were investigated by means of boring, sounding, and recovering undisturbed samples and testing in the laboratory. The locations of borings, standard penetration tests (SPT) and cone penetration tests conducted at the design stage in the vicinity of the failure zone are shown in Fig. 8 with symbols ABH. Also designated by AC in Fig. 8 are the places where the cone penetration tests (CPT) were carried out. Cone tests were also performed at the locations denoted by MC2026, 3006 and 3007.

The results of the boring and SPT N-values at the site ABH30, 31, 32 and 84 are shown in Fig. 9. It may be seen that the recent fills exist to a depth of 3 to 5m underlaid by a thick deposit of soft marine clay, M, having a SPT N-value of zero. In the middle of this clay deposit, there exists a stiff silty clay layer of fluvial origin about 3m thick at the depth of 18 to 20m. This layer has a SPT N-value of 10 to 15 and denoted by F₂.

The deposits M and F₂ are called Kallang Formation. Under the Kallang Formation, there exists a stiff silty sand layer having variable SPT N-value from 10 to greater than 100. This layer is an old alluvium and denoted by OA. Between the marine clay M and the OA layer, there exists occasionally the estuarine deposit, E, consisting of organic soil and also the fluvial layer F₂.

As a result of physical and mechanical tests, the marine clay was found to possess the plasticity index of 30 to 50 and compression index of C_c=0.9. The undrained shear strength was shown to be in

the range of $S_u=20\sim 60\text{kPa}$ at depths of 40m and the clay is still slightly under-consolidated state with $\text{OCR}=0.8\sim 1.0$. It was considered important to determine the elevation of the Old Alluvium (OA), as it was defined as the base of the bottom deposit into which the diaphragm wall was to be embedded. In the design, the toe of the diaphragm was set to be 3m below the top of the OA layer.

In view of its importance, the contour lines indicating equal elevations of the top of the OA layer were also drawn in Fig. 8, on the basis of the information available at the design stage. It is noted in Fig. 8 that the distance between two adjacent boring logs was no shorter than about 20m and consequently the buried valley near the southern wall close to the access shaft was not identified. This fact turned out to be vital for creating the worst situations to occur afterwards.

Post-collapse site investigations for forensic diagnosis

After the incident, no attempt was made to dig out the debris buried within the excavated space, because a new route for the subway construction was laid down at the southern side. Thus, the excavated space was buried instead by dumping soils and then in-situ boring and soundings were carried out. These included also the investigation of the post-collapse conditions of the broken pieces of the diaphragm wall panels, construction machines buried within the excavated space.

(a) Borings

Outside the zone of the excavation several series of boring and soundings were performed to depths of about 40 to 50m corresponding to an elevation of about 50m RL (Reduced Level). Nearest the tunnel access shaft, a pair of boring, AN1 and AS1 was performed at sites north and south, respectively, of the excavation. Further westward, another pair of boring was conducted at the locations BN1 and BS1. When the drilling hit an obstacle before reaching a targeted depth, another hole was drilled. Likewise, still other pairs were conducted westwards as indicated by the heading letter C, D, E and F in Fig. 10.

Some of the results of the borings conducted at points BS1Ea and BS1Eb about 7m south of the collapsed wall is displayed in Figs. 11(a) and (b) where it can be seen that remains of the grouted cement were encountered at an elevation of 73m and 67m RL. In another boring at CS1 shown in Fig. 11(c), the soil stratification was found to be about the same as that before the failure event.

(b) Magnetic logging test

This test was intended to detect ferrous metal obstructions such as broken pieces of walers, struts or construction equipments buried in the debris within the sliding zone. The method consists of incremental lowering or rising of a magnetic probe through a borehole and to detect the magnetic reaction at varying depths. During the boring, the kind of soils was identified permitting depthwise pictures of soil profiles to be established.

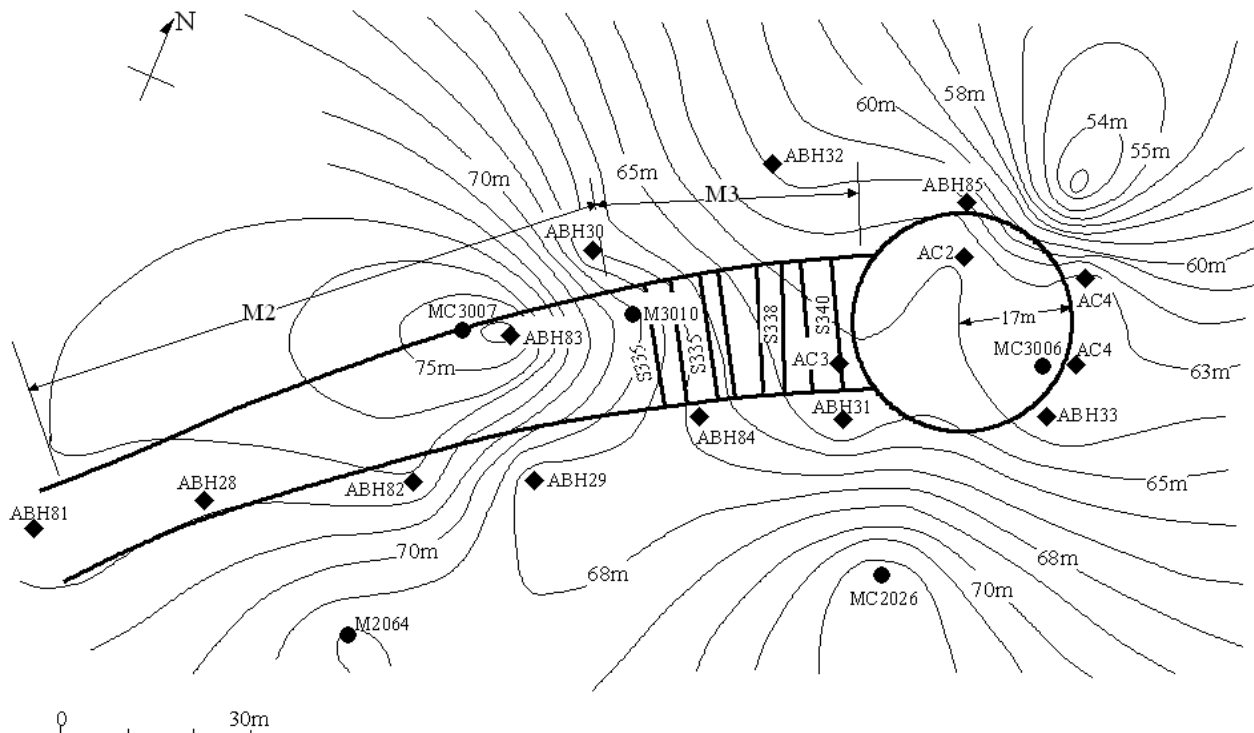


Fig. 8. Contours of elevation at the top of the old alluvium (OA-layer) based on boring data at the design stage.

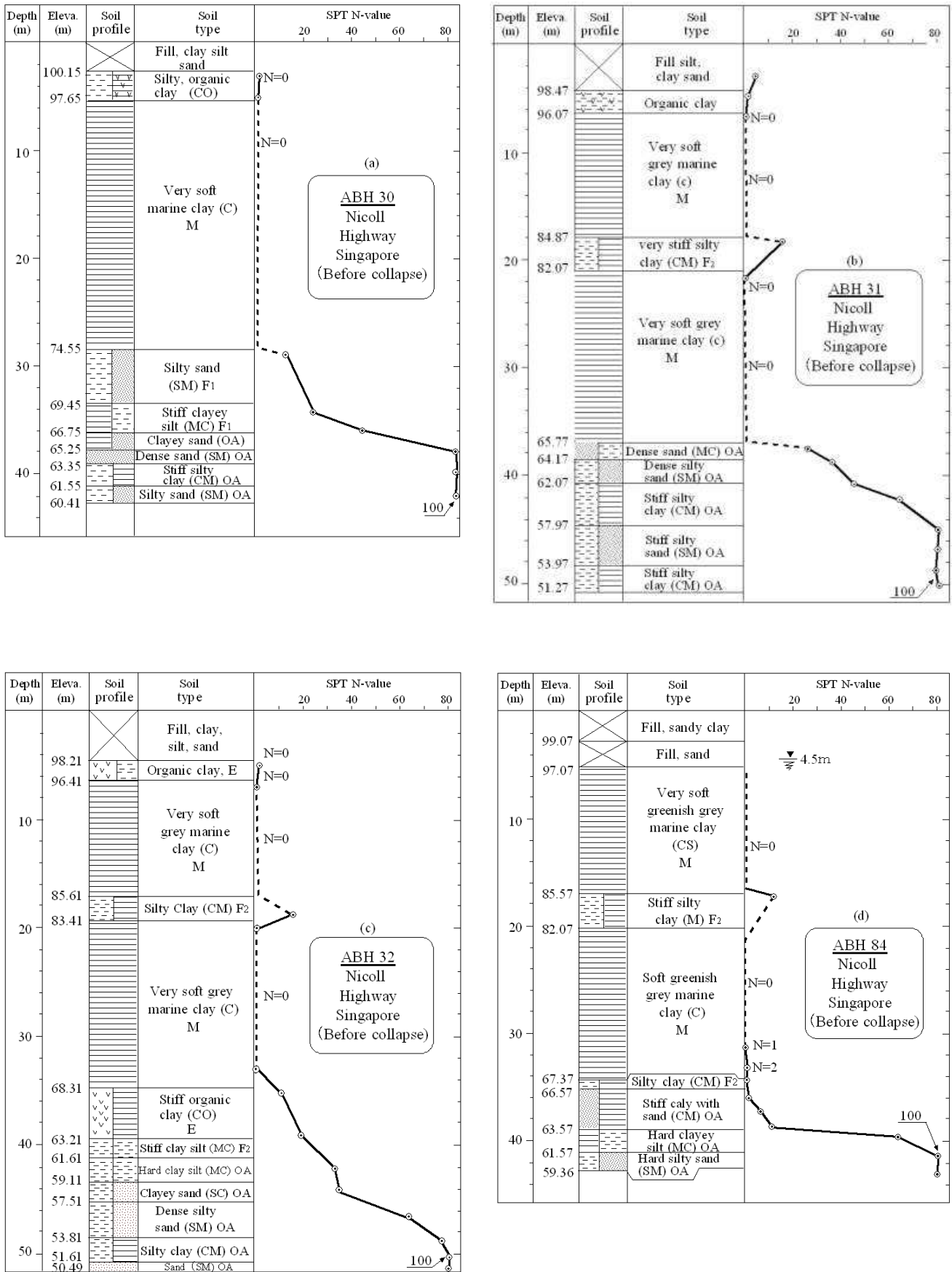


Fig. 9 Soil profiles at the sites near the excavation obtained before the collapse.

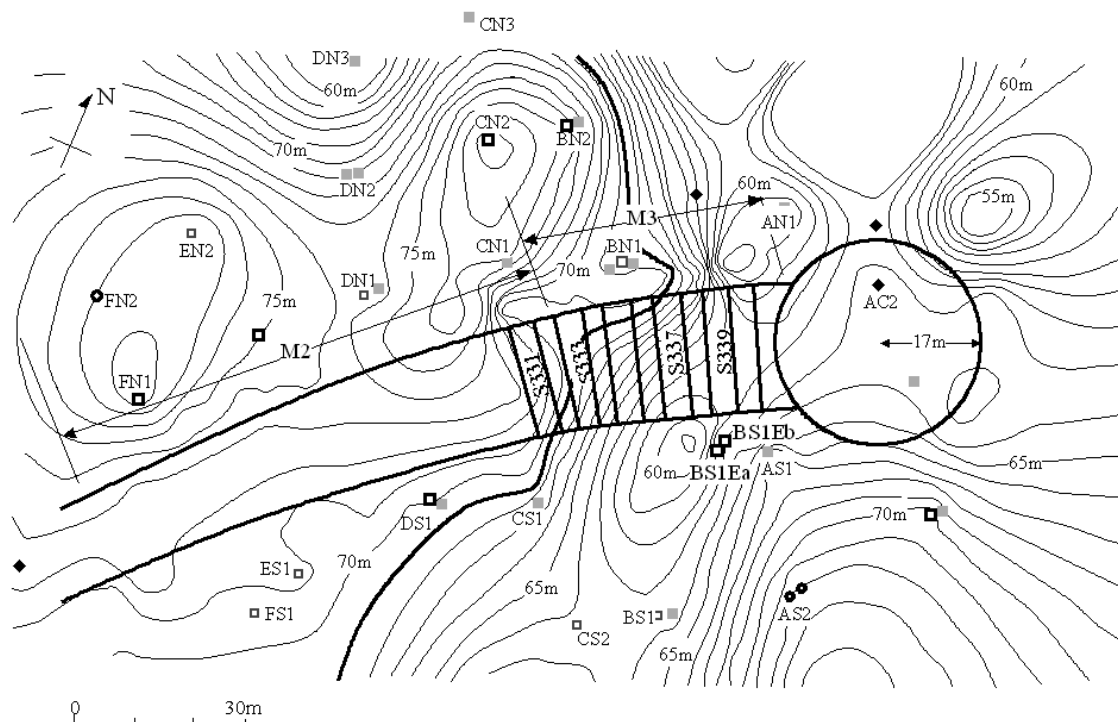


Fig. 10 Counter of elevation at the top of the old alluvium (OA-layer) based on boring data after the collapse.

The magnetic logging was conducted at a spacing of about 2m along the alignment 3m behind the northern diaphragm wall. These are denoted by WN1 southward to WN34 as shown in Fig. 12. Similarly, along the alignment 3 to 5m behind the south diaphragm wall, the magnetic logging was performed. These are indicated by WS1 to WS33 in Fig. 12. When the boring hits some obstructions, the drilling was made once again nearby. In the area between WS10 and WS14, it was not possible for the boring to reach the targeted depths because of obstructions encountered midway. Thus, several more borings were carried out until they reached the OA base deposit.

Damage features under the ground

After investigating the location of the exposed top of the diaphragm wall panels, it became possible to approximately figure out the scenario of the failure. Exact locations of the panel top in plan view are shown in Fig. 12. Based on the features shown in this figure it may be mentioned that the collapse mechanism at panels M213, M212 and M306 were different from that elsewhere and almost certainly it involved the toe of the diaphragm wall panels kicking in, and flow of the soil underneath the diaphragm wall panels. This form of the failure will have allowed the maximum southward movement of the opposite north wall panels, namely M211 and M301.

As a consequence of inward collapse of the retaining diaphragm walls, it was speculated that a

huge amount of soil mass moved into the excavated space. In fact, a subsidence of the ground surface as much as 10m was observed on the south side in proximity to the wall and the settlement extended about 50m southward, as seen in Figs. 4 and 12. The area of the ground settlement extended also north towards the Nicoll Highway but to a lesser extent as can be seen in Fig. 4. Thus, it was considered certain that the failure was initially triggered by the breakage of the diaphragm walls on the southern side, thereby involving a landslide towards the excavated space. As a result of losing the horizontal support, the northern walls moved also towards the excavation, inducing sliding of soil mass towards the south but to a lesser extent.

This scenario of the event was substantiated by the borings conducted after the failure in which the elevation of the intermediate layer F_2 was found to be located almost 10m lower than that found by the borings before the failure. Note that prior to the collapse, the elevation of the base of intermediate F_2 layer was remarkably consistent across the collapse area, and so provides a good marker for identifying the vertical displacement of the soil deposit. The location of the F_2 layers after the collapse is shown in the cross-section in Fig. 11. It was then possible to depict the mode of soil movement involved in the slides on the south and north sides. The approximate locations of the sliding surfaces are displayed in Fig. 13 for the cross section 6-6 of Fig. 12.

As a result of the post-incident in-situ investigations as above, it became possible to figure out approximately the states of collapse for each of the cross sections indicated in Fig. 14. Shown in Fig. 15 are probable features of the collapsed water-strut system buried within the ground which were speculated for the cross section 1, 5 and 7. It is to be noticed that in the cross section 1 and 5, the intermediate layer F₂ is inclined towards the excavation as verified by the boring data at ABH31 and 32 and also at AS1 and AN1 conducted after the failure. This is indicative of the settlement of the even deep-seated soil layer involved in the slide.

Existence of the buried valley

One of the key issues associated with generic cause of the collapse is the depth of embedment of the diaphragm wall toe into the stiff base layer OA which had been considered competent enough at the time the design was made. Thus, it is of crucial importance to identify exact configuration of the top of the OA layer, and its mechanical properties as well.

By compiling all the data made available after the collapse, the contour lines of the top of the OA layer were established as demonstrated in Fig. 10 with a higher level of accuracy as compared to that shown in Fig. 8 which was made up with the boring data at the time of the feasibility study and the design. The contour lines in Fig. 10 unveil some important features as follows which had not been identified in the previous map in Fig. 8.

- (1) There exists a steeply depressed buried valley at the location on the south side a few meters south of the strut S336 and S337. The bottom of the buried valley has an elevation of about 59m RL. The buried valley dips steeply towards the east from the south end of struts S331.
- (2) The buried valley extends towards the north-east running on the west side of the tunnel access shaft (TSA).
- (3) As typically observed in the soil profiles at BS1Ea and BS1Eb in Fig. 11, the SPT N-value tends to increase at the elevation of 67m RL upon hitting the base layer OA. The SPT N-values corresponding to the transition zone below the OA layer were collected from other borings and shown together in Fig. 16 by choosing the nominally identified top of the OA layer as zero point. It may be seen that the property of soils is not so competent with a SPT N-value less than about 30 to a depth of about 3m from the top of the OA layer.

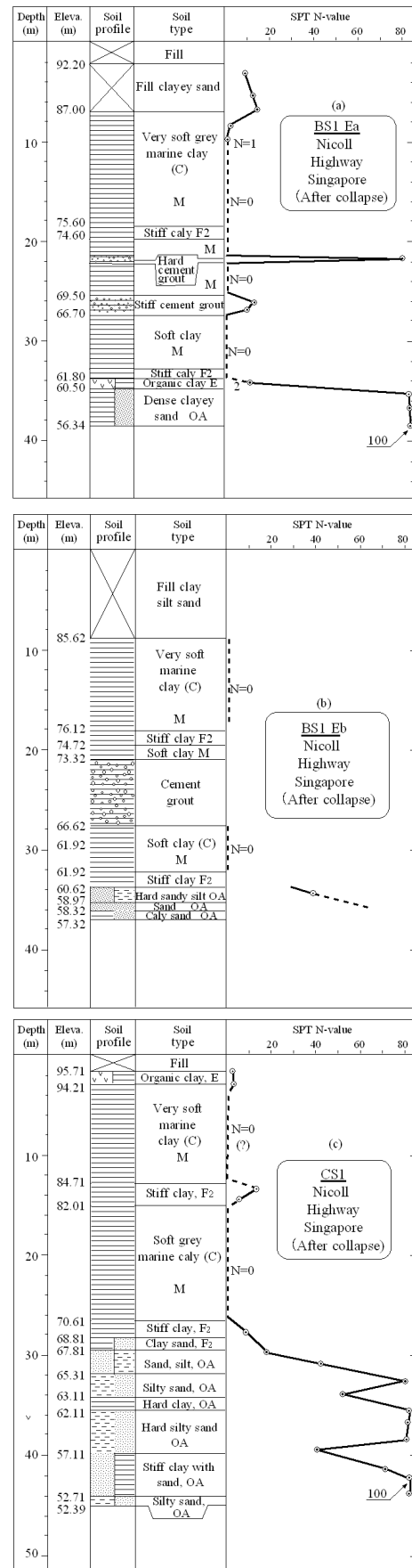


Fig. 11 Soil profiles just south of the collapsed diaphragm wall panels.

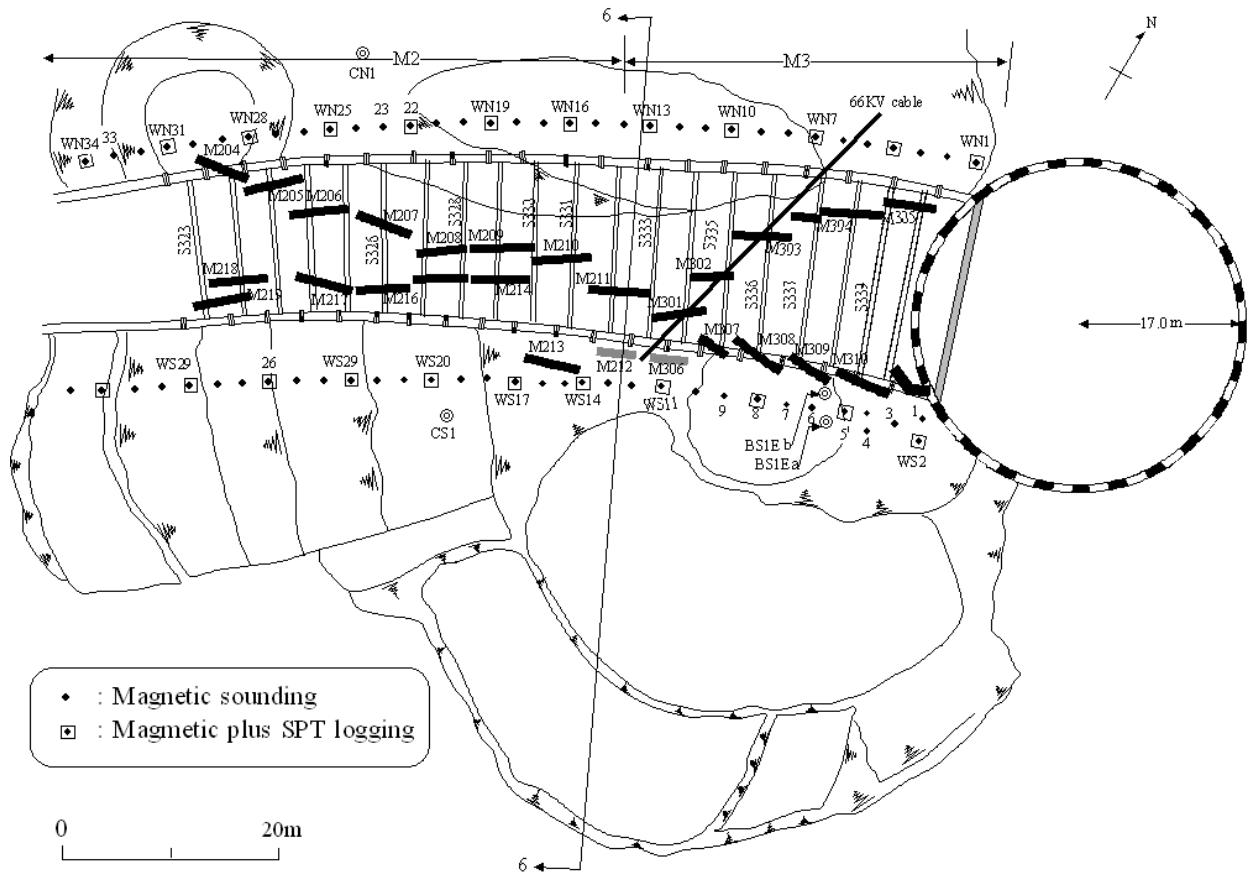


Fig.12 Locations of the magnetic sounding and boring conducted after the collapse.

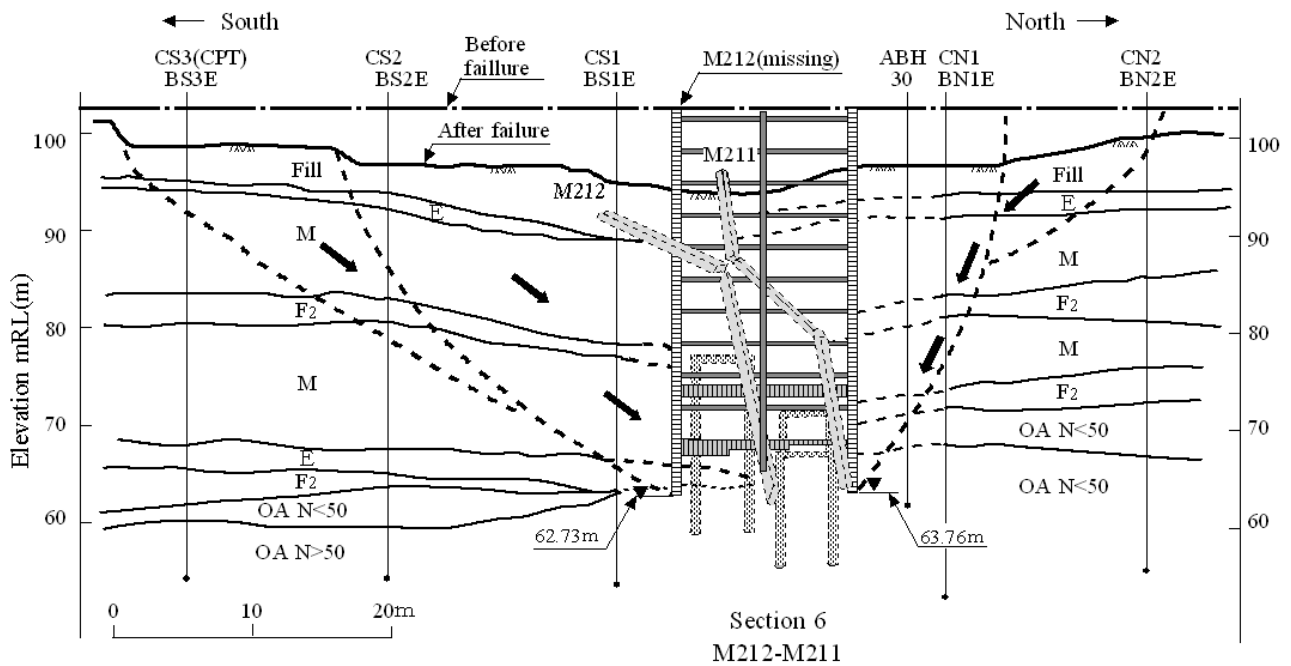


Fig.13. Features of the ground movements towards the excavated space accompanies by the subsidence.

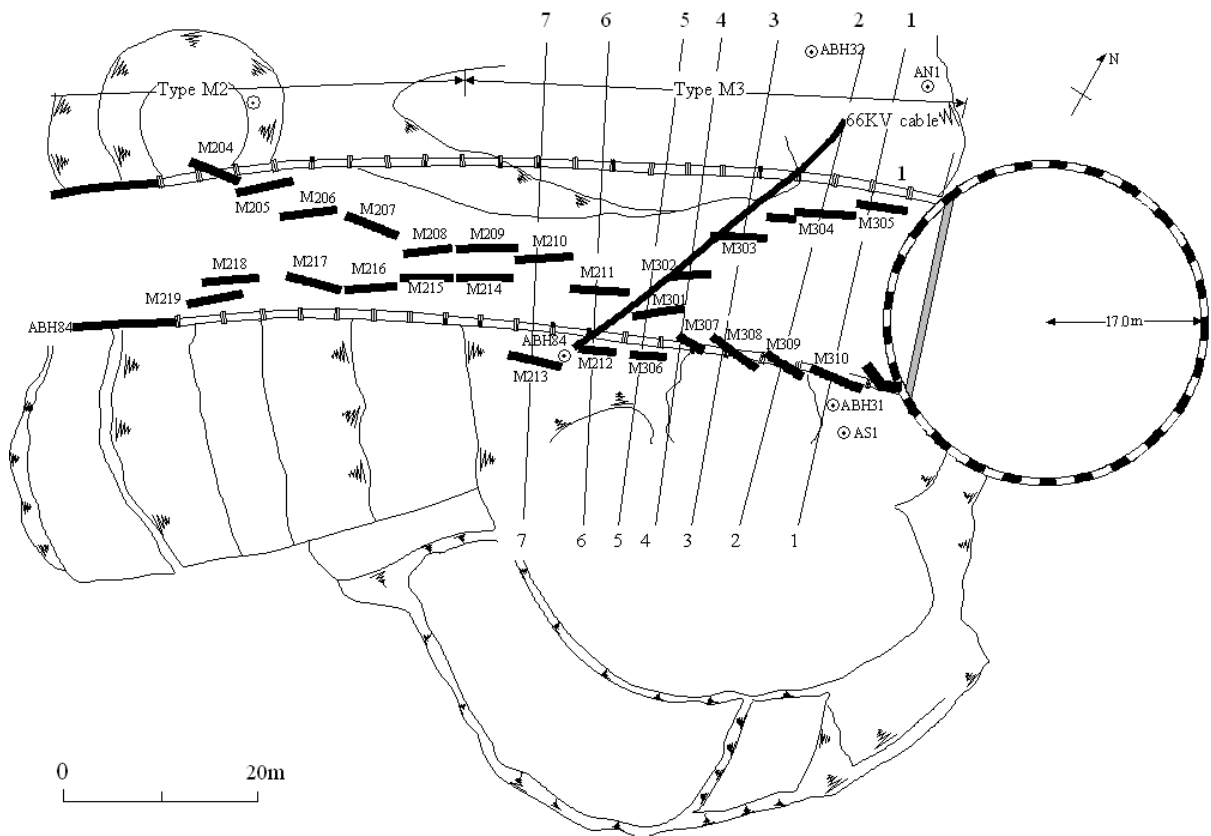
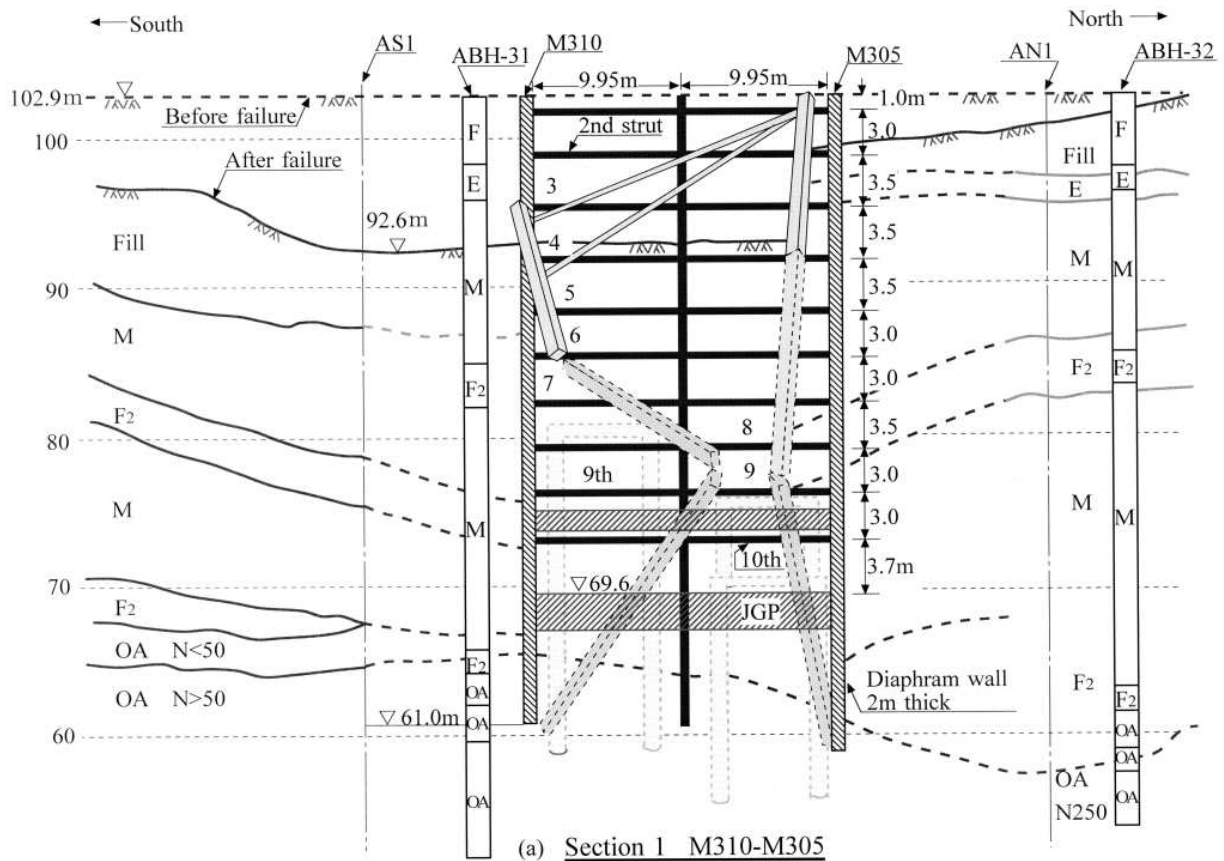
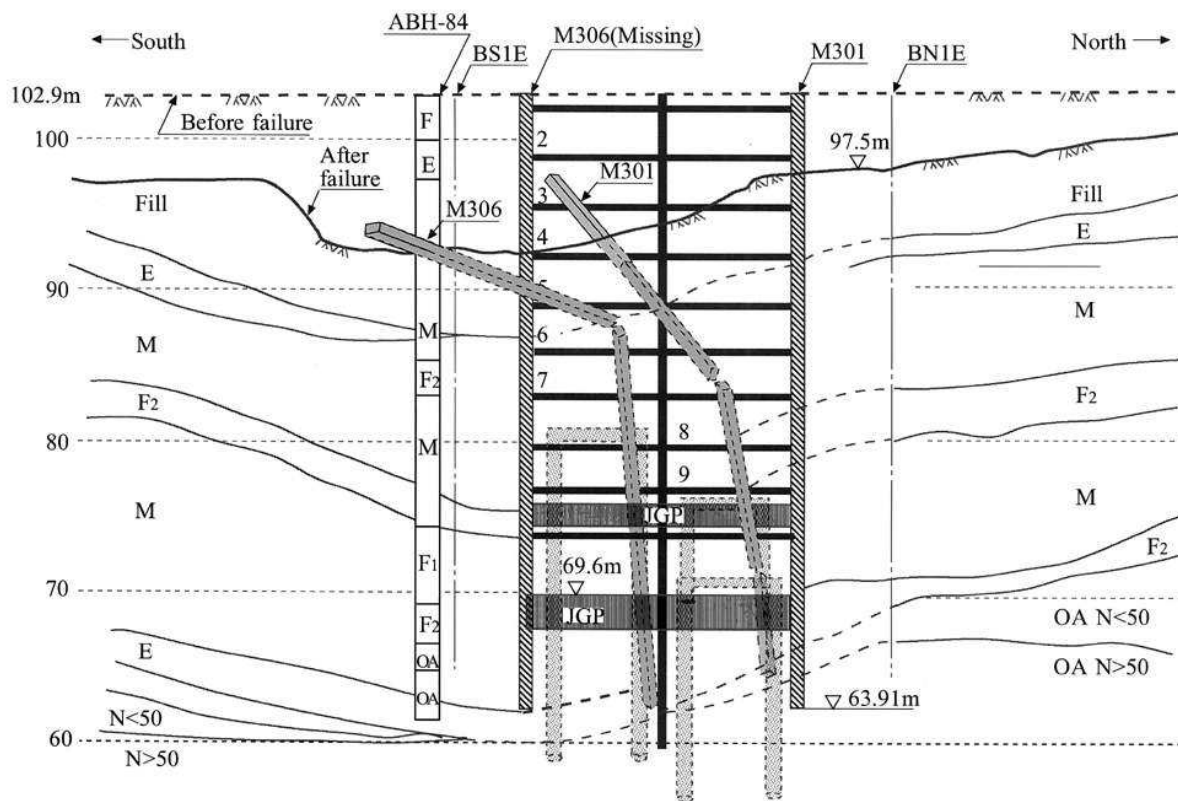


Fig.14 Cross sections for which the damage features were depicted.

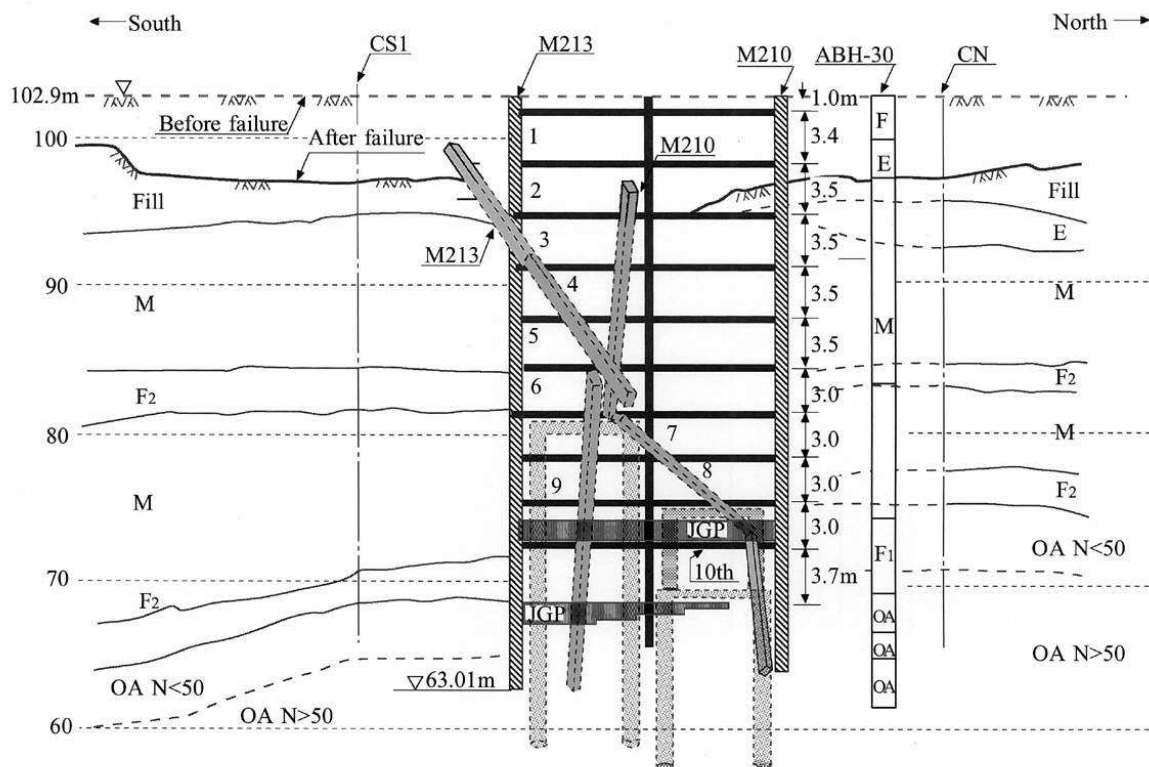


Note: All shapes of diaphragm walls underground are unverified assumption



Note: All shapes of diaphragm walls underground are unverified assumption

(b) Section 5 M306-M301



Note: All shapes of diaphragm walls underground are unverified assumption

(c) Section 7 M213-M210

Fig.15 Cross sections showing the mode of collapse in the zone of failure.

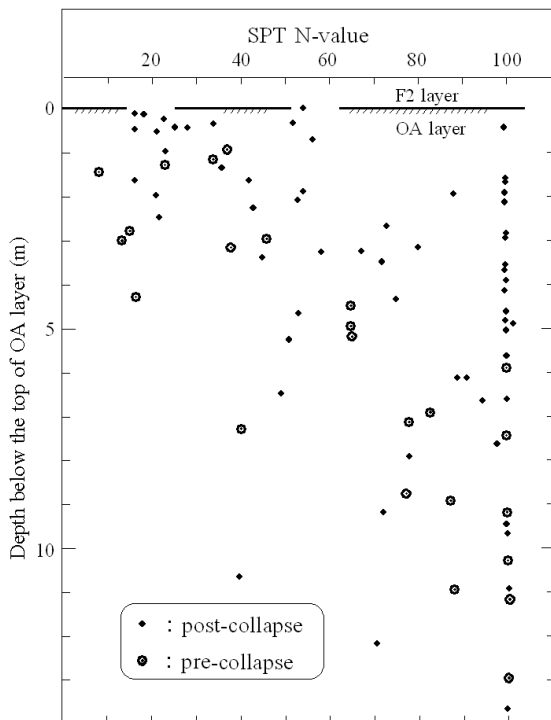


Fig.16. SPT N-values from the top of the OA layer

Features of the toe-in of the diaphragm wall

As results of the detection of the buried valley, questions cropped up as to whether the toe of the diaphragm wall had been embedded sufficiently

deep into the base layer of the Old Alluvium (OA), and also as to whether the stiffness of the infill materials near the top of the OA layer was strong enough to mobilize the resistance.

With regard to the latter suspect, closely-spaced soil investigations revealed that the upper part of the OA layer in the buried valley is comprised of sands, silts and clays sometimes containing organic materials with the SPT N-value not greater than 30 as demonstrated in Fig. 16. This fact indicates that the infill materials were not necessarily as competent as they had been expected at the design stage.

To obtain a clear picture of the toe-in depth, the results of the borings at the time of the magnetic logging tests plus some others nearby were compiled and arranged side-by-side so that soil profiles along the alignment south of the diaphragm walls can be visualized. The picture arranged in this way is demonstrated in Fig. 17 where the top of the OA layer considered at the design stage is indicated, together with the similar top line established after the collapse based on the detailed investigation by the magnetic logging. Note that the symbol such as M306 to M310 and M212 indicate the location of the diaphragm wall panel. It is also to be noted that for the section between M212 and M309, the estuarine deposit E and the upper part of the OA layer might have been displaced laterally being involved in the slide.

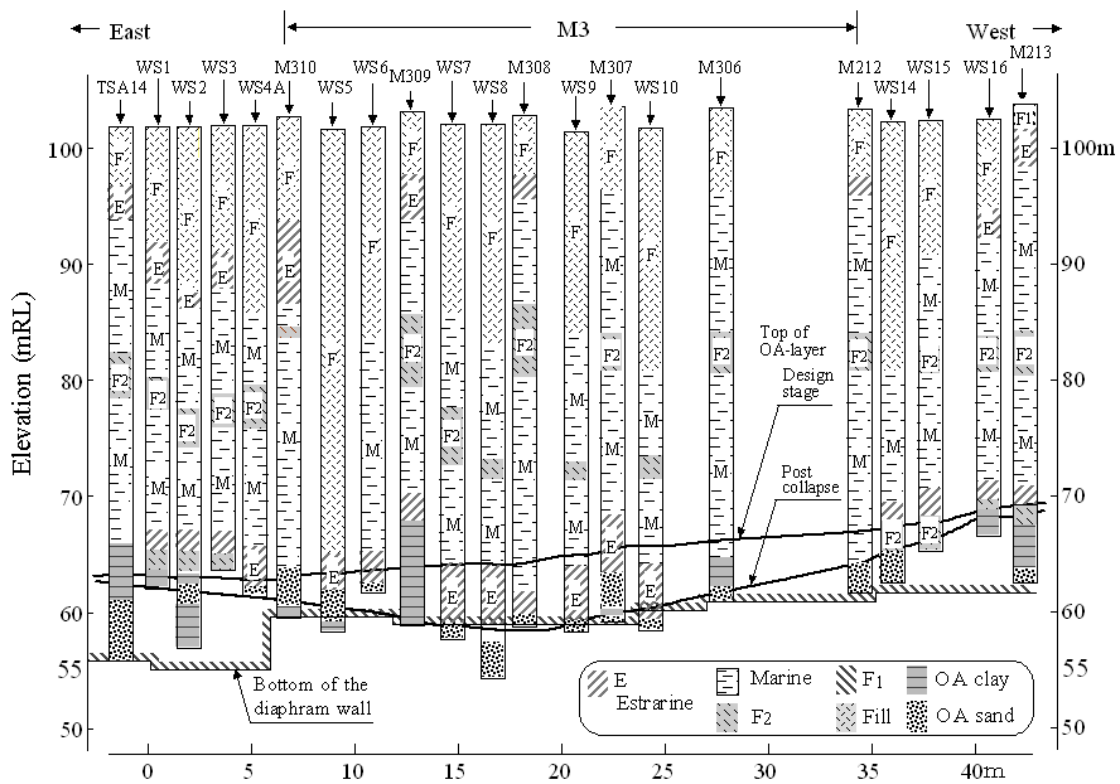


Fig. 17. Soil profile along the south wall and bottom of the diaphragm wall.

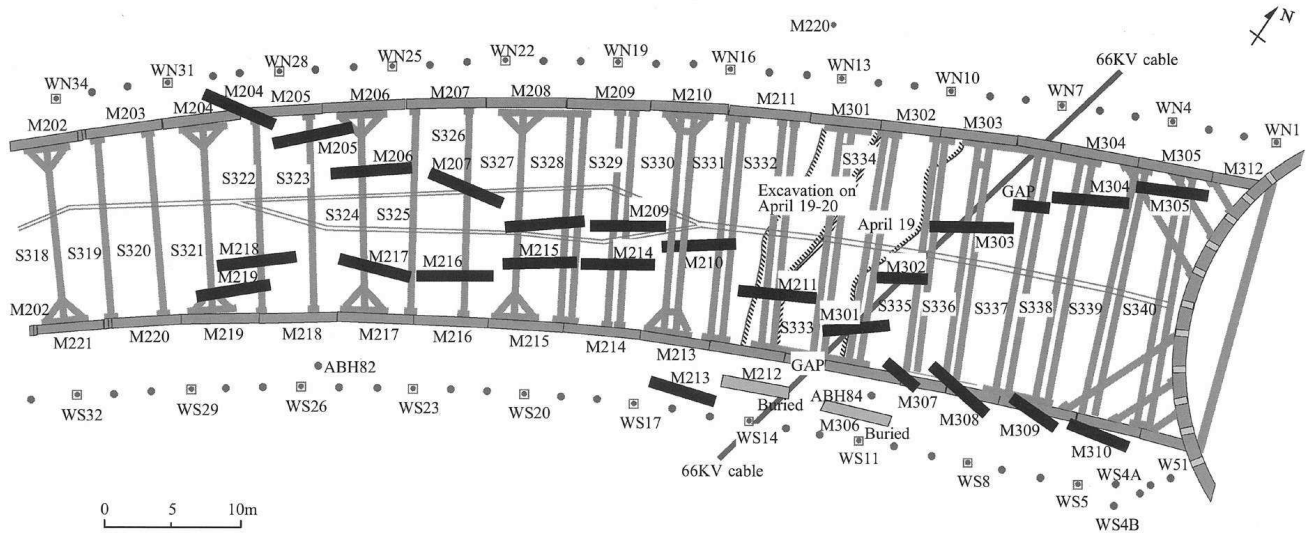


Fig. 18 Detailed arrangements of the struts, diaphragm wall panels and locations of the magnetic logging.

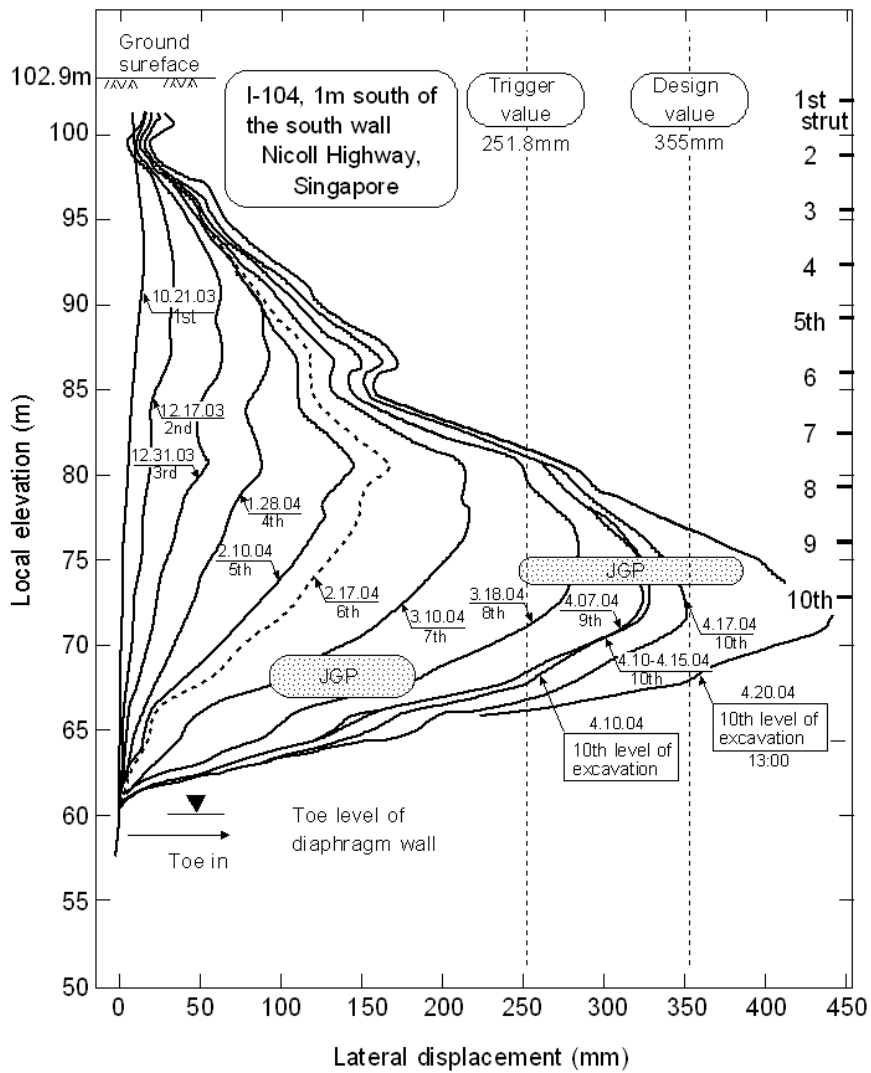


Fig. 19 Time change of depthwise distribution of the wall measured by the inclinometer in the soil deposit 1m apart from the south wall.

At the same time, settlements of the order of 5-10m are envisioned to have occurred in the upper part of the OA layer, as inferred from the settlements of the upper lying layer F_2 , as shown in Figs. 13 and 15, which was confirmed by the post collapse boring data. From the records of the diaphragm wall construction, the bottom line of the wall before the collapse is known and it is shown together in Fig. 17.

By comparing the elevations at the top of the OA layer before the collapse with those of the bottom of the diaphragm wall, it can be seen from Fig. 17 that in the eastern zone between WS1 and WS4A, the toe-in of the wall was more than 3m. In the zone of collapse between WS5 and WS16, the toe of the diaphragm wall seems not to be embedded sufficiently deep. However, the depth of embedment into the OA layer may be considered deeper than 3m in consistence with that considered in the design. It is to be pointed out, however, that the quality of the soils at the upper part of the OA layer was less competent in stiffness or strength as compared to that assumed at the time of the design. This point would have a more important bearing from the forensic point of view.

In the area which failed there were inclinometers at or just behind the wall at the north and south sides of the excavation. Shown in Fig. 19 is the time change of depthwise distributions of the wall deflection estimated from the inclinometer, I-104, which had been installed in the soils 1.0m apart from the south wall near M212 (Fig. 18).

It is known that the wall deflection started to increase significantly from April 17, 2004 as indicated by a dash line in the figure.

The maximum displacement measured at 13:12 on April 20, 2004 is known to have amounted to 43.5cm at the elevation of 72m. It is to be noticed that actual collapse took place at 15:33. i.e. 2 hours 21 minutes (141 minutes) afterwards. Thus, it is likely that just at the time of collapse, the maximum value of the wall deflection might have been larger than 43.5cm. It should be cautioned that the waler-strut system had sustained the deflection of at least 43.5cm for the period of 141 minutes without causing collapse.

To be noticed, moreover, is the fact that the distribution of the wall deflection shown in Fig. 19 was obtained by integration of tilts at several depths measured by the inclinometer on the assumption that the tilt at the toe was equal to zero. This assumption may not be correct in the present case, because the toe of the diaphragm wall had not been firmly embedded in a stiff base stratum. Thus, the distribution of the wall deflection shown in Fig. 19 should be taken simply as indicating the relative displacement counted from the toe of the diaphragm wall.

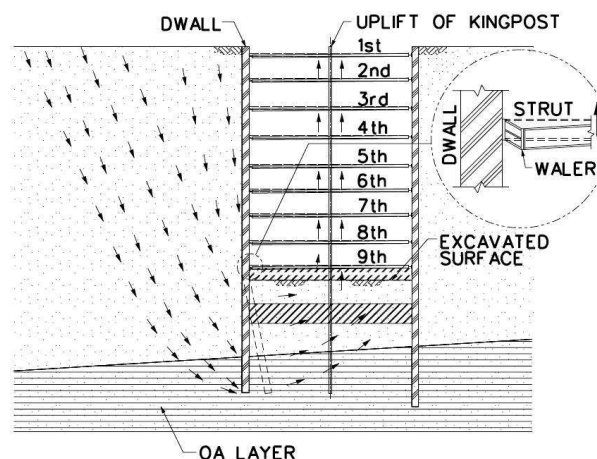


Fig. 19. Probable movements of soils near the toe of the diaphragm wall, and uplifting of the kingpost and sag of the wall leading to buckling and offset between strut and layer.

Likely scenario leading to the collapse

In the light of all pieces of available information as described above, the scenario leading to the collapse may be envisioned as follows.

- (1) When the accident occurred at 3:33p.m. on April 20, 2004, the front of westward excavation was located at the place of strut S332 as shown in Fig. 18. At that time, the 1.5m thick jet-grouted stiff soil (sacrificial strut) was being removed. The underlying soil layers of the OA having a SPT N-value less than about 30 at an elevation 61-63m must have pushed the wall toe towards excavation. The second Jet-grouted stiff layer (JGP) at an elevation of 69.6 to 66.6m was unable to resist against that earth pressure and thus the diaphragm wall panels M212 and M306 were forced to deform inwards. It is considered likely that the second JGP layer might have been possessed of horizontal stiffness lower than that expected before or it might have had brittle characteristics. It is to be noticed here that, because of the presence of the 66KV electrical cable buried at shallow depth through the two panels denoted as GAP in Fig. 18, it was not possible to install the bored piles from the ground surface in advance underneath the elevation of 71m. This is clearly indicated in the plan of Fig. 5. Because of the absence of the bored piles, it was considered easier for the toe of the M212 to M306 to move towards the excavation.
- (2) The inward movement of the wall panel toe must have caused some uplift of the soil deposits within the excavation, which induced in turn a lift-up of the kingpost in the middle of the struts. In addition, excavation has nullified

friction between the soil deposits inside and the diaphragm wall thereby inducing some sag of the heavy-weight wall. The sag must have caused the offset between the strut and walls.

- (3) This uplift, combined with the sag of the wall must have caused the local buckling or breakage of the flange of the H-beams at connection between the walers and struts, creating as much as 20cm vertical offset as illustrated in Fig. 20. As a result, the net length of the struts was shortened and diaphragm wall must have deformed further inwards.
- (4) There were inclinometers installed behind the walls on the north and south sides as mentioned above. The lateral displacement measured on the south increased to at least 43.5cm at the level of 10th strut at 13:12 on April 20. This appears to have been induced by the earth pressure from behind which was mobilized by the inward movement of the wall.
- (5) With the loss of functioning of the waler-strut system and increased inward movement of the wall panel toe, the integrity of the structural support as a whole was completely lost, leading eventually to the total collapse. This is illustrated in Fig. 20.
- (6) As a consequence of the collapse to the structural system, the diaphragm wall panels were twisted, displaced and rotated, accompanied by the slides of the surrounding ground as described in Fig. 13.

The scenario of the collapse as described above is based on the point of view that the generic cause of the failure was the excessive inward movement of the toe of the diaphragm.

Another scenario could be established alternatively on the basis of the assumption that the earth pressure acting on the diaphragm wall had been underestimated at the time of the design. A greater earth pressure than that considered in the design must have induced the large deflection of the diaphragm wall leading to the breakage of the water-strut system. This collapse scenario is addressed thoroughly in the papers by Tang (2006), Yong et al. (2006) and Yong-Lee (2007). The authors, however, prefer to take the views described in this paper.

CONCLUSION

One of the most important lessons learned from the incident in Singapore seems to be the fact that, although the spatial distance of borings or sounding is reasonably short in view of the commonly adopted practice, there always are chances to miss irregularity in the deposits such as buried valleys. There-

fore, special precaution should be paid to the soil conditions particularly in deep-seated deposits.

In the area of complicated geological setting, existence of buried valleys should be suspected at the stage when projects are put forward. In such a case soil investigations need to be carried out at a spacing closer than that normally adopted. In addition, properties of soil materials within the buried valley should be investigated in details, if it is discovered. The outcome of these studies should be meticulously reflected on the grand planning as well as on detailed design of the soil-structure system considering each phase of construction processes.

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