Closure to 'Severe Damage of a Pile Group due to Slope Failure' by D. E. L. Ong, C. F. Leung, Y. K. Chow, and T. G. Ng

Author

Ong, DEL, Leung, CF, Chow, YK, Ng, TG

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D.E.L. Ong[1], C.F. Leung[2], Y.K. Chow[3] and T.G. Ng[4]

The authors thank the discusser V. Diyaljee, for his interest in their research publication. To recap, this paper examines the pre-failure and post-failure behaviour of an instrumented cast-in-place concrete pile group comprising four 900-mm diameter piles subject to soil movements as a result of a slope failure that occurred during the unexpected early arrival of a rain storm prior to the year-end monsoon. Out of the four piles, two piles were instrumented. The front pile was instrumented with strain gauges only, while the rear pile was instrumented with both strain gauges and in-pile inclinometer. In-soil inclinometer that measured the free-field soil movement was installed about 1.5 m away from the pile group (see Fig. 1). Because the soil slip was not anticipated in the pile design, only nominal 0.5 % steel reinforcement was provided in the piles and this translates to an ultimate bending moment capacity of only 520 kNm. In comparison, the ultimate bending moment capacity for a 1075-mm diameter bored pile with 3 % steel reinforcement used by Chandrasekaran et al. (1999) was about 3000 kNm as additional bending moment due to soil movement had been anticipated.

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[1]Director (Acting), Research Centre for Sustainable Technologies, Faculty of Engineering, Science & Computing, Swinburne University of Technology, Sarawak Campus, 93350 Kuching, Sarawak, Malaysia. E-mail: elong@swinburne.edu.my

[2]Professor, Centre for Soft Ground Engineering, Dept. of Civil & Environmental Engineering, National University of Singapore, Singapore 117576. E-mail: ceelcf@nus.edu.sg

[3]Professor, Centre for Soft Ground Engineering, Dept. of Civil & Environmental Engineering, National University of Singapore, Singapore 117576. E-mail: ceechow@nus.edu.sg

[4]Executive Director, GeoEng Consultants (S) Pte Ltd./ Golder Associates (S) Pte Ltd., Singapore 329983. E-mail: ngtg@golder.com.sg

Diyaljee (2015) recommended that the authors provide further details and clarification on some specific discrete events and their associated construction sequence so that the overall construction methodology and timeline are more accurately represented. The recommendations suggested by Diyaljee are reasonable and appropriate in view that forensic engineering often requires reporting of explicit details during more critical events. Therefore, the detailed information as recommended by Diyaljee (2015) will be provided hereinafter.

The key events as shown in Fig. 5 of Ong et al. (2015) will now be further elaborated to reflect the associated discreet events that also took place within the specific construction periods. Immediately, after the occurrence of soil slip on Day 0 (Stage 1), canvas sheet was placed over the affected area to prevent further infiltration of rainwater. During Stage 2 construction, the failed slope was reinstated on Day 4 by placing imported sand onto the failed slope surface to stabilize the affected area. From Day 5 to Day 6, a stretch of about 25 m of sheet pile wall was installed at the toe of the failed slope area and its immediate vicinity in an attempt to negate further slope movements. Further excavation of 1 m was carried out uphill and behind the installed sheet pile wall on Day 7, thus exposing the installed cantilever sheet pile wall. Unfortunately, soil movements continued to occur unabatedly as observed by the increasingly tilted cantilever sheet pile wall as shown in Fig. 2 and also as recorded by the adjacent in-soil inclinometer as shown in Fig. 7 of Ong et al. (2015) until Day 62, despite the installation of waling and strutting system on Day 19 to brace the cantilever wall. Fig. 3 shows the improved representation of the construction timeline to reflect more of the discreet construction activities that had taken place in between the successive major events for better clarity.

Besides the general comments, Diyaljee (2015) also requested for specific information that could provide valuable insights to develop understanding of the behavior of piles in relation to the type of cracking so as to improve the analysis and design of piles subject to soil movements. These specific comments are described henceforth:

1. All the cast-in-place piles within and outside of the boundary of slope excavation were installed before the initial slope failure. However, the localized excavation to expose the pile heads and subsequently casting of the pile caps, was carried out at different times. For example, the strip of pile groups in the western boundary at Gridline 1 with elevation of RL103.20 m (including the instrumented pile group), was the first area to be locally excavated and casted with pile caps. This was followed by the successful capping of the pile groups located along Gridlines 3 and 4 at elevation of RL100.55 m.

   Finally, when the installed pile groups at Gridlines 2 and the installed single piles between Gridlines 2 and 3, all at elevation of RL98.0 m (to support the proposed underground water tank) were to be exposed during the subsequent excavation works, slope failure occurred overnight (Day 0) due to the unforeseen heavy rain storm. When the slope failure occurred, the excavation depth was 3.5 m as measured from the existing ground level and was just 1.7 m shy of the proposed final excavation level.

2. The locations of the piles supporting the underground water storage tank and sheet piles in relation to the boundary of the slope excavation are now re-emphasized in Fig. 1 for greater clarity.

3. From Fig. 2, it was indeed noted that further excavation of 1 m uphill of the reinstated slope and behind the installed sheet pile wall did induce additional soil movements as recorded by both the in-pile and in-soil inclinometers located adjacent to the failed slope in Ong et al. (2015).
4. The sheet piles were installed on Day 5 and Day 6 at the toe of the failed slope area and its immediate vicinity in an attempt to stabilize the affected area and to minimise further slope movements (see Fig. 2). However, on Day 7 (end of Stage 2), subsequent excavation of 1 m that was carried out uphill and behind the installed sheet pile wall inevitably caused further detrimental soil movements perhaps due to limited mobilization of the remolded shear strength of the failed clay slope in trying to sustain the original slope gradient.
5. As addressed in point 1 above, the cast-in-place piles supporting the underground storage tank were installed before the soil slip occurred on Day 0.

The authors agree with Diyaljee (2015) that the observational and numerical backanalysis performed herein are indeed useful to develop insights for the proper understanding and appropriate design of pile reinforcement in (a) cast-in-place retaining walls supporting deep basement excavations (Ong et al., 2006, 2009 & 2011) and Leung et al., 2006) or walls located adjacent to existing structures and (b) landslide stabilizing passive piles (Chow, 1996 & Maugeri et al., 1994). Other geotechnical scenarios where the concept of soil-structure interaction involving the detrimental effect of soil movements can be of relevance include pile-supported bridge abutments (Stewart et al., 1994a & 1994b, Springman, et al., 1991) and possibly riverine infrastructure located in riverbanks subject to large daily tidal fluctuations (Ting & Tan, 1997, Ting, 2004 and Wong et al., 2015), which is being investigated and shall be discussed in future papers.

Besides the fundamental understanding of soil-structure interaction, the authors would also like to acknowledge the importance of accurate assessment of the soil undrained shear strength during pile design. Ong et al. (2006) and Leung et al. (2006) have detailed the importance of differentiating the use of pre-excavation or post-excavation undrained shear strength in assessing the effect of soil movement on piles especially when large strain deformations are expected. As in most practical cases, the pre-excavation undrained shear strength values are available only. Therefore, the maximum normalized limiting soil pressure/undrained shear strength ratio, p_y/c_u of 6 was used in the backanalysis, based on the recommendation of Leung et al. (2006).

The soil limiting pressure, p_y, which is defined as the maximum soil pressure that can act on a pile without causing any further increase in pile bending moment, becomes more important during large strain soil deformations. In this case study, a 'soil flow' phenomenon most likely occurred during the slope failure, whereby the soft marine clay would just flow around the instrumented piles once limiting soil pressure had been reached. Another important design parameter worth mentioning is the soil moderation factor applied to the free-field soil movements recorded by the in-soil inclinometer, which was located adjacent the failed slope. Leung et al. (2006) and Ong et al. (2009) suggested a soil moderation factor of 0.7 for the case of a 4-pile group to reflect the phenomenon of soil reinforcing and shadowing effects due to the presence of multiple individual piles within the group.

Since the numerical backanalysis in Ong et al. (2015) yields reasonably good agreement with the field measured results that consider the appropriate use of the cracked pile properties, the practical application of the concept of limiting soil pressure based on p_y/c_u ratio of 6 and the appropriate use of soil moderation factor have now been successfully validated by this case study.

In conclusion, the reasonably close agreement of backanalysis results can be attributed to the following important contributions:

1. The quality workmanship in preparing and connecting all the geotechnical instruments to the data logging unit that involved a lot of painstaking, but worthwhile efforts. Calibrating, commissioning and maintaining the instrumentation system also formed part of the challenges in this study.
2. Discipline in maintaining a daily site monitoring log book system and photographic evidences to keep track on the construction sequence and methodology, which formed an indispensable formal record for the forensic engineering endeavour. The discrete construction events as highlighted by Diyaljee (2015) are as important to create a seamless integration of important technical information.
3. Fundamental understanding of the key parameters relevant to this study that include the concept of soil limiting pressure, soil moderation factor and progressive cracking of pile that affects its response when subject to soil movements.

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Fig 1

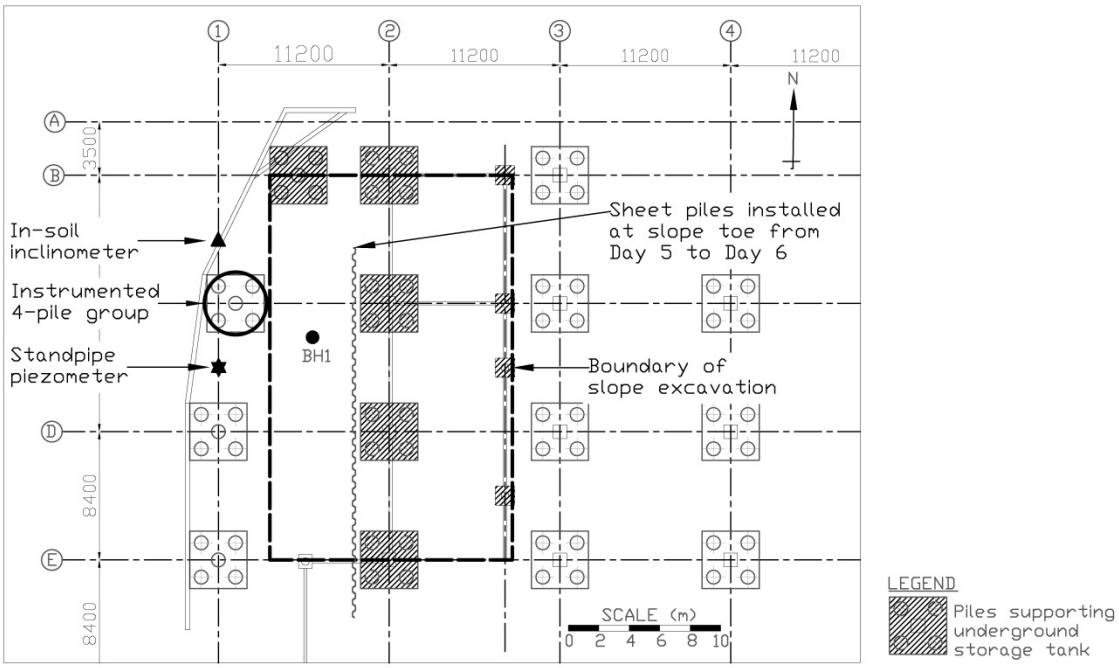
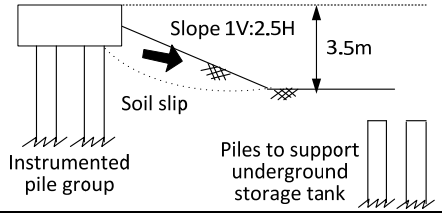
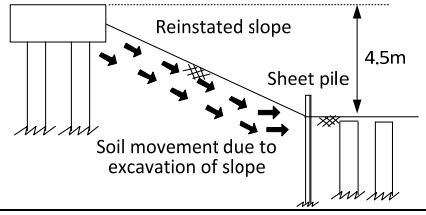
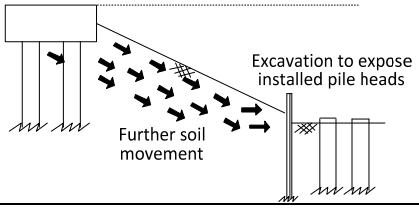
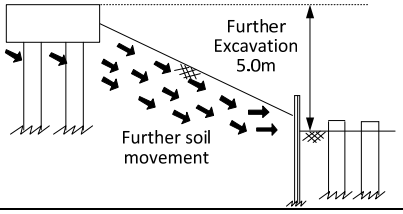
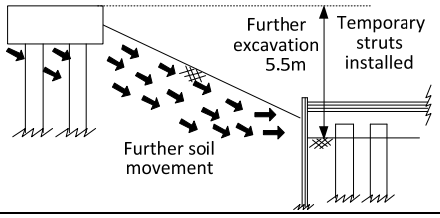
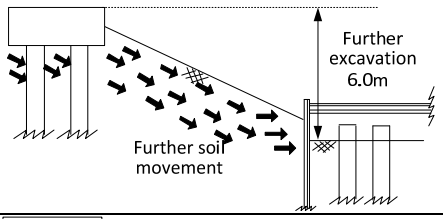
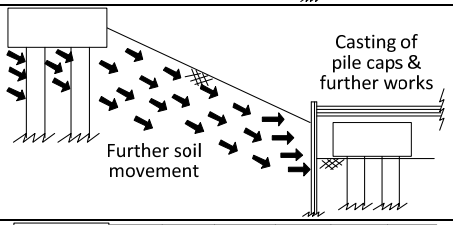
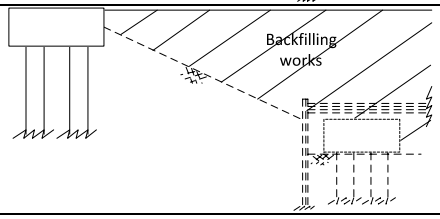

1
2
3
4
11200
11200
11200
11200
N
A
B
D
E
3500
8400
8400
In-soil inclinometer
Instrumented 4-pile group
Standpipe piezometer
BH1
Sheet piles installed at slope toe from Day 5 to Day 6
Boundary of slope excavation
SCALE (m)
0 2 4 6 8 10
LEGEND
Piles supporting underground storage tank

Fig 2

Fig 3

Stage	Day	Key Event	Construction Sequence
1	Day 0	• Soil slip occurred • Excavation depth at 3.5 m	
2	Days 1-7	• Day 4: Reinstatement of failed slope • Days 5-6: Sheet pile installation • Day 7: Further excavation to 4.5m depth • Further soil movement occurred	
3	Days 8-14	• Localized excavation to expose installed pile (heads) supporting underground storage tank	
4	Days 15-18	• Further excavation to 5.0m depth	
5	Days 19-21	• Days 19-20: Temporary waling & strutting system installed • Day 21: Further excavation to 5.5m depth	
6	Days 22-29	• Further excavation to 6.0m depth	
7	Days 30-62	• Casting of pile caps & further works	
8	Day 63	• Backfilling works	

LIST OF FIGURES