APPLICATION AND MONITORING OF A COMPLEX SOIL REINFORCED STRUCTURE NEXT TO FAULT ZONE

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ABSTRACT

Wrap-around reinforced structure gradually became proposed hilly road repair method due to its flexible structure and rapid construction. Moreover, in-situ soil was successfully used as backfill material in Taiwan due to the lack of natural aggregate, which further reduced construction cost. Under well-controlled compaction quality of in-situ soil and thoughtful arrangement of drainage system circumstances, here complex reinforced structure combined with drilling piles and soil anchors was built. Through one year of inclinometer monitor and water level wells observation, increment of displacement of overall slope was decreased with ground depth and almost none displacement is detected under 10m depth.

Keywords: Fault zone, collapse, landslide, complex reinforced structure

INTRODUCTION

In geotechnical engineering, pile can be applied as the foundation to support upper structure and to resist float caused by groundwater; Moreover, when the depth of penetration is over the potential sliding surface at a certain depth, it can reinforce foundation soil below slope toe. Soil anchor system acts as the tie-back element and float resisting system in the engineering of deep excavation. In slope and retaining wall engineering, grout anchorage provides pull to tie the structure into soil and then increases overall stability of slopes and walls.

Geosynthetics reinforcement increases apparent cohesion in the interlocking between soil and reinforced geogrid to reach self-mechanical stabilization. This case is a road repair project, located next to the fault zone, combined the three mentioned systems with drainage system.

In this paper, the use of complex structure composed of reinforced retaining wall, piled concrete retaining wall and soil anchor system are discussed. Moreover, displacement monitoring system was executed by tube inclinometer and one-year water level observation well after the structure completed.

A quick review of case study and review of monitoring results are explained in later section.

PROBLEM

On June 8, 2007, a great landslide up to 80 meters wide and 30 m high was caused by a torrential rainstorm. The existing anchored slope was severely damaged with main structure being pulled out and exposed after the landslide, forming a huge heave on the toe of the slope. Image of collapsed slope and tumbledown betel nut trees are shown in Figs. 1 and 2.

Fig. 1 Severely damage of original slope

Fig. 2 Temporary solution for upper slope

The regional geologic map shows that there are several fault zones cuts through the job site area, rock mass is fragile and weak. It was assumed that the layer at landslide area was affected by fault, see Figs. 3 and 4. According to the on site boring hole results: 0m to 5m is colluvial soil, 5m to 35m is crushed stiff shale rocks and more complete stiffer shale rocks distributes at 35m to 60m. Groundwater level is between the colluvium layer and crushed stiff shale rock layer. Moreover, fault gouge is also mixed in the crushed shale layer, as Fig. 5. Table 1 shows the soil properties of each layer.

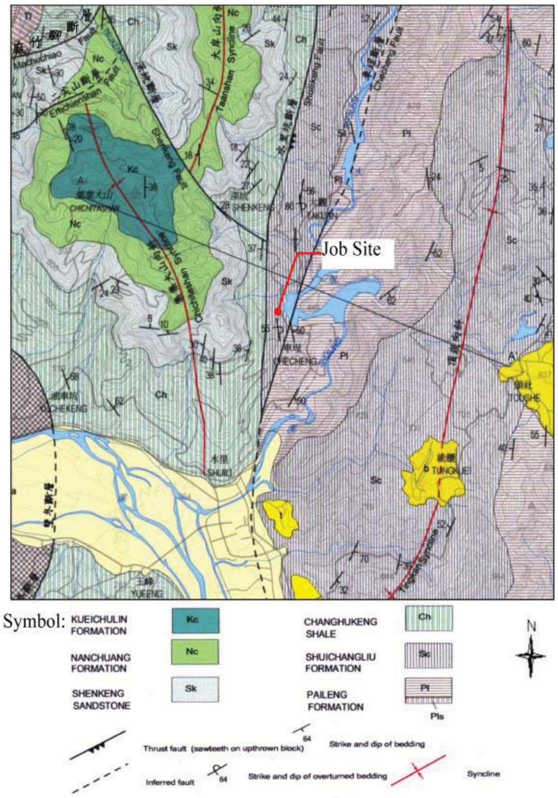

Fig. 3 Regional geological map (Central Geological Survey, MOEA)

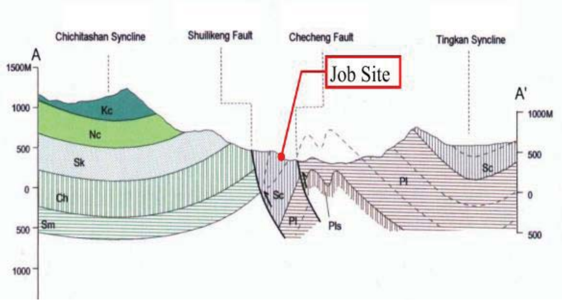

Fig. 4 Regional geological of A-A' section (Central Geological Survey, MOEA)

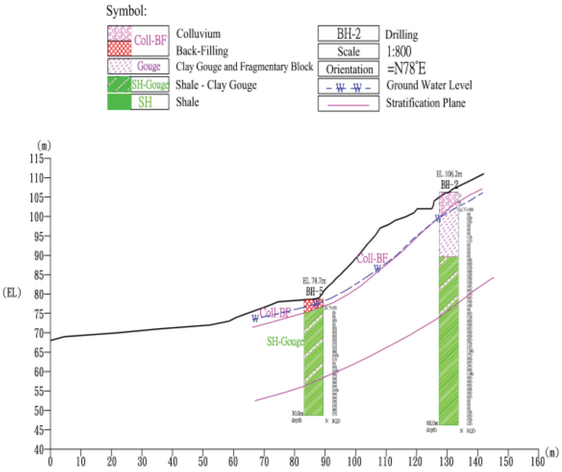

Fig. 5 Layers and groundwater level distribution

Table 1 Soil Properties

Layer	Unit weight	Cohesion	Friction angle
	kN/m3	kPa	degree
Colluvium	20.58	5.0	27.0
Crushed Shale and Gouge	19.30	14.5	23.5
Stiffer Shale	23.62	56.0	34.0

According to the statistic from Central Weather Bureau, the accumulated precipitation of the day before sliding was 123mm, and 88mm on the day sliding failure occurred. Many tension cracks were found on the upper slope in in-site investigation. Rain water penetrated into the cracks and the poor drainage of geological material in this area caused high pore water pressure to the layer thus the great sliding fault happened due to all the factors.

SOLUTION

Owing to particular geographical environment reservoir zone, groundwater level fluctuation brought the stability issue to the job site. Considering this problem, full casing piles, 1.5 meters in diameters and 20 m to 30 m in length, were drilled below 8.5m-high concrete retaining wall at interval of 2.5 meters to be the foundation with complex reinforced method to support the structure. Moreover, three stages of soil anchor were installed from the surface of the concrete retaining wall into the structure. A height of 17 meters wrap-around reinforced structure was built into four stages and was adopted on the concrete retaining wall. According to the analysis, as Figs. 6 to 10 and Table 2., the slope ratio was 1 : 0.3 (V:H) and embedment length of geogrid were 10 meters and 8 meters. Interior and exterior drainage systems were designed for solving poor drainage problem effectively. The in-site soil (crushed shale with silt) was used as backfill material.

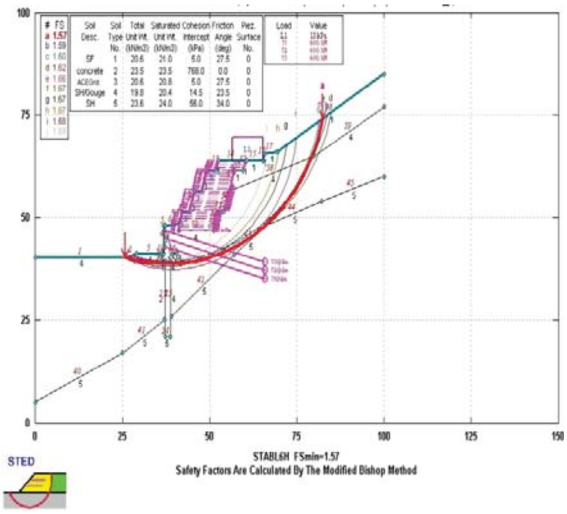

Fig. 6 Stability analysis (Normal mold)

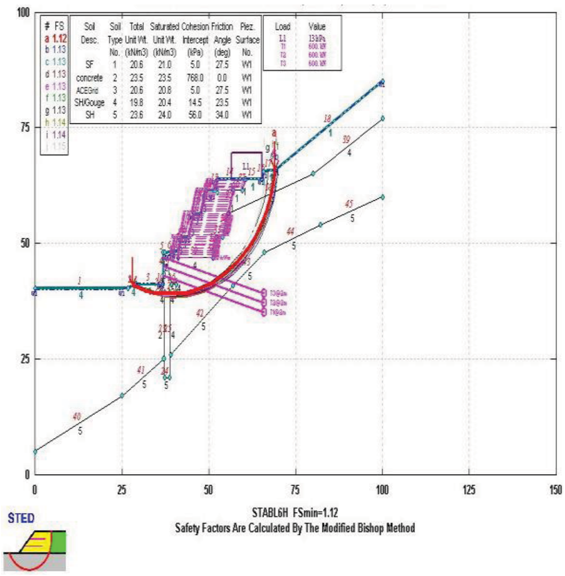

Fig. 7 Stability analysis (Heavy rainstorm mold)

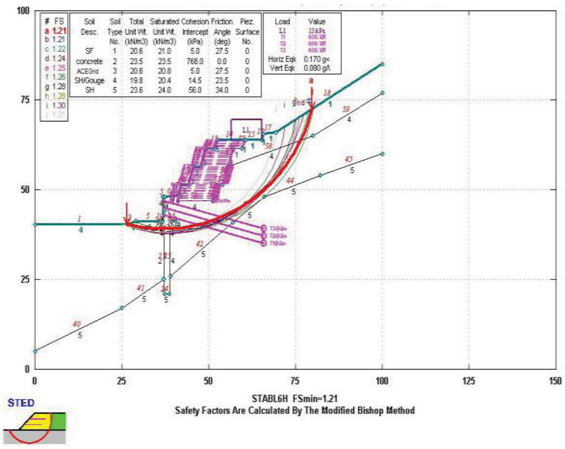

Fig. 8 Stability analysis (Earthquake mold)

Table 2 Factor of safety

Mold	Normal	Heavy Rainstorm	Earth-quake
Factor of Safety	1.57	1.12	1.21

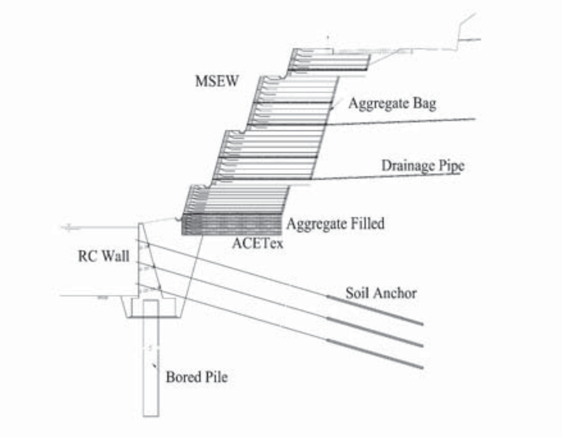

Fig. 9 Cross section of the reinforced structure

Fig. 10 After construction

SLOPE MONITORING AND INTERPRETATION

Surrounded by many fault zones, the geology at site is very fragmental and groundwater concentrates on fault gouge easily. To monitor long term deformation of the structure, 8 inclined tubes and 6 groundwater observation wells were installed on upper slope and reinforced zone of the structure for one-year monitoring of slope displacement and variation of groundwater level, see Fig. 11. Data of maximum displacement, maximum accumulated displacement and displacement rate are shown in Table 3.

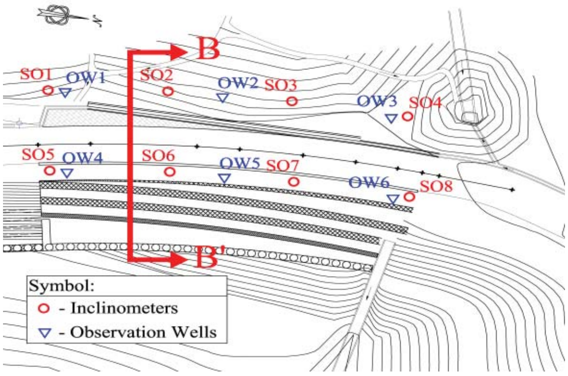

Fig. 11 Monitoring configuration

Table 3 Accumulated displacement and displacement rate of inclined tubes

Tube No.	Place	Accumulated displacement	Max. displacement point	Displacement rate[1]	Displacement rate[2]
		mm		mm/month	mm/month
SO1	Upper Slope	11	0.5m under orifice	0.01	0.002
SO2		14	2m under orifice	0.004	0.003
SO3		13		0.01	0.003
SO4		8	2m under orifice	0.0004	0.002
SO5	Reinforced Area	32	Orifice	0.03	0.007
SO6		22	Orifice	0.02	0.005
SO7		19	Orifice	0.02	0.004
SO8		11	9m under orifice	0.01	0.002

[1] After 5 Months
[2] After 12 Months

Monitoring of Inclined Tube

Four 20 m long inclined tubes were installed behind concrete retaining wall and reinforced zone of the structure respectively (SO1~SO8). The maximum displacement occurred at 2 meters under orifice approximately within the 4 tubes in upper slope. It was assumed that backfill material of the structure was the factor which caused the displacement at first stage. Displacement became smaller and smaller when the depth reaches 18 meters. Maximum accumulated displacement was 14 mm after one year; displacement rate changed from 0.0004~0.01 mm per month in first half of the year to 0.002~0.003 mm per month after one year, see Figs. 12 and 13.

In reinforced zone, the maximum displacements were on the orifice and that of 9 meters under orifice. Both SO-6 and SO-7 had the inflection point at depth of 13 meters where the concrete retaining wall located and constrained the displacement. Obviously, displacement became smaller when it got deeper and deeper. Figure 14, shows the comparison between displacement and layer distribution. Maximum accumulated displacement was 32 mm, displacement rate reduced from 0.01~0.03 mm per month to 0.002~0.007 mm per month after one year.

According to the classification of activity of unstable slope from Japan Association for Slope Disaster Management(1978), the slope in this case was classified stable after one-year monitoring as both the displacement rate of 5 months or one year were all below 0.5 mm. See Table 4.

Table 4 Classification of activity of unstable slope (Japan Association for Slope Disaster Management, 1978)

Activity type	Emergency	Confirmed	Half confirmed	Potential
Displacement (mm/day)	≧20	≧1	≧0.1	≧0.02
Displacement (mm/month)	≧500	≧10	≧2.0	≧0.5
Accumulation in 1-D	Very Obvious	Obvious	A bit of Obvious	Slight
Activity Judgment	Violent	Active	Slow	Need Observed
Sliding Type	Collapse Mud Flow	Colluviums Deep Sliding	Clay Backfill	Clay Talus

Monitoring of Groundwater Observation Well

The groundwater observation data indicated that it was dry season when the construction was just finished, the lowest water level was 10~15 meters, lower than fragmental shale layer. Water level rose 2.5 meters after the attack of Typhoon Morakot, which caused most serious flood to Taiwan since 1949. After one-year observation, the water level of upper slope kept at same level while that of reinforced zone rose 2.5 meters. Figure 15 shows data of three phases, after construction, after attack of Typhoon Morakot and after one year; water level was mostly reached the lowest stage of the reinforced structure.

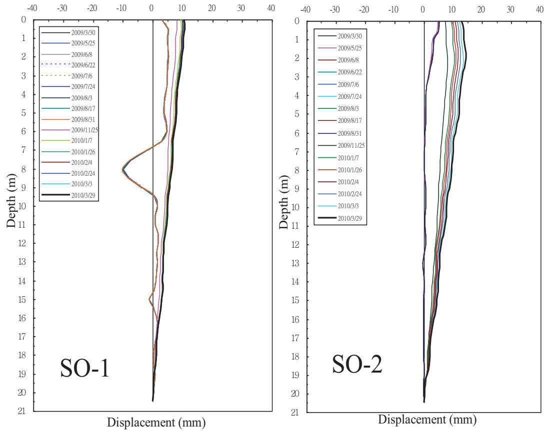

Fig. 12 Monitoring of inclined tube in upper slope

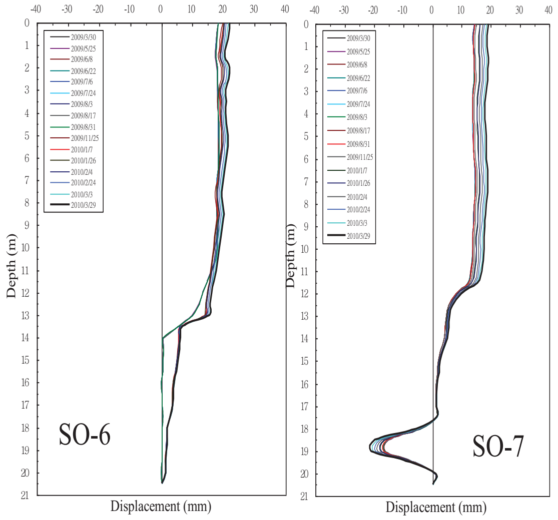

Fig. 13 Monitoring of inclined tube in reinforced zone

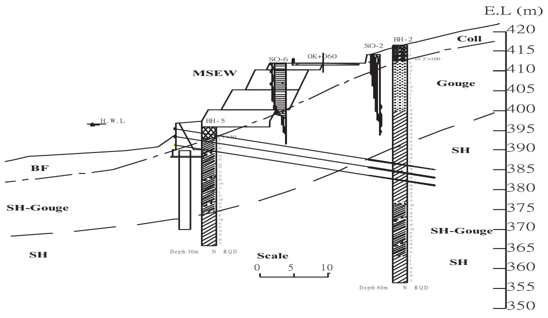

Fig. 14 Comparison between displacement and layer distribution (B-B')

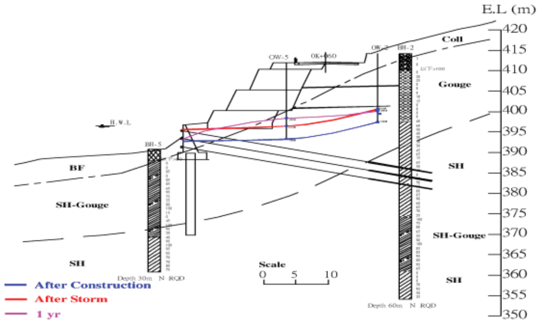

Fig. 15 Comparison between variation of underground water level and layer distribution (B-B')

Displacement Rate and Accumulated Precipitation

According to the study of slope stabilization by Liao (2011), the displacement rate and situation, instead of displacement distance, determine the slope stabilization. Figures 16 and 17 show the comparison of displacement rate with accumulated precipitation and variation of groundwater level.

These figures present that displacement rate was high when the construction was just finished, especially in the reinforced zone. However, the displacement rate in reinforced zone reduced after the particles of backfill rearranged and consolidated. The groundwater level of upper slope rose 2 meters due to the heavy rainfall brought by Typhoon Morakot, and that of reinforced zone rose 4 meters even. But, the water level of both zones was quite similar when the monitoring was close to end. Moreover, the displacement rate didn't increase with the rise of underground water level and got stable instead. Summarily, with proper design and construction of drainage system, complex reinforced method is applicable in high filled structure located in fragmental geology area.

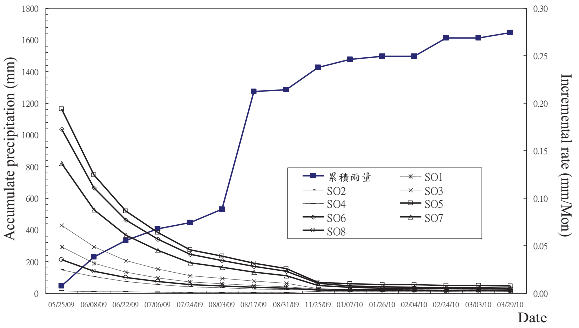

Fig. 16 Comparison between accumulated precipitation and displacement rate

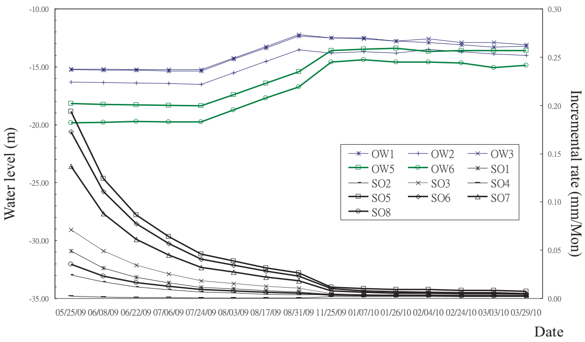

Fig. 17 Comparison between variation of underground water level and displacement rate

CONCLUSIONS

In this case, the fragmental geology at site was caused by faults near it. Bad drainage of colluvium in rainstorm season reduced the stabilization of original slope and brought about the great landslide. This case was designed with interior and exterior draining system. Compaction quality of in-site backfill material was under strict control. Moreover, inclined tubes and groundwater observation wells were installed for one-year monitoring. During monitoring period, Typhoon Morakot attacked Taiwan with heavy rainfall. However, the slope displacement rate did not increase even there was a rise with water level, On the contrary, it has become more stable after a period of time. Summarily, as long as draining system is designed and installed properly and well-monitored, for site of fragmental geology where high filled structure is needed, complex reinforced method is applicable to it.

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