CAISSON FOR SLOPE STABILIZATION

by
W.S. Guo, L.W. Wong and K.I. Hsiung

Reprinted from
Proceedings, 9th Southeast Asian
Geotechnical Conference, pp. 1-45 - 1-56
Bangkok, 1987

9TH SOUTHEAST ASIAN GEOTECHNICAL CONFERENCE
BANGKOK, THAILAND
7-11 December 1987

CAISSON FOR SLOPE STABILIZATION

W.S. GUO
Moh and Associates, Inc.
Taipei, R.O.C.

L.W. WONG
Moh and Associates, Inc.
Taipei, R.O.C.

K.I. HSIUNG
Moh and Associates, Inc.
Taipei, R.O.C.

SUMMARY Due to plate tectonic movement and rainy climate, the rocks in Taiwan are usually fractured and highly weathered. Consequently, landslides have occurred and covered the slope with layers of colluvial materials. Generally, the colluvium is a heterogeneous geological material, often with various sizes of boulders which limits the use of conventional bored or driven piles. Caissons have recently been introduced to stabilize deep seated failure slopes in Taiwan. The earth retaining structure, composed of closely spaced caisson, was constructed by hand excavation and successively cast in place. This paper presents a comprehensive site investigation of a landslide area in northern Taiwan, the mechanism of slope progressive failure, the design of caissons to stabilize the slope, and the construction of the retaining structure. Inclinometers were installed to monitor the behavior of the landslide mass and to evaluate the performance of the retaining structure.

INTRODUCTION

On the foothill areas near the cities in Taiwan, increasing constructional activities have been carried out which have brought forth a multiplicity of geotechnical problems in recent years. The extensive formation of hilly land slopes are composed mainly of Tertiary sedimentary rocks and Quaternary superficial deposits.

The slopes which were originally formed from soft and weak sedimentary rocks were subjected to folding and faulting due to tectonic stresses and earth crust movements and became unstable under humid climate conditions. The slopes are generally covered with colluvial deposits which slid into their present positions from farther on uphill.

A slope failure occurred in 1981 at a residential development site in Hsintien, a suburban hilly area in Northern Taiwan. This unstable area of 4 hectares has caused frequent disruption to road

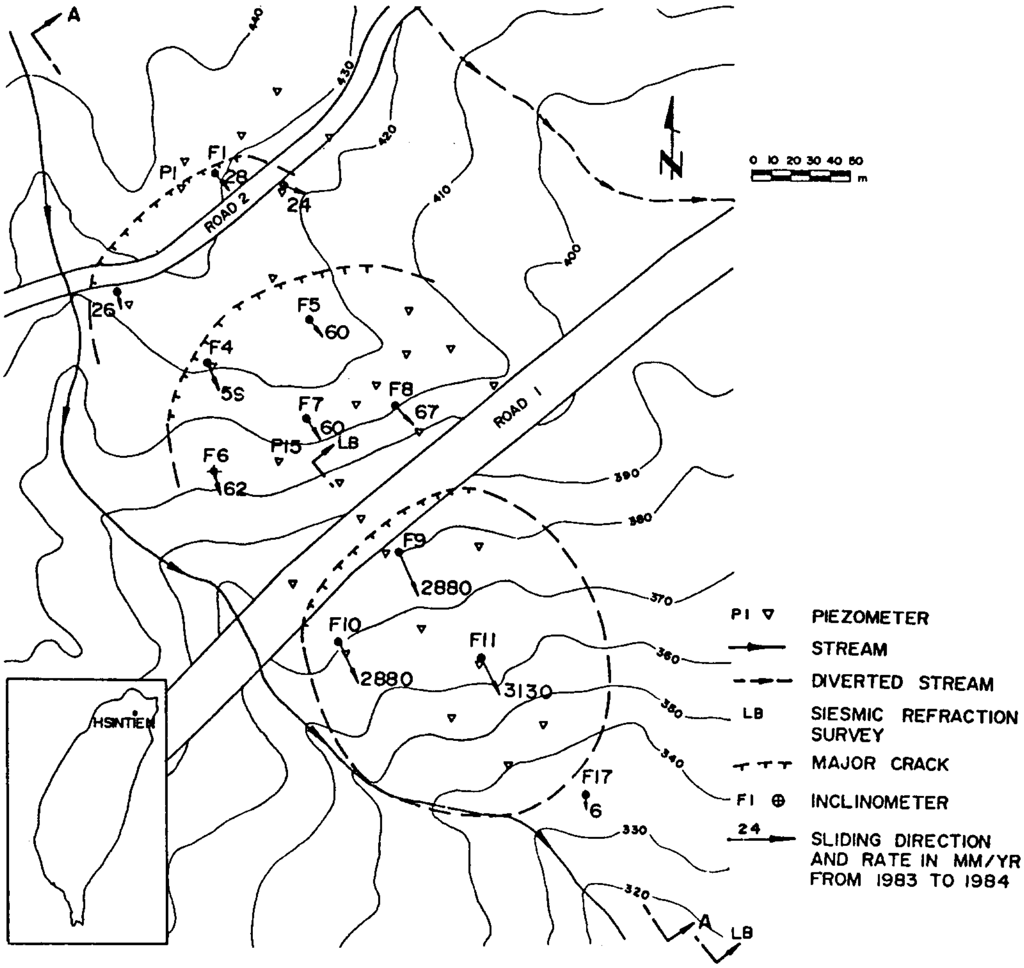


Fig. 1 Site Plan

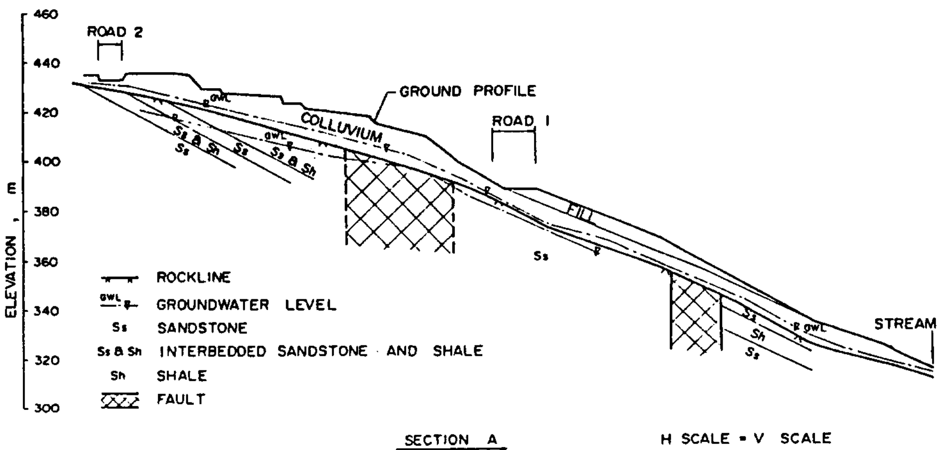


Fig. 2 Geologic Profile

traffic and posed significant threats to future development. Intensive site investigations were conducted and instrumentations were installed to monitor the movement.

SITE CONDITIONS

The slope is located in Hsintien, a suburban hilly area in the southern part of Taipei City. Instabilities had not been noticed until subsidence of two sections of the major roads occurred in May 1981.

As shown in Fig. 1, the topography of the site at the time of the study inclined from northwest to southeast with average gradient of about 25°. The elevation of the slope ranged from 300 to 450 m. Three major cracks were observed at the slope. The dimensions of the unstable area were approximately 300 m in length and 150 m in width. A down cutting stream which was the natural drainage outlet of the site was flowing along the southwestern side of the slope.

SITE INVESTIGATION

Surface and subsurface engineering geological mapping, seismic refraction survey, drilling and instrumentation were conducted on the site. In the surface geological survey, the bedrock outcrop of sandstone and shale indicates the bedding planes strike at N 70° E and dip at 25° to the southeast. Seismic refraction results of two survey lines show that there are four layers at the slope. They are the fill layer, a colluvial layer, highly fractured rock, and bedrock. Several fault sheared zones of low compressive wave velocity are identified. The loggings of 40 drill holes reveal that the slope is composed of colluvium with thickness ranging from 10 to 15 m overlain bedrocks of sandstone, shale, and their interbeddings.

Figure 2 shows the geological section of the slope. The existing upper fault zone in Fig. 2 was inferred from the inconformity with beddings and the extremely fractured cores. It was confirmed at a later stage by direct observation during the excavation of the stabilization works.

HYDROGEOLOGY

About 50 piezometers were installed to monitor pore water pressure which is an important factor causing instability in the vicinity of the slope. The piezometer tips were placed in the colluvium, in bedrock, and at the bedrock surface.

The piezometer monitoring results reveal that different piezometric heads existed in the colluvium and in the rock strata. The piezometric level in the rock strata generally lied 3 m below the bedrock surface. As for the more permeable colluvium, the piezometric level lied at 0.5 to 5 m above the rockline. The

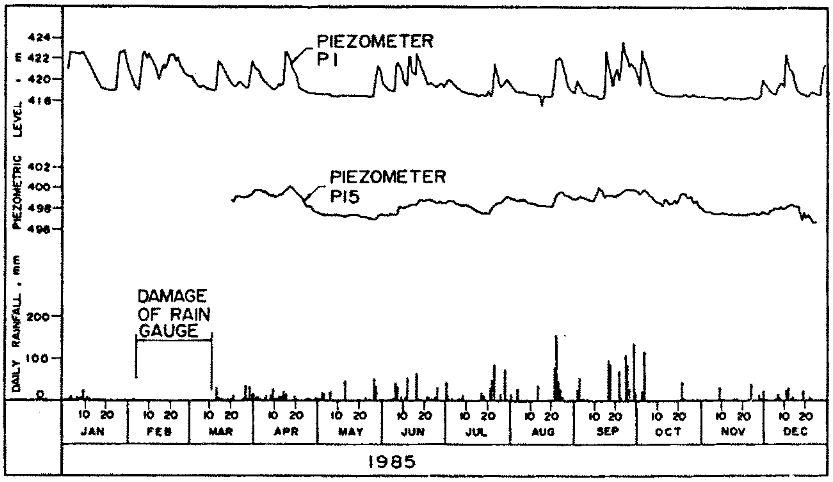


Fig. 3 Typical Result of Piezometer Reading

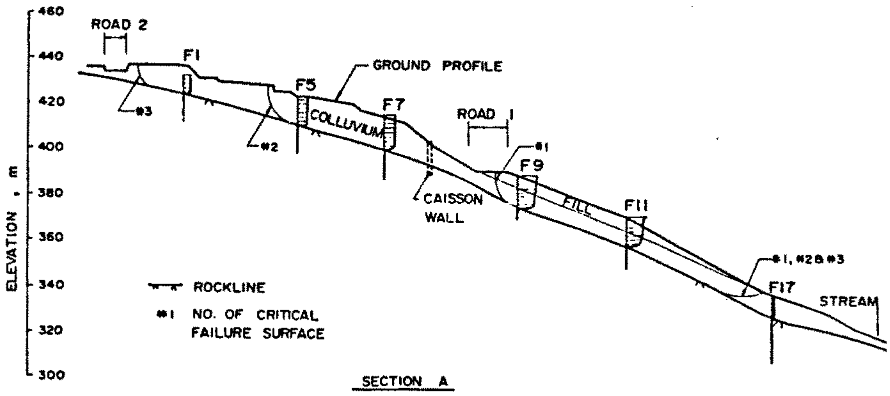


Fig. 4 Failure Pattern of the Slope

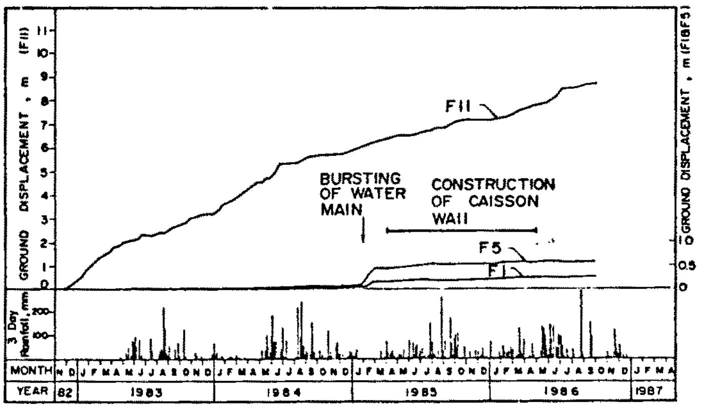


Fig. 5 Time-Displacement Record of the Slope

distribution of the groundwater table is presented in Fig. 2. The presence of the fractured fault zone obviously affects the groundwater distribution. The piezometric head at the upper fault zone is about 3 m higher than that at other rock strata. Figure 3 shows the piezometric response to rainfall of the colluvial slope, even though there was no rainfall data in February 1985 due to damage to the rain gauge. Both piezometers P1 and P15 were installed at the base of the colluvium. The piezometric levels reached to a peak in the first and second days after rainfall and the dissipation took 5 to 7 days. Piezometer P1 had a higher piezometric response, ranging from 2 to 4 m. The response of piezometer P5 ranged from 1 to 2 m. The amount of piezometric response depends on rainfall intensity, local conditions, and the installation quality of the piezometer.

SLOPE MOVEMENT OBSERVATION

Twelve inclinometer casings were installed in order to observe movement of the slope. The depths to the sliding surface were measured before the inclinometer casings were sheared by excessive movement. The position of the failure surface is presented in Fig. 4, indicating the sliding surface coincides with the base of the colluvium. Several piezometers of open tube type were also sheared by excessive ground movement showing the same location of the sliding surface.

The rate and the direction of movement of the inclinometer casings are presented in Fig. 1. Differential rates were measured. The toe portion of the slope was moving at an annual rate of 2,900 mm, the middle portion at 60 mm per year and the crown portion at 30 mm per year. The time-displacement plots of the three portions are shown in Fig. 5. The movement observation demonstrates that the colluvial slope has a retrogressive type failure.

ENGINEERING PROPERTIES OF SOIL AND ROCK

The materials encountered at the slope can be divided into the following three categories: The colluvium, weathered shale, and bedrock.

Colluvium

The colluvium is a heterogeneous material consisting of boulder size rock fragments as large as 2 m embedded in fine grain materials of sand or silt. Although the presence of boulders obstructed soil sampling operation, sufficient fine grained soil samples were obtained by the 76 mm thin-wall tube sampler.

Triaxial consolidated undrained compression tests were conducted on the fine grained soil specimens and the results are summarized in Figure 7. The average effective shear strength envelope can be represented by Eq. 1;

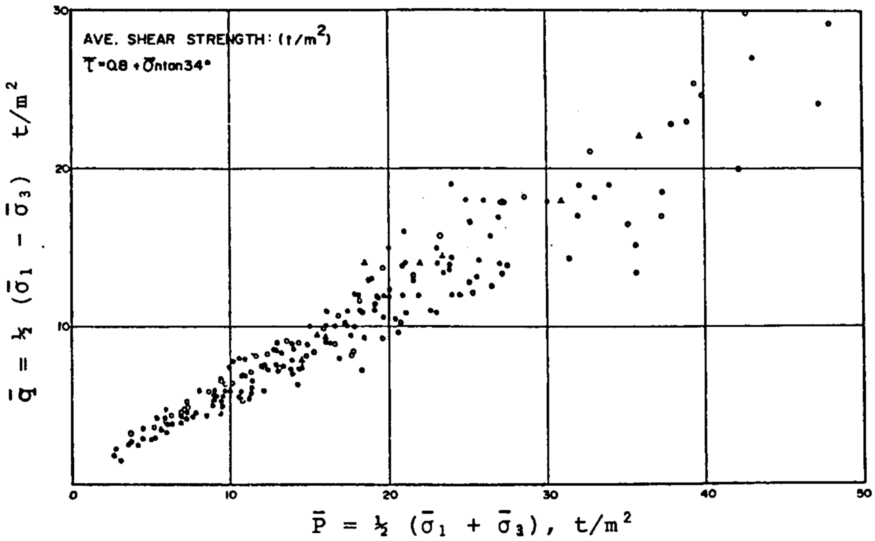


Fig. 6 Consolidated Undrained Triaxial Test Results of Colluvium

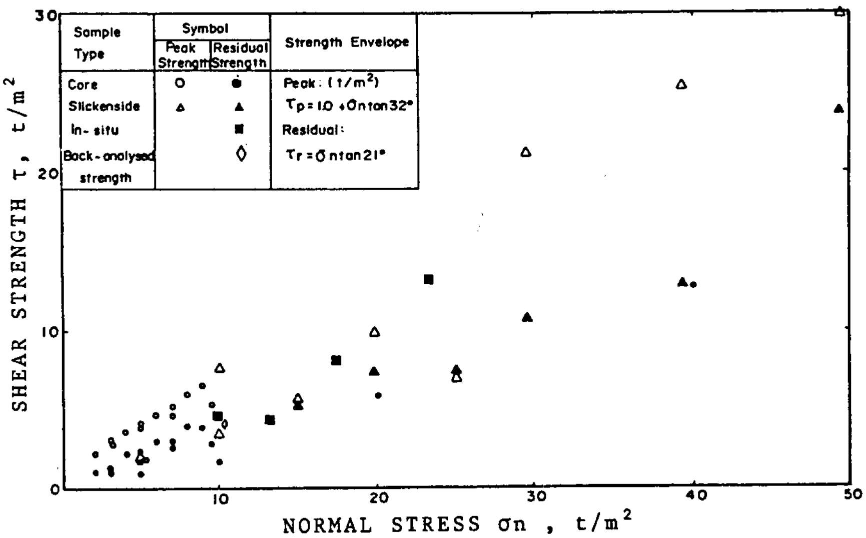


Fig. 7 Direct Shear Test Results of Highly Weathered Shale

Table I Summary of Engineering Properties of Rocks

Rock Type	Unit Weight t/m^3	Water Content %	Uniaxial Compressive Strength MPa	Direct Shear Strength	
				c_r	ϕ_r
Sandstone	2.4 - 2.6	1.0 - 3.0	10 - 70	0	30°
Shale	2.1 - 2.6	2.5 - 1.0	0.1 - 10	0	24°

$$\bar{\tau} = 0.8 + \bar{\sigma}_n \tan 34° \quad (t/m^2) \qquad \text{Eq.1}$$

in which $\bar{\tau}$ is the effective shear strength and $\bar{\sigma}_n$ is the effective normal stress.

Highly Weathered Shale

Clay-like shale materials were encountered at the base of the colluvium. The weathered shale is mainly yellowish brown with some retaining greyish colour. Results of grain size analysis show that the highly weathered shale can be classified as clayey silt material. Three types of direct shear tests were conducted on the highly weathered shale material. They were the laboratory test on core sample, laboratory test on in-situ prepared slickenside sample and the in-situ shear box test.

The slickenside sample was obtained at a site nearby where the shale layer was exposed by a plane failure. The in-situ sample was prepared by pressing the 6 cm diameter shear ring into the greyish softened shale layer. Samples obtained by this procedure were subsequently sheared along the plane parallel to the bedding.

The laboratory direct shear tests were carried out by basically following the ASTM-3080 procedure and the in-situ shear box tests followed the ASTM STP 479. The samples were soaked at least for three days prior to shearing. The direct shear test results are shown in Fig. 7. The residual strengths were generally obtained on the third or fourth reversal cycle when the shear strength substantially remained unchanged. Although the results are scattered, the shear strength of the highly weathered shale may be represented by Eqs. (2) and (3).

$$\tau_p = 1.0 + \sigma_n \tan 32° \quad (t/m^2) \qquad \text{Eq.2}$$

$$\tau_r = \sigma_n \tan 21° \qquad \text{Eq.3}$$

in which τ_p denotes the peak shear strength, τ_r denotes the residual shear strength and σ_n is the normal stress.

The strain softening nature of the weathered shale may induce progressive slope failure as suggested by SKEMPTON (1964) and BJERRUM (1967).

Bedrock

The bedrock underneath the slope consists of Tertiary sandstone, shale, and their interbedding. The shales are susceptible to weathering effects. Slaking tests reveal that the shale specimens would disintegrate quickly when contacted with water.

The uniaxial compressive strength and direct shear strength of the various rocks are summarized in Table I. The direct shear tests were conducted on saw-cut intact core specimens in order to assess the shear strengths along rock joints.

STRENGTH PARAMETER

Janbu's routine method (JANBU, 1972) was adopted for slope stability analysis. Since failure had already occurred, the factor of safety was equal to unity. Once the groundwater level and the position of the slip surface were determined from observation, and the shear strength of the colluvium was obtained, the actual shear strength along the failure surface can be assessed.

Assuming zero cohesion intercept, the back-calculated effective angle of shear resistance for the base of the colluvium is 21°. It can be seen that the actual shear strength along the failure surface is very close to the residual strength of the highly weathered shale obtained by direct shear tests.

STABILIZATION MEASURE

Feasibility studies show that the use of subsurface drainage alone will not be adequate. The use of conventional retaining wall requires a large amount of temporary excavation which would aggravate the situation. Bored or driven piles can not penetrate through the colluvium into bedrock due to the obstruction of boulders. The caisson wall scheme was adopted.

A stability analysis indicates that two rows of retaining structures, sited at the middle and at the toe of the slope, are necessary to stabilize the whole area. However, development of the lower portion of the slope would not be carried out in the near future, the extent of the stabilization was then confined to the uphill area of Road 1 only. A single row of caisson wall located in the middle of the slope will suffice the requirements. Figure 8 shows the layout of the caisson wall.

Design

The designed caisson wall comprised 1.2 m diameter caissons, spaced at 2 m center to center, supported by three to four rows of rock anchors with design loads of 75 or 90 t. The caissons were socketed at least 2.5 m into the bedrock. Figure 9 shows a typical section of the retaining structure.

Bearing in mind that the area downhill of the wall will not be stabilized, the toe resistance relies solely upon the bedrock. The wedge failure mode of the rock mass right in front of the socket is represented, which is similiar to that illustrated by GREENWAY et al (1986). Factor of safety of 2.0 against toe failure has been adopted.

The anchor loads were assessed by equilibrium with the triangular distribution of the sliding pressure. It was expected that the caisson wall would inevitably deflect during construction, no matter whether it is due to slope movement or excavation in front of the wall. The anchors were then specified for stressing to 80% design loads so that the anchor system would not be overstressed.

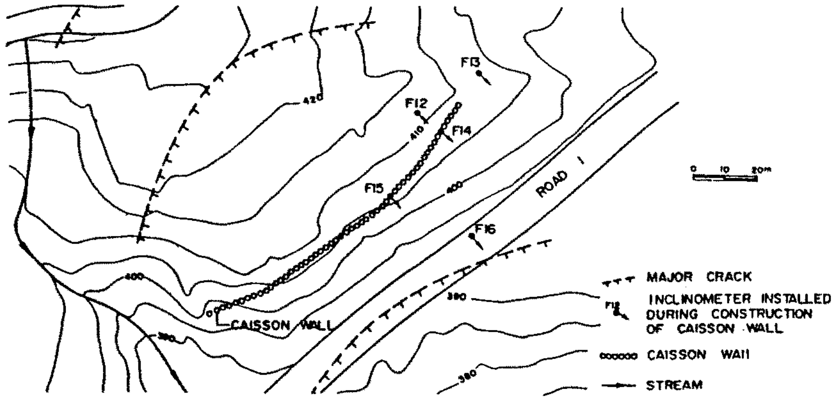


Fig. 8 Stabilization Works for the Slope

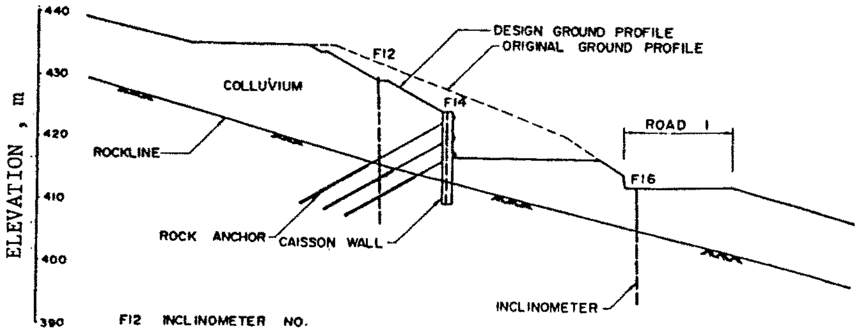
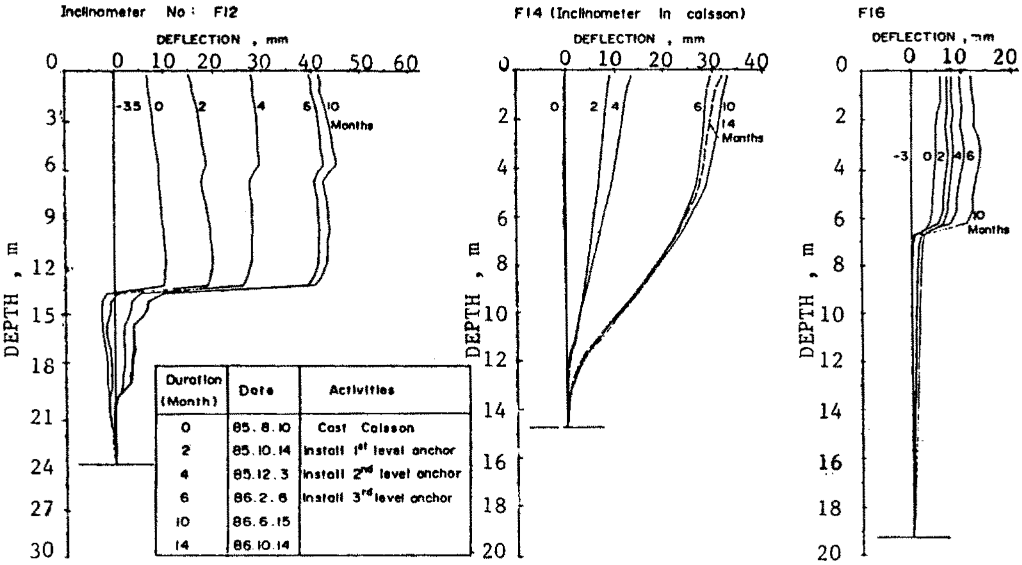


Fig. 9 Typical Section of Stabilization Works



Duration (Month)	Date	Activities
0	85. 8. 10	Cast Caisson
2	85. 10. 14	Install 1st level anchor
4	85. 12. 3	Install 2nd level anchor
6	86. 2. 6	Install 3rd level anchor
10	86. 6. 15	
14	86. 10. 14	

Fig. 10 Movement of Inclinometers Around Caisson Wall

Caissons

The caissons were constructed by hand excavation. This was a new construction method in Taiwan. Generally the caisson was excavated by two crews, one digging in the caisson and the other operating the hoist at ground surface. Pneumatic tools were also used. Unreinforced concrete linings of 0.8 m high were cast along the caisson shaft as temporary supports. When the required level was reached, reinforcement was placed and concrete was poured into the caisson.

Since the groundwater conditions have an important role on the stability, horizontal sub-drains were installed along the uphill side of Road 1 prior to the excavation of caissons. Three additional inclinometers and 3 piezometers were installed and they were measured daily during the caisson excavation period. A criteria was set up that if ground movement exceeded 2 mm in the previous day, the worker should not enter the caisson. Fortunately this situation did not happen, mainly due to the effectiveness of the groundwater drainage measure.

Rock Anchors

In order to assess the bond strength between grout and rock, four pull-out tests were conducted. Two test anchors were installed in fractured shale and the other two were installed in the interbedded sandstone and shale. They were with 5 m fixed length and 102 mm in diameter and the test anchors were drilled at an inclination of 30°to the horizontal. The tested ultimate loads were 34 t and 60 t, and the corresponding ultimate bond strengths of 22 t/m^2 and 38 t/m^2 were adopted for the fractured shale and for the interbedded sandstone and shale, respectively.

Performance of the Caisson Wall

Three additional inclinometers were installed around the retaining structure and another two inclinometers were installed inside two of the caissons. The locations are shown in Figs. 8 and 9, and the observation results are presented in Fig. 10. The results show that the ground behind the caisson wall and the caisson itself moved at a monthly rate of 5 mm prior to the completion of anchor installation. At the post-construction period, the movement rate behind the wall reduced to 0.5 mm/month and virtually there was no movement at the caisson. It can be concluded that the anchor force gradually resisted the sliding pressure and that the slope behind the wall was stabilized.

The ground in front of the wall has continued to move at a rate of 1 mm/month both before and after the completion of the structure. This movement was already expected in the planning stage.

Observation of Colluvium

During the excavation of the caissons, the subsoil strata was logged by an engineering geologist on site. The excavation record

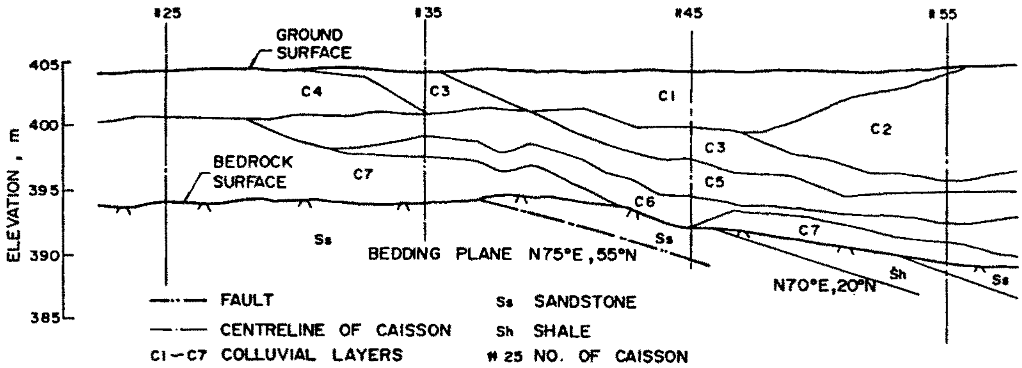


Fig. 11 Profile of Colluvium

Table II Description of the Colluvial Layers

No. of Colluvial Layer	Descriptions
C1	Yellowish brown sandy silt, with occasional sandstone and shale fragments.
C2	Yellowish brown silt, with boulders of sandstone.
C3	Dark grey, highly weathered clay-like shale fragments
C4	Yellowish brown sandy silt, with occasional shale fragments.
C5	Yellowish brown sandstone and shale boulders with occasional silt. Fragments are horizontally bedded and mostly fractured.
C6	Dark grey soft shale fragments, can disintegrate into clay-like material on contact with water.
C7	Brownish boulders of sandstone, with occasional silt and clay in-fill along rock joints.

is shown in Fig. 11. The profile reveals that the colluvium had deposited in layers with an inclination more or less parallel to the bedrock surface or dip slope. Moreover, a greyish soft shale layer (C6) was encountered near the base of the colluvium. This substantiates that the presence of a weak layer along the base of the colluvium as indicated by stability analysis. Description of the colluvial sub-layer is presented in Table II.

CONCLUSIONS

The geotechnical study of the colluvial slope and the experience with the caisson wall suggest the following conclusions:

(a) The failure surface of the colluvial slope lies at the base of the colluvium. Progressive failure mechanism has been observed.
(b) Clay-like highly weathered shale was encountered near the base of the colluvium.
(c) The back-calculated shear strength along the failure surface is very close to the residual shear strength of the highly weathered shale.
(d) In view of the findings of (a) to (c) above, it can further be concluded that the progressive failure of the colluvial slope may be attributed to the presence of the highly weathered shale at the base of the colluvium.
(e) The groundwater level of the colluvial slope fluctuated in respond to rainfall. Piezometric rise of 2 to 4 m has been observed during the monitoring period.
(f) The anchored caisson wall performed satisfactory during and after the construction.

This case study demonstrates that comprehensive site investigation and instrumentation are necessary for analysis and design of geotechnology. However, the success of a geotechnical project could be fortified with intimate site supervision and continue feed back of subsoil information during the construction stage.

ACKNOWLEDGEMENT

The writers wish to express their appreciation to Dr. Z.C. Moh and Dr. T.C. Kao of Moh and Associates, Inc. for review of the manuscript and constructive suggestions. Thanks are also given to Miss Andi Y.F. Song for typing the manuscript.

REFERENCES

BJERRUM, L. (1967), "Progressive Failure in Slopes of Overconsolidated Plastic Clays and Clay Shales", Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 93, No. SM5, September 1967.

GREENWAY, D.R., POWELL, G.E. and BELL, G.S. (1986), "Rock Socketed Caissons for Retention of an Urban Road", Proceedings, Rock Engineering and Excavation in Urban Environment", The Institution of Mining and Metallurgy Conference, Hong Kong.

JANBU, N. (1972), "Slope Stability Computations", Embankment Dam Engineering, Cassadgrande Volume, Ed. R.C. Hirschfield & S.J. Poulos, John Wiley & Sons, New York, pp. 47-86.

SKEMPTON, A.W. (1964), "Long Term Stability of Clay Slopes", Geotechnique, Vol. 14, No. 2, pp. 77-102.