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RESIDUAL STRENGTH OF THE MUDSTONES OF THE CHOLAN FORMATION

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ABSTRACT

A section of an expressway was aligned through mudstone and sandstone formations of the Pliocene epoch. At one location, the expressway was at a distance of about 100 m from a syncline axis. Shortly after construction, signs of instability were first observed at the road embankment and at the adjacent natural rock slope. Results of back-analyses indicated that the actual shear strength which was mobilized along the failure plane in mudstones had an angle of shearing resistance of 9.6 degrees. This agreed with the residual strength obtained by direct shear testing on a slickensided sample. This paper presents the creep behaviour of the landslips and summarizes the shear strengths which were back-analysed from various landslips in mudstones.

Key words: mudstones, residual strength, syncline, landslip

INTRODUCTION

Mudstone is a weakly cemented sedimentary rock composed of fine grain particles. According to Skempton (1964), stiff clays, shales and mudstones exhibit the characteristics of peak and residual strengths. However, slopes formed in mudstones, either cutting into the rock strata or filling on mudstone layers, usually involve stability problems which can mainly be attributed to the low shear strength of the material.

During the construction of a section of the Second Freeway in northern Taiwan, several landslips occurred in the mudstone. Slope stability studies, including additional site investigations, and a program of instrumentation and laboratory testings, were conducted. The residual shear strength and stress/strain relationships of the mudstones obtained from the laboratory testings and by the back-analysing technique are reported in this paper. Results of these studies should provide valuable input for the future design of slopes in mudstone.

REGIONAL GEOLOGY

The 20 km long Kwanshi-Chutung section of the Second Freeway is located at the geologic province of the Western Foothills which is the site of a late Cenozoic sedimentary basin. In the western section, the rock is composed of the Cholan Formation of the late Pliocence age. Based on the results of geological surveys and site investigation of drillholes, the Cholan Formation is composed of interbedded sandstones and mudstones. While the sandstones can be as thick as 3 m, the mudstones are generally in thin layers ranging from 0.5 m to 1.5 m.

There are geological features, such as faults and folds, located alongside the alignment of the freeway. The synclines and the anticlines generally strike in a northeast direction. The rock strata situated on the flanges of the synclines and the anticlines are gently dipping at a maximum inclination of 25 degrees from the horizontal.

SITE CONDITIONS

As shown in Figure 1, chainages 82+400 to 83+000 of the Second Freeway were aligned at about 100 m north of the Kantzuchi Syncline. The beddings strike at N60°E and dip from 0° to about 25°. The ground elevation beneath the embankment ranged from 70 m to 150 m. The ground surface inclined at a maximum slope of 25 degrees from the horizontal. The ground

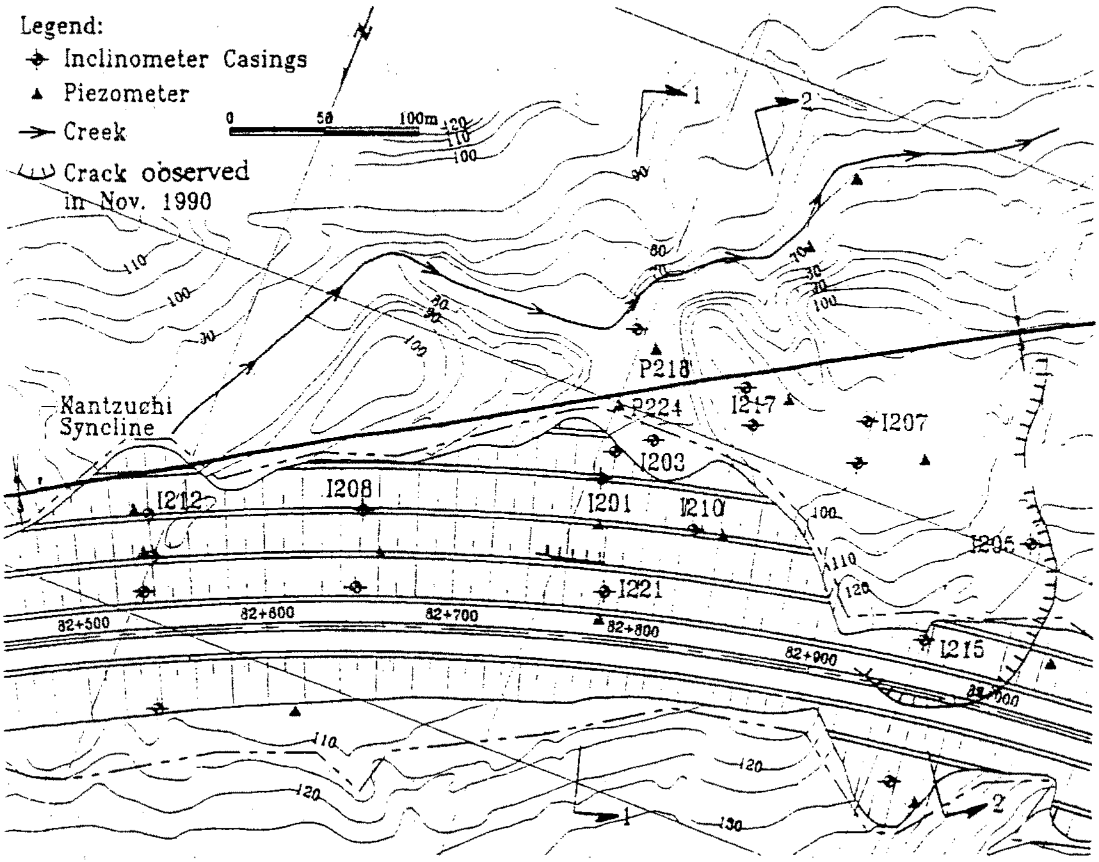

Figure 1. Plan of the original road embankment.

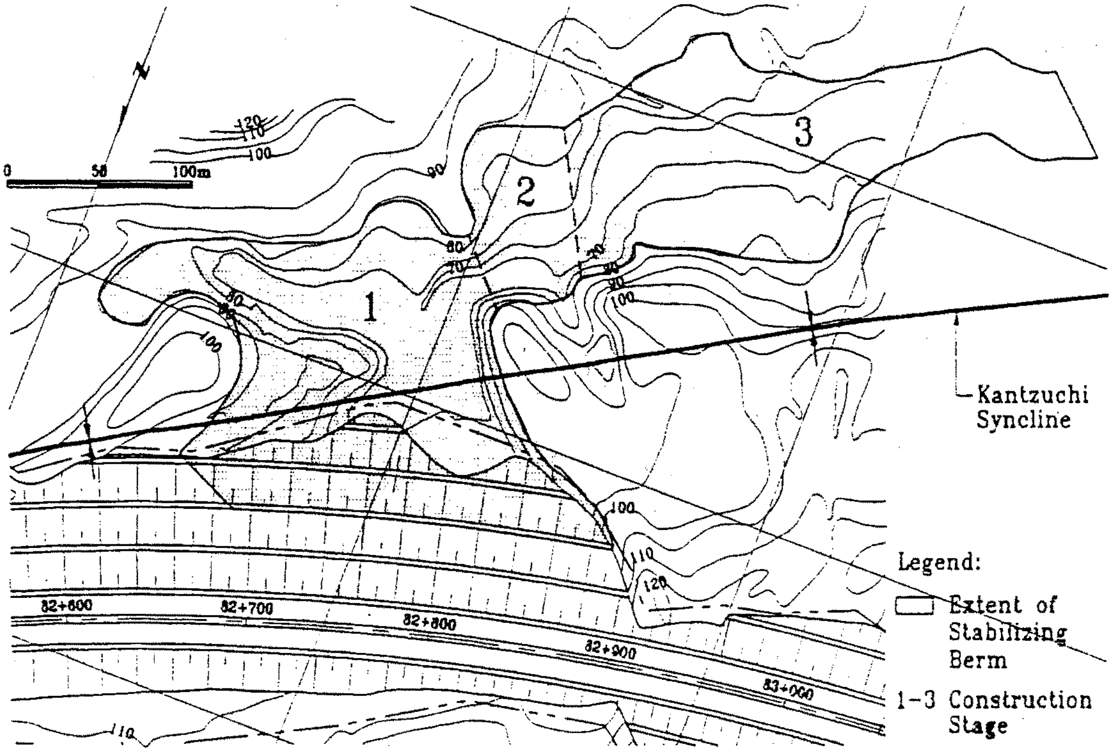

Figure 2. Plan of stabilizing berm.

was terraced for paddy fields and ponds for irrigation were located in the vicinity. The height of the fill embankment varied from 25 m to 40 m. The design gradient of the slope was 1 on 2 (vertical on horizontal). Benches of 3 m in width were provided on the slope at the vertical intervals of 10 m. Around chainage 82+900, the fill embankment rested on a natural rock slope.

Construction of the road embankment began in February 1989. Fill materials of rolled sandstones and mudstones were placed in horizontal layers of about 300 mm in thickness and were compacted to 90% of maximum density. In November 1990, the embankment was filled to the elevation of about 98 m. However, signs of instability were first observed on 18, November 1990 at chainage 82+770. Tension cracks of about 10 mm wide were observed around the crest of the fill slope. Inspection of the nearby area discovered that instabilities had also developed on the ground surface of the rock slope located at chainage 83+000. Figure 1 shows the locations of the tension cracks. Construction of the embankment was subsequently suspended in December 1990, and additional site investigations were conducted. Extensive instrumentation consisting of 37 piezometers, 27 inclinometer casings, 10 crack width gauges and 26 settlement markers was installed.

STABILIZATION MEASURES

After the readings of the crack gauges indicated that the rock slope at chainage 83+000 was moving at a rate as high as 200 mm/month, stabilization measures were instigated. As shown in Figure 2, a stabilising berm of 10 m to 20 m in thickness and 400 m in length.

underlain by a 500 mm-thick filter blanket, was provided in 3 stages along the toe of the unstable slopes. Stabilising berm Stage 1 supported the fill embankment from chainages 82+700 to 82+900. Berm Stages 2 and 3 supported the rock slope from chainages 82+900 to 83+100. Figures 3 and 4 show the sections of the unstable slopes and the extent of the stabilising berm. The progress of the 3 stages of the stabilising berm is shown in Figure 5a.

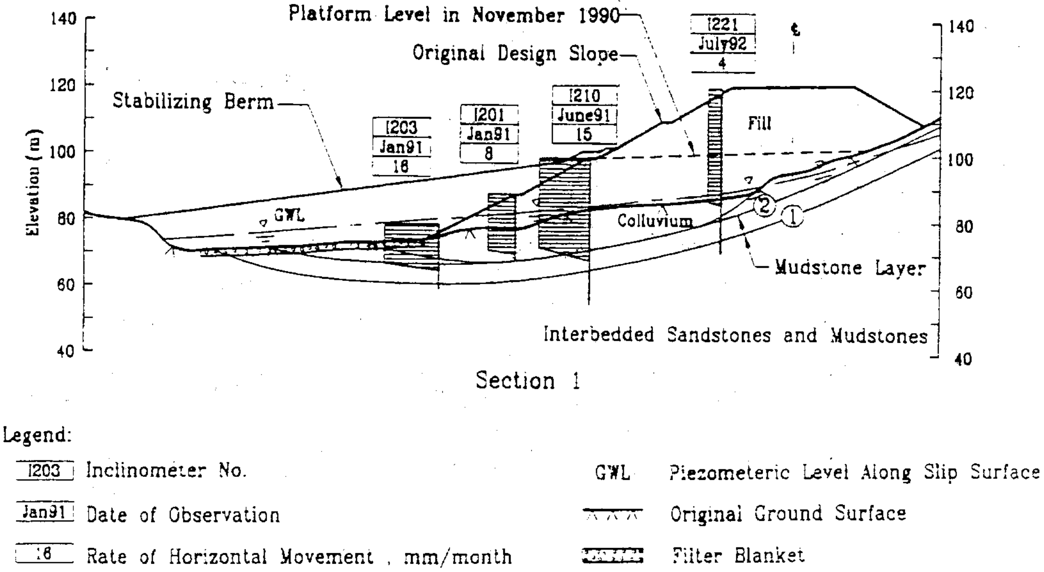

Figure 3. Geologic profile for the road embankment.

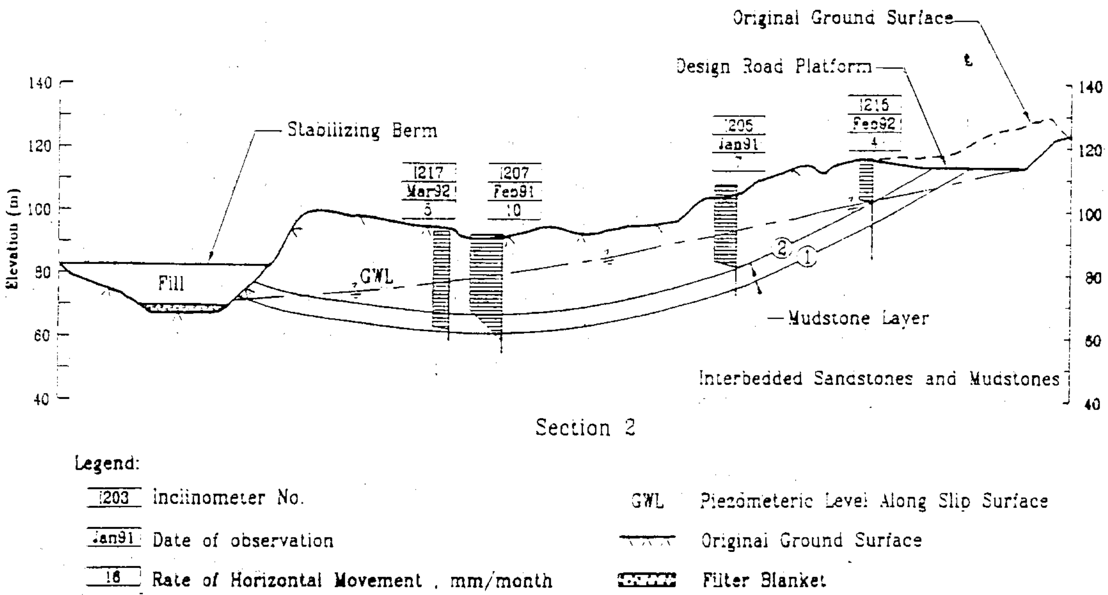

Figure 4. Geologic profile for the rock slope.

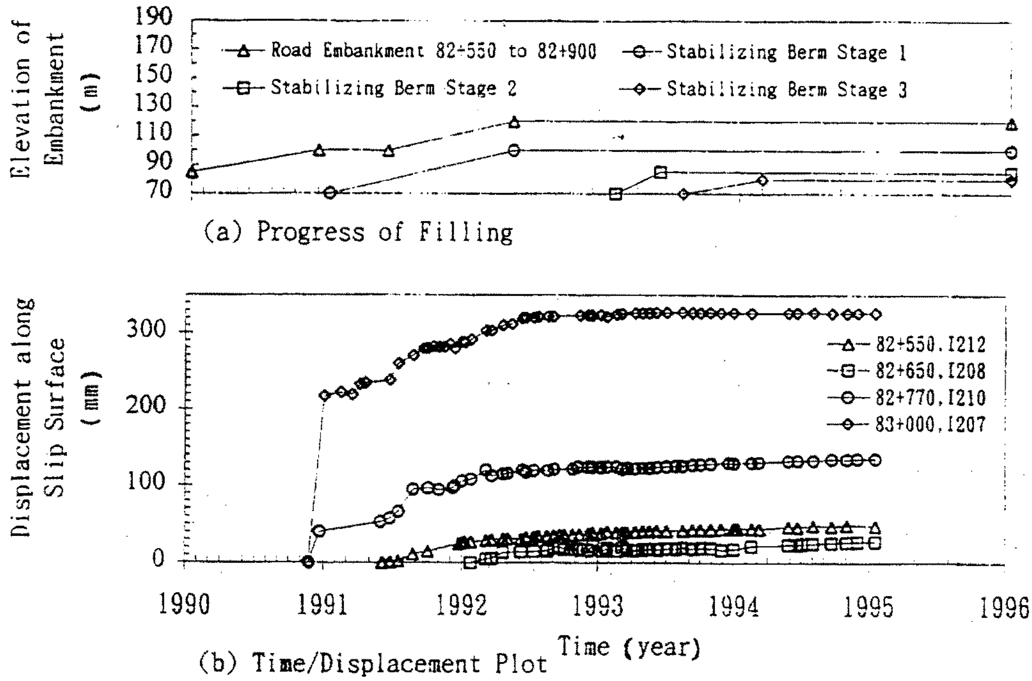

Figure 5. Displacement of the road embankment.

INSTRUMENTATION RESULTS

Piezometer readings indicated that the piezometric levels in the rock strata generally coincided with the original ground surface. However, piezometers P218 and P224, which were installed in the vicinity of the syncline axis (Fig.1), recorded piezometric elevations of 75 m and 79 m, which were 5 m and 9 m above the original ground level, respectively. During the period from 1991 to 1995, rises in groundwater level of less than 1 m due to rainfalls were recorded by the piezometers.

As shown in Figures 3 and 4, from the inclinometer monitoring results and from the slickensided planes observed at the rock cores (boreholes I207 and I205) at depths of 28 m and 26 m, the slip surfaces of the unstable slopes located at chainages from 82+770 to 83+000 could be inferred. Two slip surfaces, namely, mudstone layers 1 and 2, were indentified. These two mudstone layers intersected the syncline axis at elevations of 61 m and 68 m, respectively. The southern block of the rock slope slid along mudstone layer 1, while the northern block slid along mudstone layer 2. The landslip at chainage 82+770 occurred along mudstone layer 2.

The slope movement monitoring results are plotted in Figure 5b, and the rates of movement are summarized in Table 1. The maximum rate of movement occurred at the beginning of the landslips. The monthly rates of 190 mm and 50 mm were recorded at the rock slope at chainage 83+000 and at the fill slope at chainage 82+770, respectively.

During the construction of stabilising berm Stage 1, the monthly rates of slope movement at chainages 83+000, 82+770 and 82+700 to 82+550 were 5.5 mm, 4.4 mm and 2.5 mm, respectively. It should be noted that although creep was observed at chainages 82+700 to 82+550, no signs of distress could be found on the fill embankment.

Table 1. Summary of the movement of slopes

Type of Slope	Chainage	Rate of Horizontal Movement (mm/month)			
		Nov.1990-Dec.1990	Jan.1991-Apr.1992	May 1992-Apr.1993	May 1993-Dec.1995
Rock	83+000	190	5.5	0.8	0
Fill	82+770	50	4.4	0.5	0.4
Fill	82+500-82+700	---	2.5	0.5	0.4

After the stabilization of berm Stage 1 was complete, the monthly rates of movement were significantly reduced to between 0.8 and 0.5 mm for the slope located at chainages from 83+000 to 82+550. As soon as stabilization of berm Stages 2 and 3 was commenced, the movement of the rock slope at chainage 83+000 virtually ceased, while the movement of the fill embankment located from chainages 82+770 to 82+550 was further reduced to the monthly rate of 0.4 mm.

GEOTECHNICAL PROPERTIES

On the basis of the results of laboratory testings, fresh mudstones had the general properties of clay content ranging from 25% to 33%, and the water content ranging from 6% to 23%. The total unit weight varied between 22.7 kN/m^3 and 20.1 kN/m^3. Reversal direct shear tests on pre-cut cored specimens of fresh mudstones were conducted. With the zero cohesion intercept, the peak and residual effective internal angle of shearing resistance, ϕ'_p and ϕ'_r were 29° and 22° for the peak and the residual strengths, respectively.

A block sample of about 400 mm in size was collected from a slicken-sided surface which had been exposed by a plane failure in the mudstone at chainage 81+550. A drained reversal direct shear test was conducted. The results of shear strengths at various displacements are presented in Figure 6. The shear strength, τ is normalized with the effective normal stress, σ'_n. The term τ/σ'_n is equal to $\tan\phi'$. Figure 6 shows that the shear strength of the mudstone depends upon the range of normal stress and the amount of dispacement. At a displacement of 9 mm, the τ/σ'_n values are 0.23 to 0.41. At displacements beyond 50 mm, the τ/σ'_n value reduces to the residual strength of 0.17, corresponding to the ϕ'_r value of 9.6°. On the other hand, with the same amount of displacement, higher shear strengths are available at lower effective normal stresses ranging from 120 kPa to 240 kPa. However, this low effective stress effect diminishes at displacements beyond 50 mm.

BACK ANALYZED STRENGTH

Janbu's routine method (Janbu, 1972) was adopted for slope stability analyses. Since failure had already occurred, the factor of safety of the unstable slope was equal to unity. The actual shear strengths along the failure surfaces could be assessed as long as the groundwater levels and the positions of the slip surfaces were observed.

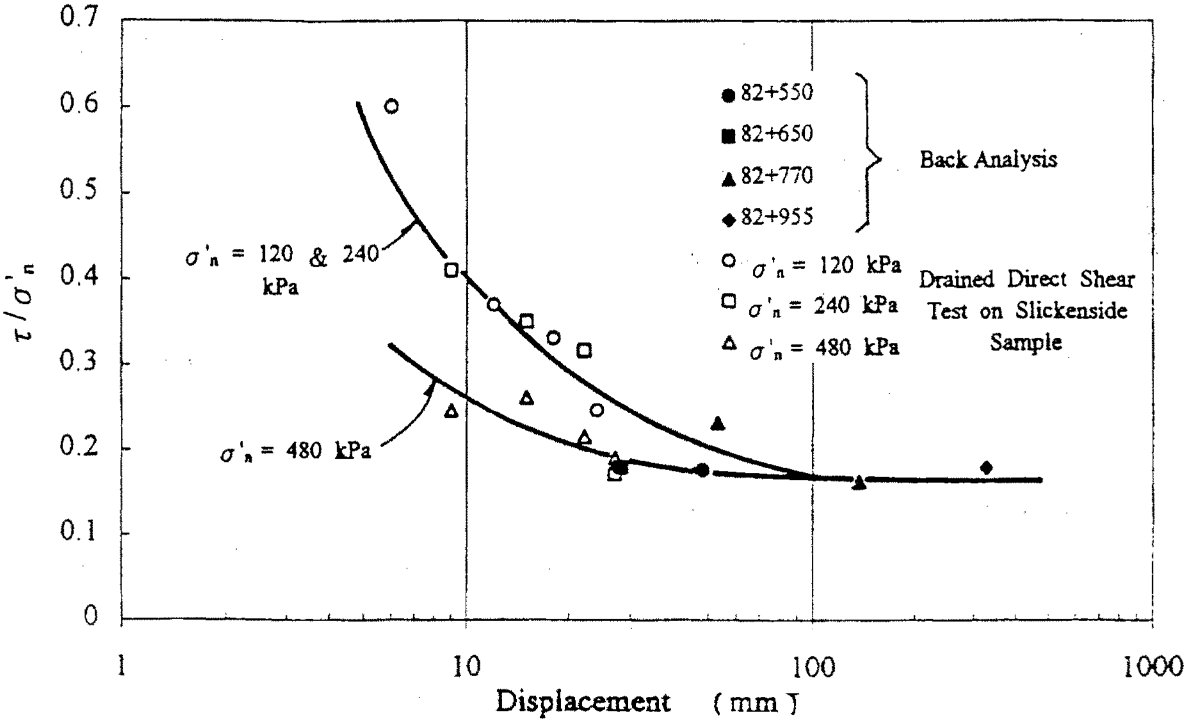

Figure 6. Stress-displacement curve for mudstone.

Assuming a zero cohesion intercept, the effective angle of shearing resistance, ϕ', for the mudstone was calculated. Results of the back analyzed ϕ' values for various slopes are presented in Figure 7. It can be seen that the actual shear strengths which were mobilized along the slip surfaces are close to the residual shear strengths which were obtained from the direct shear test conducted on the slickensided sample.

The shear strengths at effective normal stress ranging from 20 to 50 kPa were back-analyzed from plane failures of the rock slopes located at chainages 80 k to 82 k. The rock wedges were 10 to 20 m in height and slid along the bedding planes of the mudstones. The residual strength envelope for mudstones can be expressed by:

$$\tau = \sigma'_n \tan\phi'_r$$

where τ is the shear stress, and ϕ'_r is the effective residual friction angle. For effective normal stresses ranging from 250 to 600 kPa, ϕ'_r is equal to 9.6°. For effective normal stress less than 250 kPa, ϕ'_r is 14°. These back-analysed strengths of mudstones are slightly less than those of the highly weathered mudstones as reported by Skempton (1985).

CONCLUSIONS

The results of studies on landslips occurring along the bedding planes of the mudstones of the Cholan Formation yield the following conclusions:

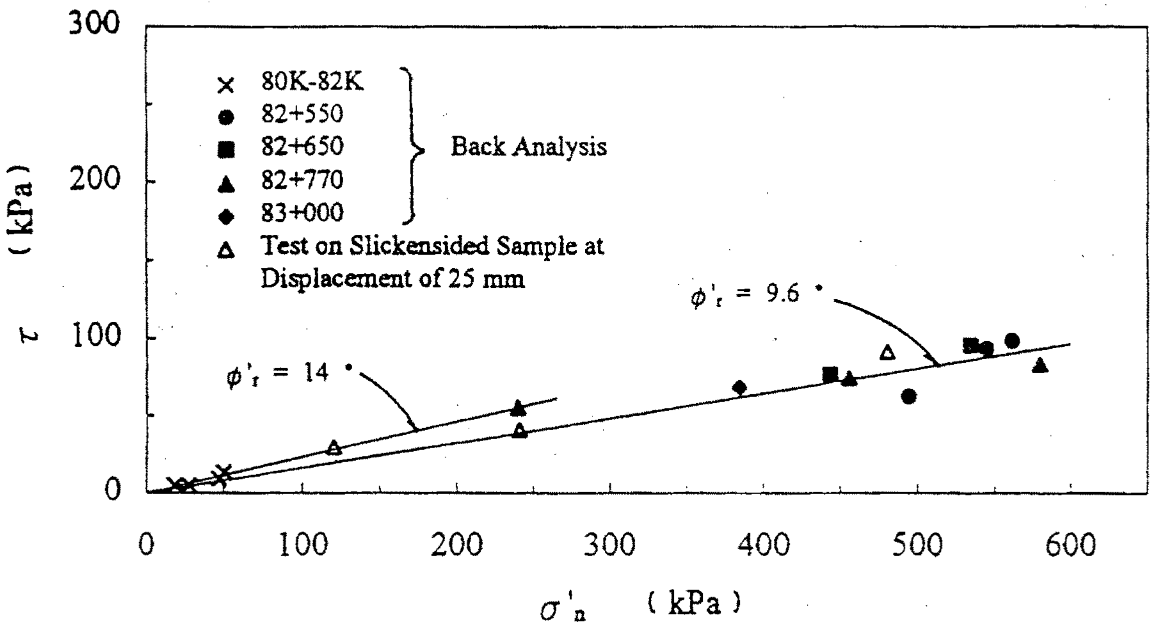

Figure 7. Residual stress envelope for mudstone.

1. The rock strata of mudstones around synclines are landslip prone areas. Factors contributing to instabilities include the low shear strength of the mudstones, inclined bedding planes, high piezometric levels and the undermining of the toe by down-cutting streams.
2. Results of back analyses indicate that the residual shear strength parameter ϕ'_r is 9.6° for mudstone. This back analyzed strength agrees with that obtained from the result of a reversal direct shear test conducted on a slickensided sample.
3. Based on the results of laboratory testings and field observations, the residual strength is fully mobilized when displacements of the slip mass exceed 50 mm to 100 mm or when the effective normal stress exceeds 250 kPa.

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