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THE MAJOR CAUSE OF ANCHOR BLOCK SLOPE FAILURES

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Abstract

Currently, a large number of anchors are used in Taiwan to stabilize slopes. The design specifications have been continuously revised after the occurrence of anchor block slope disasters. However, disasters at anchor block slope that meet design specifications have become more and more serious, even in the absence of wind, rain, and earthquakes. This proves that the current design specifications do not include the major cause that induces anchor block slope failures. In view of this, this paper takes the anchor block slope disaster happened at 3.1km mark of the Formosa Freeway as an example. By comparing the endoscope inspection results and those results from in-situ tensile strength tests, it was found that, although the anchor was rusted, the tensile strength was still high. Therefore, in this paper, a theoretical equation is derived and used to find whether the anchor block slope has experienced a very slow sliding process before the disaster. During this process, the tensile strength of the anchor has been significantly reduced to zero with the increase in the amount of shear banding. Therefore, it is proven that shear banding is the major cause that causes the anchor block slope failure. Finally, it is suggested that anchor design specifications should restrict the amount of shear banding, as well as regulate that once anchors are used to stabilize a slope, the amount of shear banding must be monitored in order to greatly reduce the occurrence of anchor block slope failures.

Keywords: Anchor Block Slope, Disaster, Corrosion, Shear Banding.

Introduction

Anchor block slope disasters have continued to occur in Taiwan over the past 20 years. The severe cases include: (1) during Typhoon Winnie in 1997, the Lincoln County disaster occurred in New Taipei City (detailed in Figure 1), killing 28 people; 80 households were fully destroyed and 20 were partially destroyed; and (2) in 2010, an anchor block slope disaster occurred at the 3.1km mark of the Formosa Freeway (detailed in Figure 2), even though the site was absent of wind, rain and earthquakes, the disaster still occurred, causing all the lanes to be buried by falling rocks, and burying 4 people alive.

Figure 1. Lincoln County anchor block slope disaster (New Taipei City Government, 1997)

(a) Photographed from the southwest to the northeast (Water Resources Department Newsletter, 2018)

(b) Photographed from the northeast side to the southwest (Taiwan Geotechnical Society, 2011)

(c) Photographed from the southeast to the northwest (Taiwan Geotechnical Society, 2011)

Figure 2. Anchor block slope disaster happened at the 3.1km mark of the Formosa Freeway

Figure 3. The rusted snapped anchors remaining on the sliding failure plane

(a) The snapped anchor

(b) The unsnapped anchor

(c) The anchor affected by groundwater

Figure 4. Endoscopic inspection results for the corrosion conditions of the anchors remaining in the anchor block slope below the sliding failure plane (Taiwan Geotechnical Society, 2011)

However, it can be seen from Figure 2 that the sliding failure of the anchor block slope was caused by a yellow sandstone block slid on the top surface of grey shale. After the failure sandstone was cleared, Figure 5 shows that the entire sliding failure plane could be divided into upper and lower parts. The upper part of the sliding failure plane was not equipped with anchors, while the lower part was equipped with anchors. Figure 5a shows that the cracking was not severe on the surface of the shale for the upper part of the sliding failure plane. Figure 5b shows that the cracking was very serious on the surface of the shale for the lower part. A comparison of Figure 5a and Figure 5b shows that the anchors remaining on the sliding failure plane were rusted, but the tensile strength was still high, causing the surface of the shale adjacent to the anchors to be seriously cracked when the anchors were pulled off. In addition, the in-situ tensile strength test results of the anchors can also be used to prove that the anchors were rusty, but their tensile strengths were still high.

(a) Upper part of the sliding failure plane (no anchor deployed)

(b) Lower part of the sliding failure plane (the deployed anchors have been pulled off)

Figure 5. Comparison of the degree of rupture of the sliding failure plane

In-situ Anchor Tensile Strength Test Results

After the anchor block slope disaster happened at the 3.1km mark of the Formosa Freeway, the investigators conducted two sets of in-situ tensile strength tests on the existing anchors. The first set of tensile strength tests was for the snapped anchors remaining on the sliding failure plane (detailed in 3c). The second set of tensile strength tests were for unsnapped anchors that remained below the sliding failure plane. The results of these two sets of tensile strength tests are described below.

Tensile strength test results for the first set of anchors

Figure 5b shows that the snapped anchors remaining after the disaster including steel strands of the free length on the sliding failure plane and the fixed length embedded below the sliding failure plane.

Before conducting the tensile strength test on the first set of anchors, five anchors were randomly selected; the steel strands were cut and prepared as test samples (detailed in Figure 6). Afterwards, the tensile strength test of the anchors was carried out separately, and the test results are shown in detail in Table 1.

Figure 6. Test anchors remaining on the sliding failure plane

Table 1. The tensile strength test results of the first set of anchors (Taiwan Geotechnical Society, 2011)

Test sample numbers	Basic information of anchor components					Maximum pulling force applied
	Steel strand numbers-diameters	Anchor length	Steel strands of free length	Fixed length	Designed tensile strength	
1	7-12.7mm	20m	10m	10m	588kN	1098kN
2	7-12.7mm	18m	8m	10m	588kN	1098kN
3	7-12.7mm	28m	18m	10m	588kN	1098kN
4	7-12.7mm	22m	12m	10m	588kN	1098kN
5	7-12.7mm	24m	14m	10m	588kN	1098kN

Note: None of the 5 test anchors snapped when a tension of 1098kN was reached.

The first set tensile strength test results of anchors in Table 1 show that: (1) when the tensile force applied reached 1098kN, all five test anchors did not snap; in other words, the tensile strength of all the test anchors was greater than 1098kN; and (2) since the initial tensile strength of this type of anchor was 1285kN (Taiwan Geotechnical Society, 2011), it is known that the influence of corrosion on the tensile strength of the anchor is not large.

Tensile strength test results for the second set of anchors

For the second set of anchor tensile strength tests, five test anchors were randomly selected from the remaining anchors shown in Figure 7. Since the 5 test anchors were located below the sliding failure plane, they were retained intact after the disaster. Figure 8 shows the strength test of the second set of anchors in the field; the test results are shown in detail in Table 2.

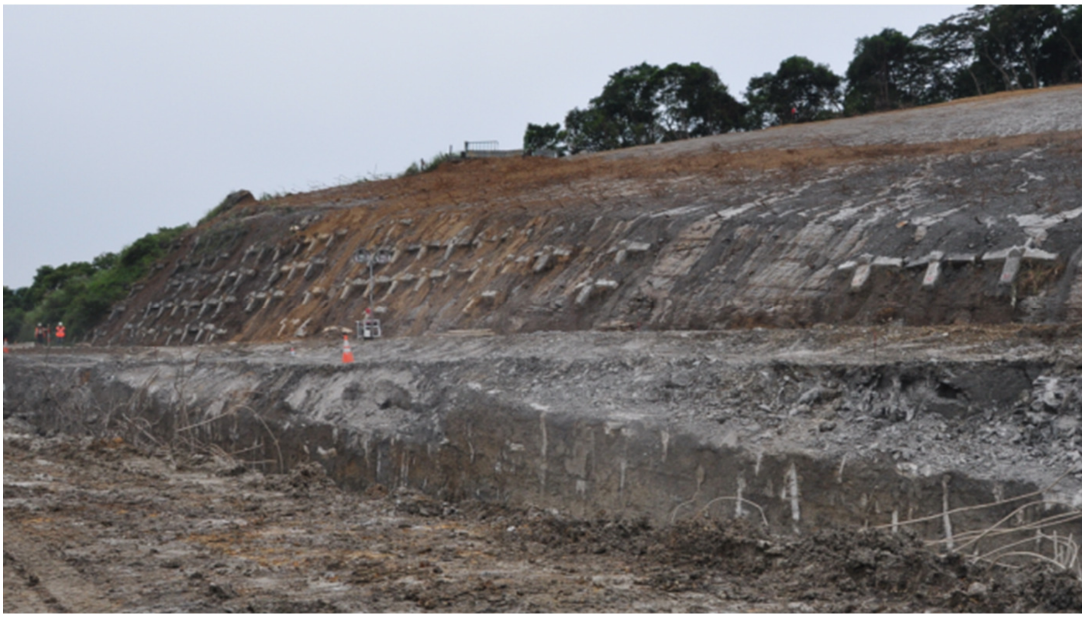


Figure 7. Anchors that were completely intact below the sliding failure plane

Figure 8. Tensile strength tests for the second set of anchors (Taiwan Geotechnical Society, 2011)

Table 2. Tensile strength test for the second set of anchors (Taiwan Geotechnical Society, 2011)

Test sample numbers	Basic information of anchor components					Maximum pulling force applied
	Steel strand numbers-diameters	Anchor length	Steel strands of free length	Fixed length	Designed tensile strength	
1	7-12.7mm	20m	10m	10m	588kN	920kN
2	7-12.7mm	18m	8m	10m	588kN	883kN
3	7-12.7mm	24m	14m	10m	588kN	618kN
4	7-12.7mm	22m	12m	10m	588kN	588kN

5	7-12.7mm	24m	14m	10m	588kN	490kN

For the 5 test anchors that were completely intact on the remaining anchor block slope after the disaster occurred, according to Table 2: (1) among all 5 test anchors, only one test anchor had a lower tensile strength than the designed tensile strength; and (2) since the initial tensile strength of this anchor was 1285kN (Taiwan Geotechnical Society, 2011), it is known that the tensile strength reduction of these 5 experimental anchors were 28.4%, 31.3%, 51.9%, 54.2%, and 61.9%.

Based on the results of the two sets of anchor tensile strength tests, although the anchors were rusted, they still retained a relatively high tensile strength; therefore, anchor corrosion is obviously not the major cause of the disaster. When the investigators use anchor corrosion as the major cause of the disaster, the investigation report will clearly deviate from the results of the in-situ anchor tensile strength tests.

Also, the first set of anchor tensile strength test results shown in Table 1 are much higher than the second set of anchor tensile strength test results shown in Table 2.

Signs Of Shear Banding For The Anchor Block Slope Before The Disaster

Before the occurrence of the anchor block slope disaster happened at the 3.1km mark of the Formosa Freeway, there were some signs of shear banding on the anchor block slope. These signs included:

1. During Typhoon Xangsane in 2000 and Typhoon Nari in 2001, slope failures occurred locally on the southwest side of the sliding block (detailed in Figure 9).
2. With the shear banding of the block, the pier of the elevated land bridge, as shown in Figure 10, was squeezed, causing bending deformation and cracking phenomena to appear locally.
3. The anchor block slope induced shear textures of various strikes, as shown in Figure 1, with slow sliding.
4. The cementitous materials of the shear band rocks exuded onto the retaining wall (detailed in Figure 12a).
5. Due to the excessive amount of shear banding, local abrupt cracking occurred on the abutment of the expressway (detailed in Figure 12b).

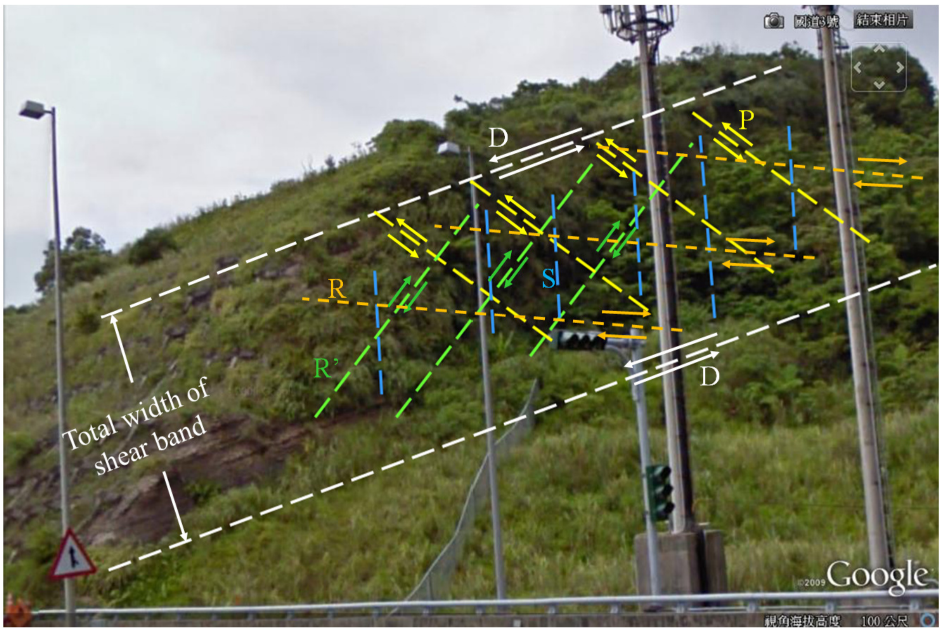
(a) During Typhoon Xangsane

(b) During Typhoon Nari

Figure 9. Local slope failures appearing on the southwest side of the sliding block (Taiwan Geotechnical Society, 2011)

Figure 10. Bending deformation and cracking on the pier (Google Earth, 2010)

(a) Before overlaying



(b) After overlaying

Figure 11. Shear band and various shear textures existing in the sliding block (background image from Google Earth, 2010)

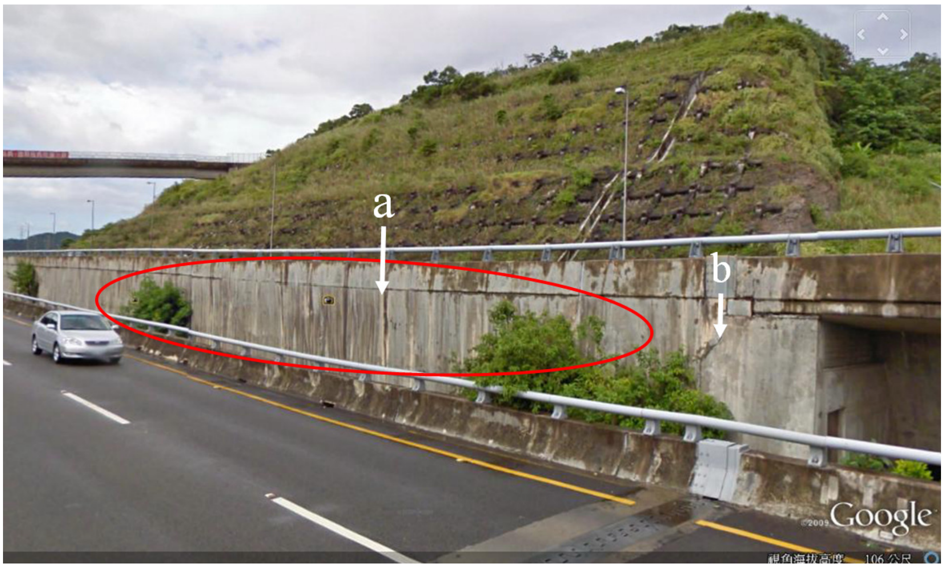


Figure 12. The a in the figure is the cementitous materials exuded from the shear band rock to the retaining wall, and b is the local fracture of the abutment induced by shear banding (background image from Google Earth, 2010)

Anchor Stabilized Shear Banding Rock Slope

In general, engineers often use anchors to stabilize the shear banding rock slope. When the anchors have just been installed, the amount of shear banding for the rock slope shown in Figure 12 is e = 0. After the anchors are installed for a period of time, the amount of shear banding for the rock slope is e > 0.

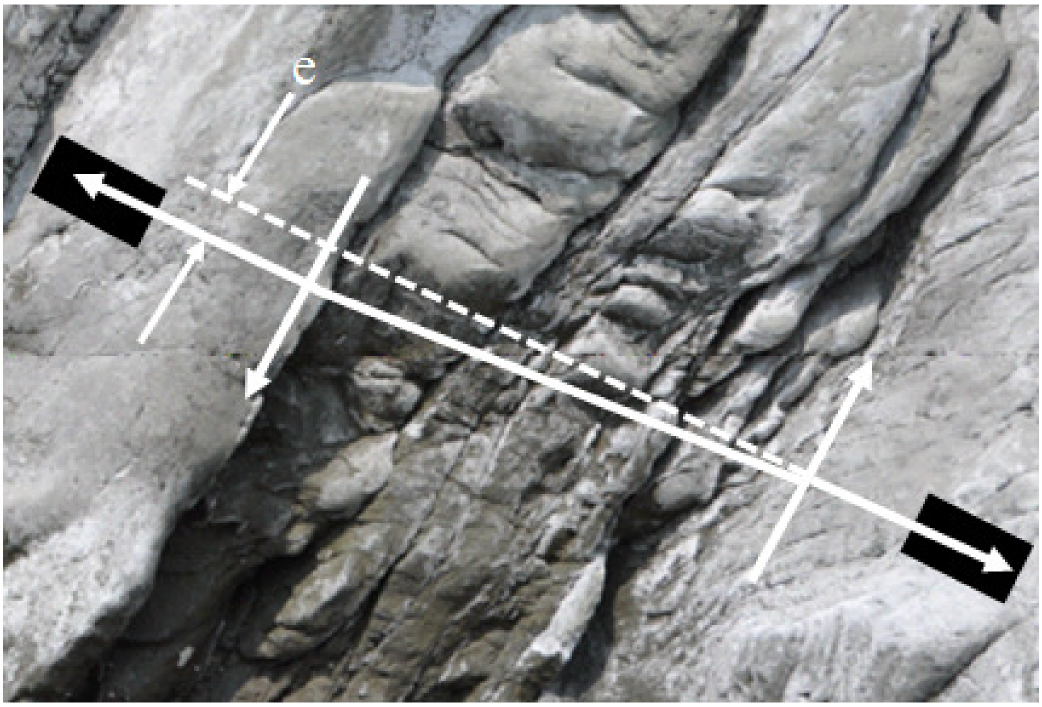


Figure 12. Schematic diagram of the anchor through the shear banding rock (redrawn from Hsu et al., 2017)

Formulation Of The Theoretical Equation

Currently, anchor tension strength, as defined by the anchor design specifications (Taiwan Geotechnical Society, 2000), refers to the tensile strength of the anchor P_{f0} corresponding to the amount of shear banding e = 0, but not the tensile strength of the anchor P_f corresponding to the amount of shear banding e > 0. Therefore, anchor engineers mistakenly consider the tensile strength of the anchor as a fixed value P_{f0}. Even during the investigation of the disaster caused by the sliding failure of the anchor block slope, it is completely overlooked that the tensile strength of the anchor has been significantly reduced while shear banding occurs. Therefore, in the investigation report on the cause of the disaster, it is stated that "the tensile strength of the anchor provides the last force to stabilize the slope before the slope slides… (Taiwan Geotechnical Society, 2011)". Thus, the cause of the anchor block slope disaster is attributed to the anchor corrosion. However, even though anchor corrosion is not the main cause of the anchor block slope disaster, the anchor design specifications have continued to strengthen the anchor corrosion prevention regulations. As a result, anchor block slope disasters have not reduced; instead, they have become more and more serious.

In view of this, this section will formulate the theoretical equation for the reduction of the tensile strength of the anchor P_f with an increase of the amount of shear banding *e*, in order to quantify the tensile strength of the anchor P_f actually existing during the sliding failure of the shear banding rock slope. Subsequently, the major cause that induces anchor block slope disasters can be found.

For the anchor block slope shown in Figure 13, if the anchor diameter is D_A, its section area $A = \pi D_A^2 / 4$, and its section modulus $S = \pi D_A^3 / 32$.

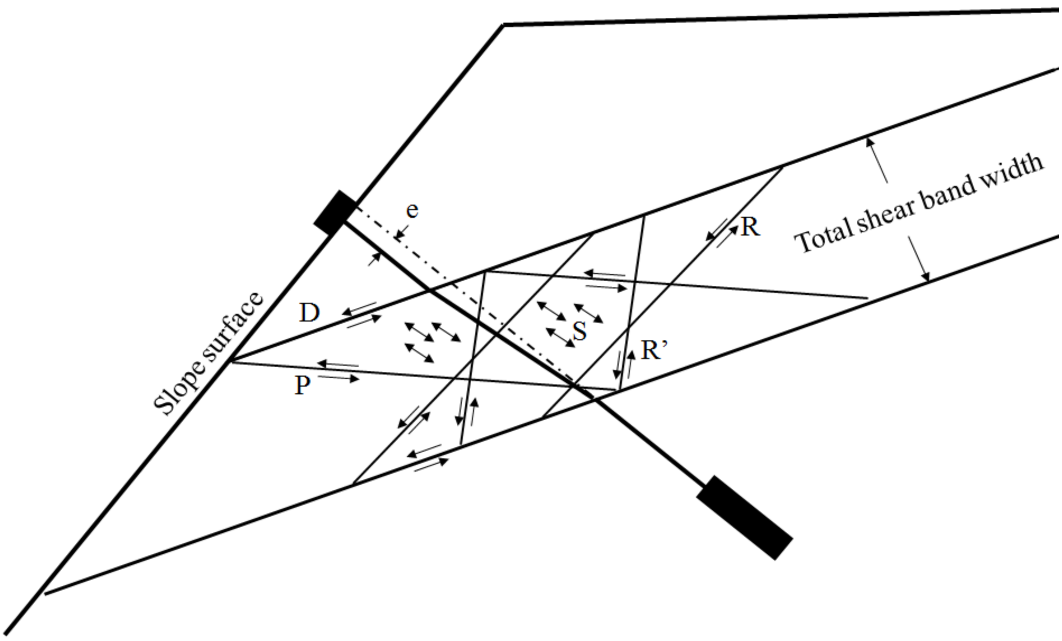


Figure 13. Schematic diagram of the anchor passing through the shear band with a displacement of *e*

If the tensile strength of the anchor is P_{f0}, its stress under failure conditions is $\sigma_f = P_{f0} / A$. If the tensile strength of the anchor is P_f, its stress under failure conditions is $\sigma_f = (P_f / A) + (M_f / S)$. Therefore:

$$\frac{P_{f0}}{A} = \frac{P_f}{A} + \frac{M_f}{S} = \frac{P_f}{A}\left(1 + \frac{8e}{D_A}\right) \qquad \text{............(1)}$$

Reducing Equation 1:

$$\frac{P_f}{P_{f0}} = \frac{1}{1 + 8e / D_A} \qquad \text{............................ (2)}$$

Application Of The Theoretical Equation

Using Equation 2, the relationship of P_f / P_{f0} changing with e / D_A can be drawn (shown in Figure 14). When e / D_A increases from 0 to 1, P_f / P_{f0} decreases dramatically; when e / D_A continues to increase above 1, the P_f / P_{f0} reduction tends to slow down. Eq. 2 can be used to calculate that, as e / D_A continues to increase from 0 to 1/24, 1/8, 1, 16/9, and 32/9, P_f decreases to $0.75P_{f0}$, $0.5P_{f0}$, $0.25P_{f0}$, $0.125P_{f0}$, $0.111P_{f0}$, $0.0657P_{f0}$, and $0.034P_{f0}$, respectively.

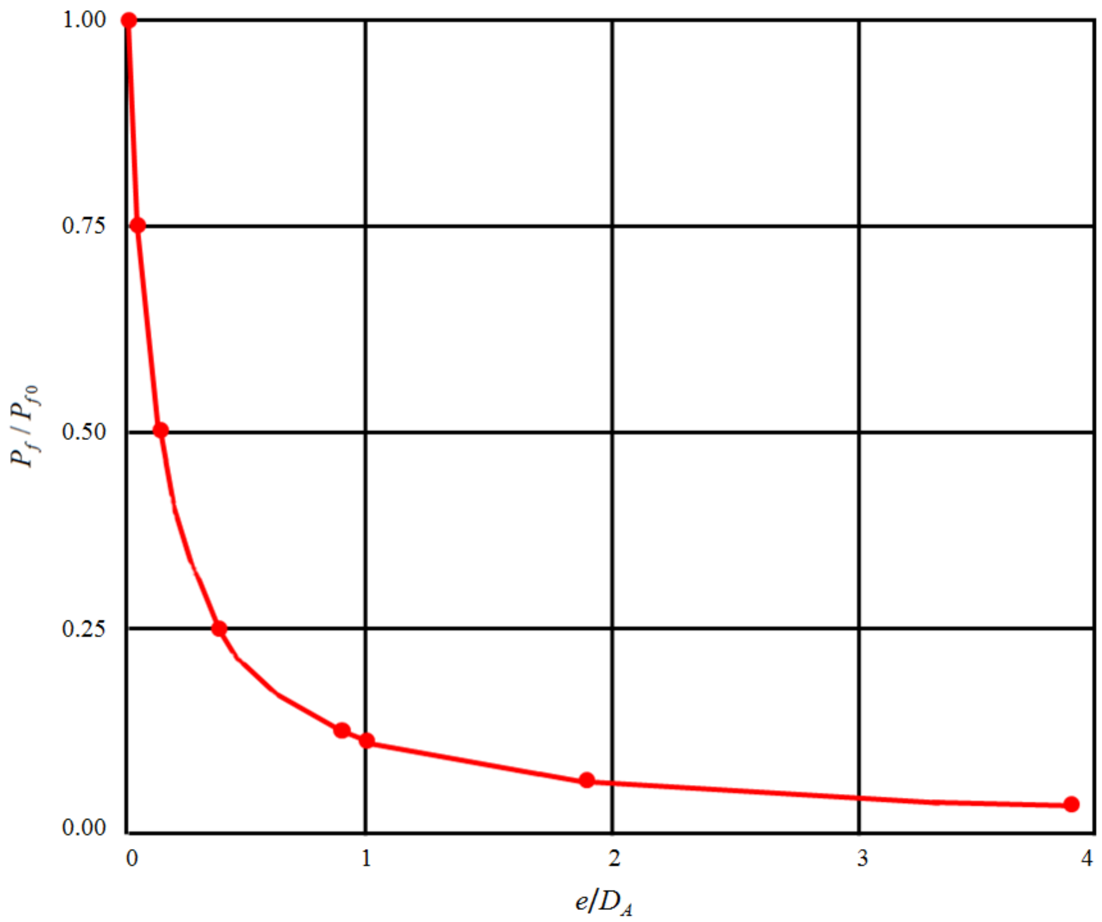


Figure 14. Relationship between P_f / P_{f0} and e / D_A

Taking the anchor block slope located at the 3.1km mark of Formosa Freeway as an example, since the anchor diameter D_A is 9cm and P_{f0} is 1285kN, it can be calculated from Eq. 2 that: (1) when e=0.375cm and e/D_A =1/24, P_f is 963.8kN; (2) when e=1.125cm and e/D_A =1/8, P_f is 642.5kN; (3) when e=9cm and e/D_A =1, P_f is 142.6kN; (4) when e=16cm and e/D_A =16/9, P_f is 84.4kN; and (5) when e=32cm and e/D_A =32/9, P_f is 43.7kN.

For a very slow sliding of an anchor block slope, since the amount of shear banding e of the sliding block relative to the stable block is up to 1.6cm each year (ResearchGate, WP/WLI 1995 and Cruden and Varnes 1996), once the anchor block slope is maintained for 10 years at a very slow sliding state, the theoretical P_f has been reduced to 84.4kN. After the anchor block slope is maintained for 20 years with a very slow sliding state, the theoretical P_f has been reduced to 43.7kN.

However, the actual anchor may have the fracture damage of the free length shown in Figure 15 when the amount of shear banding is adequately large; therefore, it is impossible to maintain a very slow sliding state for 20 years.

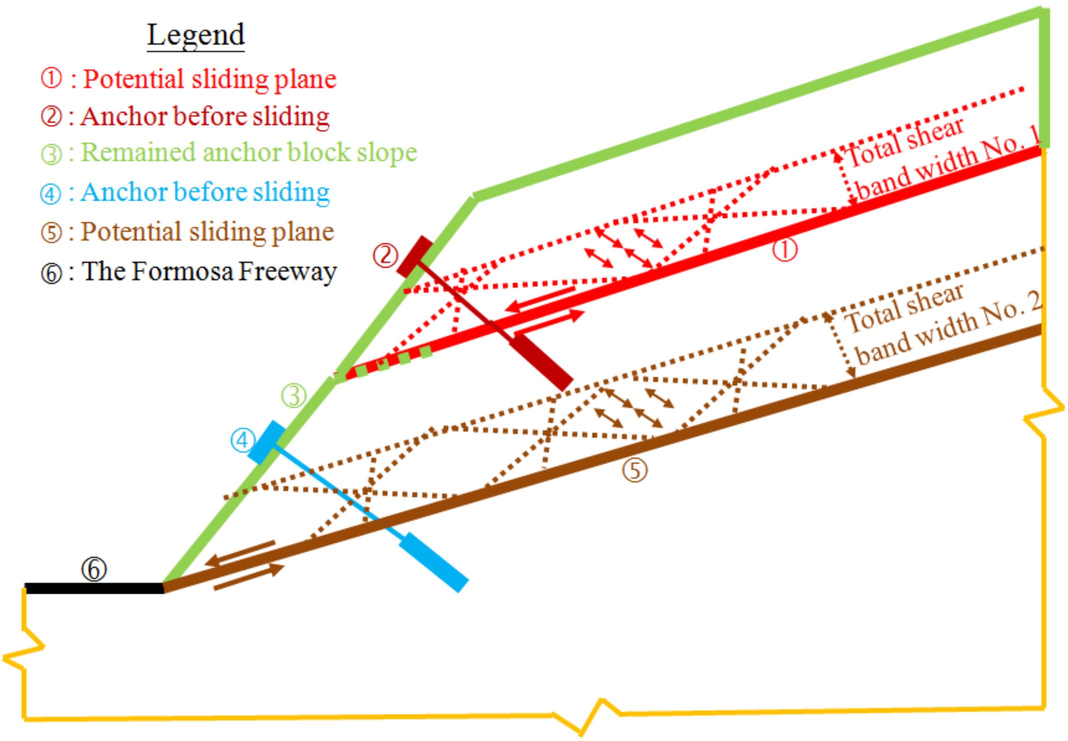
Figure 15. Damage of the anchor head induced by the breaking of the free length

Comparison And Discussion Of Results

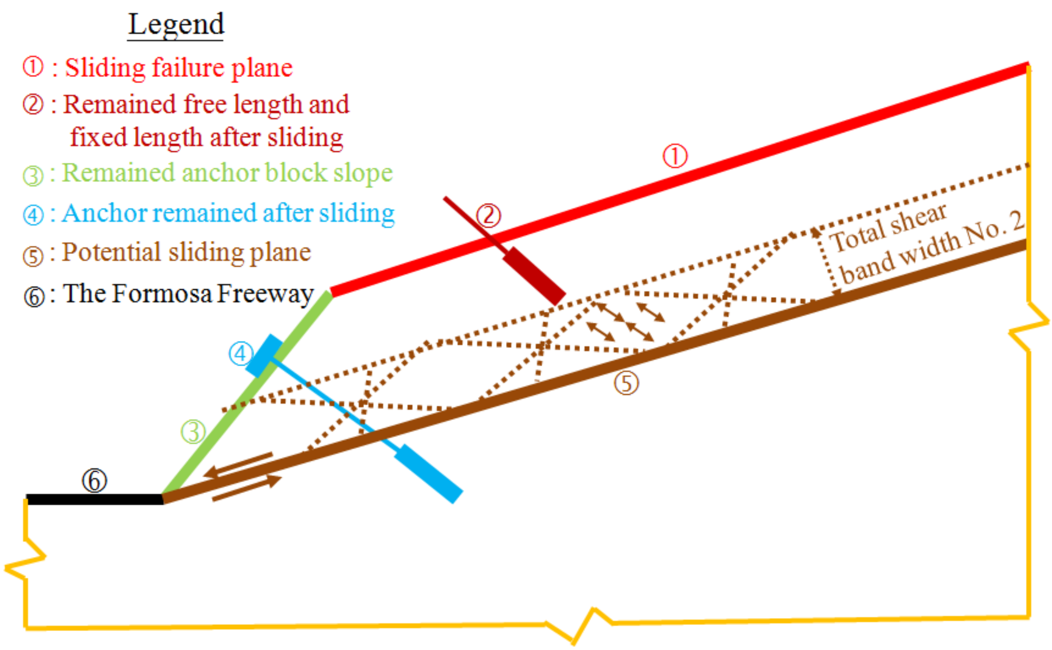
For the snapped anchor remaining on the sliding failure plane as shown in Figure 3 or Figure 5b, since the block sliding on the potential sliding plane, shown in Figure 16a①, is disappeared, all shearing textures located within the total shear band width No. 1 no longer exist. Therefore, the anchor (Figure 16b②) remaining on the sliding failure plane shown in Figure 16b① is actually not affected by the total shear band width No. 1. In other words, the tensile strength test results of the first set of anchors shown in Table 1 are derived from $e = 0$; the tensile strength test results of all five anchors are therefore higher than 1098kN.

For the anchor in the residual anchor block slope below the sliding failure plane after the disaster shown in Figure 7 and Figure 16b④, the effect of the total shear band width No. 2 on all five anchors are shown in Figure 12, Figure 13, and Figure 16. In other words, the tensile strength test results of the second set of anchors shown in Table 2 are derived from $e > 0$; therefore, after the tensile force is applied at 920kN, 883kN, 618kN, 588kN, and 490kN, the five anchors are pulled off respectively. Equation 2 can inversely calculate the amount of shearing banding e of the five anchors, equal to 0.446cm, 0.5125cm, 1.2143cm, 1.3313cm, and 1.8225cm, respectively.

The amount of shear banding e obtained by the above-mentioned back calculation can be achieved when the slope is in a very slow sliding state; therefore, it is known that when the in-situ anchor block slope is in a very slow sliding state, the anchor tensile strength P_f will decrease due to the increase of the amount of shear banding e. The value reaches zero after breaking.



(a) Before landslide



(b) After landslide

Figure 16. Schematic diagram of anchors, anchor components, and shear textures existing in the total shear band width that exists on the anchor block slope and sliding failure plane

Conclusions And Suggestions

An anchor block slope actually experienced a very slow sliding state before a disaster occurred. Therefore, in the investigation of the cause of an anchor block slope disaster after its occurrence, the influence of shear banding on the tensile strength of the anchor must be considered. In the past, the anchor design specification completely ignored the influence of shear banding on the tensile strength of the anchor, so it was incorrectly assumed that the anchor tension strength is a fixed value during the very slow sliding process of the anchor block slope. Hence, in the investigation after the occurrence of an anchor block slope disaster, anchor corrosion was mistaken as the major cause.

In view of this, the authors first compare the endoscopic detection results of the anchor with the results of the in-situ tensile strength test. A theoretical equation is then formulated, in which the anchor tensile strength P_f decreases as the amount of shear banding e increases. Then, taking the anchor block slope disaster happened at the 3.1km mark of the Formosa Freeway as an example, the major cause for the anchor block slope disaster is investigated. The results of the investigation support the following five conclusions.

1. After the anchor block slope disaster occurred at the 3.1km mark of the Formosa Freeway, the investigators examined the major cause of the disaster, including the corrosion conditions of the anchors obtained by performing the endoscope inspections. However, the tensile strength of the rusted anchor is high based on the results of the in-situ tensile test; under the circumstances, the investigators deliberately ignored the results of the tensile test, and, based only on the results of the endoscope inspections, suggested anchor corrosion as the major cause for the disaster.

2. In the past, the design specification of anchors was continuously revised according to the investigation results of the anchor corrosion, but the anchor block disasters continued to occur. There is a tendency for them to be more and more damaging, showing that anchor corrosion is not the major cause of anchor block slope disasters.

3. Before the occurrence of disasters, the anchor block slopes experienced very slow sliding conditions, so there must be some clear signs of shear banding.

4. The results of the in-situ anchor tension test and the theoretical equation formulated in this paper show that the tensile strength of the anchor decreases greatly with an increase of the amount of shear banding. Therefore, when engineers use the anchor to stabilize the slope, the major cause of the anchor block slope disasters is shear banding.

5. Presently, according to the latest anchor design specifications, the old anchors of various roadway slopes in Taiwan have been replaced with double-layer anti-corrosion anchors; however, these anchor block slopes will still slide and break due to shear banding in the future.

Based on the above five conclusions, the authors suggest that in the future revisions of anchor design specifications, the influence of shear banding must not be neglected. Therefore, once the anchor is used to stabilize the slope, it is suggested to monitor the amount of shear banding. When the amount of shear banding is greater than the warning value, the reinforcement of the stability of the anchor block slope must be implemented to compensate for the lost sliding resistance, so that the stability of anchor block slopes can be ensured.

Acknowledgements

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