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Behavior of Frictional Shallow Anchors subjected to Vertical Loadings in Rock

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1. Introduction

Anchors can be used for reinforcement of near slope stability of radioactive waste disposal facilities. This paper presents the results of full-scale loading tests performed frictional anchors to various lengths at several sites in Korea. Various rock types were tested, ranging from highly weathered shale to sound gneiss. In many tests, rock failure was reached and the ultimate loads were recorded along with observations of the shape and extent of the failure surface. Laboratory tests were also conducted to investigate the influence of the corrosion protection sheath on the bond strength. Based on test results, the main parameters governing the uplift capacity of the rock anchor system were determined.

2. Laboratory Tests and Results

2.1 Test Set-up

The present laboratory pullout tests were conducted to determine bond strength and bond stress-shear slip relation at the tendon/grout interface when a corrosion protection sheath is installed in the cement-based grout. Two different anchor types were considered in these tests, as shown in Table 1. The first type is conventional deformed reinforcing bar (hereafter called ″ rebar″) with diameters of 32 mm and 51 mm. The second type is strong Macalloy steel thread bars (hereafter called ″thread bar″) with diameters of 36 mm and 50 mm. These anchors were inserted in cement grout contained in irregularly surfaced rigid steel cylinders.

Table 1. Mechanical properties of anchor and grout types

Anchor types			Yield force (kN)	Compressive strength of grout after 7 days (MPa)
Rebar	SD40-D32	A	305~400	34.3
	SD40-D51	B	770~1011	34.3
Thread bar	36-mm	C	863	34.3
	50-mm	D	1665	34.3

2.2 Results of Pullout Tests

The shear bond strengths at the interface are mainly due to mechanical interlocking and friction associated with movement of the tendons relative to the surrounding grout. Test results supported the following empirical equation relating the ultimate bond strength (τ) to the unconfined compressive strength of grout ($f'c$) as

$$\tau = \alpha \cdot f'c \tag{1}$$

where the constant α was found to be 18.5± 4 % for the rebar and 21.5±4 % for the thread bar. Thus, the Macalloy thread bars have approximately 16 % higher bond strength than the conventional reinforcing bars. It should be noted that Equation (1) is obtained at grout unconfined compressive strength of 34.3 MPa. One of the important findings from these tests is that the measured strains along the corrosion protection sheath were so small that practically the reduction of bond strength by the presence of sheath would be negligible.

Figure 1 shows load-displacement curves obtained from the pullout tests. The load-displacement relations of rebar can be characterized by three distinctive stages. The first stage is related to elastic behavior approximated by a straight line. The second stage is associated with elasto-plastic behavior reaching the ultimate strength. The last stage is related to softening behavior due to progressive debonding, which will eventually approach the residual strength. During the loading stages, the Macalloy thread bars show higher strengths than the conventional rebar. However, as the debonding is completed, both types of anchors reach essentially the same residual strength.

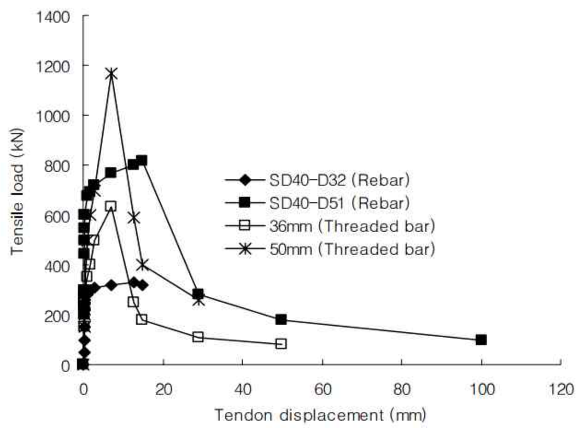


Fig. 1 Load-displacement curves by pullout tests

3. Full Scale Field Tests

3.1 Test Sites and Characteristics of Anchors

Full scale field tests were performed, at three different locations (Taean and Okchun in Chungcheong

Province and Changnyong in Gyeongsang Province). Static pullout tests were conducted for 54 passive rock anchors and 4 anchored footings. In the majority of test sites, flat bed rocks are exposed on the ground surface. To determine the properties of in situ rock mass, 34 rock cores of NX-size were obtained from drilled boreholes in the vicinity of the test sites. The rock cores show some fractures and horizontal thin beds ranging in thickness from 30～200 mm within a depth of 1～5 m.

Table 2 shows geometrical and mechanical properties of the rock mass obtained from these rock core samples. Note that the values of compressive strength represent the unconfined compressive strengths conducted on intact core samples.

Reinforcing steel rods (rebar) used for anchors have a nominal diameter of 32 mm and 51 mm. These steel bars have an elastic limit of 384～504 MPa and an ultimate strength of 553 MPa. A hydraulic crawler drilling machine was used to make 100 mm diameter holes. Anchors were inserted into the holes with cement grout to their full length varying between 1 m and 6 m and pullout tests were then performed after the grout was completed. The cement grout was made from Ordinary Portland Cement (OPC) with a water-cement ratio of 0.4. To offset the shrinkage of cement grout, an expansive agent (CONACE AC) at a ratio of 1 % by cement weight was added. The compressive strength of the grout at 7 days is 34.3 MPa. Strain gages were installed along the rebars for 3 tests in Taean and 2 tests in Changnyong to measure the variation of strain profiles as the applied load increases.

Table 2. Geometrical and mechanical properties of the rock

Testing sites	Taean		Changnyong		Okchun
	Hole No. (TA-1)	Hole No. (TA-4)	Hole No. (CY-2)	Hole No. (CY-3)	Hole No. (OC-5)
Rock type	Gneiss	Gneiss	Shale	Shale	Limestone
Core recovery (%)	93	69	75	40	86
R.Q.D. (%)	84	32	26	10	50
Unit weight (KN/㎥)	29.3	27.6	25.9	26.1	28.6
Poisson's ratio	0.22	0.23	0.20	0.23	0.23
Modulus of elasticity (GPa)	69.8	63.4	62.3	65.6	74.7
Compressive strength (MPa)	112.3	105.6	110.4	99.8	108.3
Friction angle (degree)	39	37	35	37	43
Cohesion (MPa)	43	38	33	34	38

Table 3 show a description of the test setup and installation, respectively, for single rock anchors. As shown in Table 3, single rock anchors were installed over a wide range of rock types and qualities with a fixed anchored depth of 1～6 m. The majority of installations used 51 mm high grade steel rebar to induce rock failure prior to rod failure. However, a few installations included the use of 32 mm rebar at relatively deeper anchored depth so as to induce rod failure.

Table 3. Installation of single rock anchors

Depth (m)	Metamorphic rock (Taean)				Sedimentary rock (Changnyong)	Metamorphic rock (Okchun)	Total
	H	M	S	Sub total	M	M	
1.0	2	4	3	9	-	1	10
1.5	1	-	1	2	2	-	4
2.0	3	4	2	9	3	4 (D=32mm)	16
2.5	-	-	2	2	-	2 (D=32mm)	4
2.6	-	1	-	1	-	-	1
3.0	3	3	-	6	4	2 (D=32mm)	12
3.7	-	-	-	-	3	-	3
4.0	1	-	-	1	-	-	1
5.0	-	-	-	-	-	2 (D=32mm)	2
6.0	-	-	-	-	-	1 (D=32mm)	1
Total	10	12	8	30	12	12	54

(H: Highly weathered, M: Moderately weathered, S: Slightly weathered rock)

Table 4 show a description of the test setup and installation, respectively, for group anchored foundations. The main objective of these full scale tests is to demonstrate the practical applicability of rock anchor foundations subjected to the design uplift load. As illustrated in Table 4, group anchors consisting of 8 holes with anchored depth of 2 m and 5 m were tied to the square foundations (2.5 m x 2.5 m) with depths of 0.8, 1.0 and 1.2 m. Two 32 mm rebars were inserted into each hole.

Table 4. Installation of group rock anchored-foundation

No.	Dim. of foundation (m)	Anchor hole		Tendon (each hole) (SD40-D32)	Depth (m)	Location
		Dia. (mm)	No. of Hole			
1	2.5x2.5x1.2	100	8	2	5.0	Okchun
2	2.5x2.5x1.0	100	8	2	5.0	Okchun
3	2.5x2.5x0.8	100	8	2	5.0	Changnyong
4	2.5x2.5x1.0	100	8	2	2.0	Changnyong

3.2 Results of Single Anchor

The present test results for single anchors show the bond failures along the interface between grout and rock in the case of very shallow anchor depths of 1～1.5 m in highly weathered rock (Figure 2) and the rock pull-up failures in the case of fresh, sound rocks or deeply embedded rock anchors. According to the test results, rock-grout bond failure is governed by the rock conditions and the average bond failure is 10~12 % of the unconfined compressive strength of the surrounding rocks. Bond failure along the interface between the rod and grout was not observed throughout the present tests. For the majority of rock pull-up failures, cracking and heaving on the ground surface were extended radially to a distance equal to the half depth of the anchor, as shown in Figure 3.


Fig. 2 Rock-grout bond failure (Highly weathered, depth=1.0m)   Fig. 3 Rock failure (Moderately weathered, depth=2.0m)

Test sites in the city of Taean are classified as metamorphic gneiss, which covers a wide range of rock conditions. Test results revealed an uplift capacity ranging from 150 kN to 940 kN, which depends mostly on the embedded length, RQD(Rock Quality Designation), and core recovery. As the failure of the rock mass was reached, ″+″ shape cracks developed around the anchor and extended radially. Readings from strain gages installed in moderately weathered rocks along the depth showed a large strain increase at the half depth as the applied load is reached to failures. These test results showed the extent of cracking on the ground surface which is about the half embedded depth.

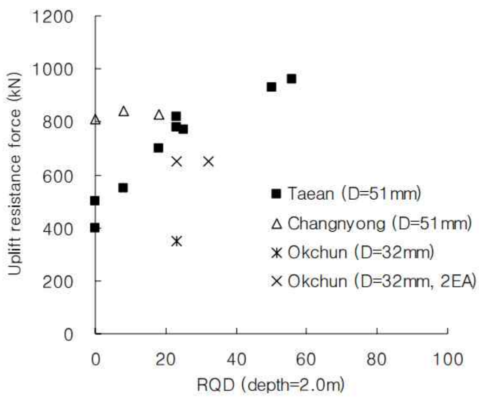
Bond failures between the grout and rock were observed for the anchors with a fixed length of less than 1.5 m, embedded in low RQD rocks.

Test sites in the city of Changnyong are classified as sedimentary shale with a RQD of 0～30 % and a core recovery of 34～73 %. The rock masses were so weak that they could be separated by hand, with substantial horizontal discontinuities. Though the rocks are of very poor quality, it showed an uplift capacity of over 700 kN in the case where the fixed anchor depth was over 3 m. As the applied load reached the uplift capacity of the rock mass, the rock surfaces were pulled up with many small cracks around the anchor. This failure behavior is believed to be due to the separation/loosening of discontinuities associated with the stress concentration on the ground surface around the anchor. Readings from strain gages attached at different depths indicated a sudden large increase of strains at the half length of the fixed anchors as the applied load initiated the rock failure.

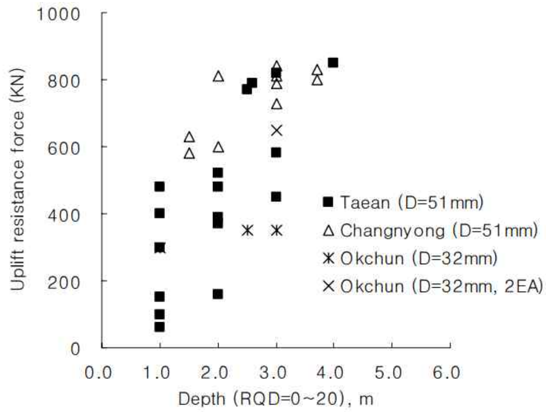
Test sites in the city of Okchun are classified as metamorphic limestone with a RQD of 0～52 % and a core recovery of 62～96 %. Anchor lengths were varied between 1 m and 6 m. The measured uplift capacity of anchors ranged from 300 kN to 350 kN for a single 32mm rebar and from 600 kN to 650 kN for double 32 mm rebars, indicating the yield of rebar prior to rock mass failure.

Uplift resistances for single anchors are summarized as a function of RQD in Figure 4(a) and as a function of anchor depth in Figure 4(b). For test results of Taean and Changnyong, the uplift resistances measured at the same anchor depth generally increase with RQD and these resistances for the rock masses of the same RQD also consistently show an increase with fixed depth of the anchor. For the test sites of Okchun, however, the uplift resistances simply reveal the strength of rebar regardless of the RQD or anchor depth, since the single 32 mm rebar was embedded at sufficient depth to avoid other modes of failure.

Based on the strain readings and the extent of surface cracks at moderately weathered rocks in Taean and Changnyong, the rock pull-up failure was estimated to be an inverted cone with the apex at half the embedded depth and having a contained angle of 90°.



(a)



(b)

Fig. 4 plift resistances vs. embedded length and RQD: (a); (b)

Anchors in poor quality rocks generally fail along the grout/rock interfaces when their depths are very shallow (a fixed length of less than 1.5 m). However, even in such poor rocks, we can induce a more favorable mode of rock pull-up failure by increasing the fixed length of the anchors. On the other hand, anchors in good quality rocks show rock pull-up failures with high uplift resistance even when they are embedded at a shallow depth.

3.3 Results of Group Anchor

Close examination of test No. 1 reveals that there were large cracks developed suddenly around the rebar at an applied load of 3.6 MN, and thereafter the uplift resistance increased gradually until it reached a maximum capacity of 4.6 MN. This measured maximum load of 4.6 MN is approximately equal to the load calculated by the ultimate strength methods (BS 8081, 1989; DIN 4125, 1990), provided that the effective free length is 50 % of the total anchor length, thus supporting our hypothesis of an inverted cone failure surface with its apex at the middle of the anchored depth.

Closely spaced anchors fail as a group due to interference of the adjoining failure surfaces of individual anchors, resulting in an individual anchor efficiency of less than 100 %. Thus, the overall capacity of group anchored foundations depends not only on the material properties of the anchor system but also on the spacing and depth of the anchors (Ismael, 1982; Littlejohn, 1992).

Group anchor tests of No. 1 and No. 2 in Okchun revealed the uplift capacity of about 4,000~4,600 kN. On the other hand, single anchor tests in the same sites show the average uplift capacity of about 700~800 kN. Based on such single and group anchor tests, we have derived a curve, as shown in Figure 5, representing the efficiency as a function of the ratio a/R, where a is the anchor spacing and R is half of the embedded anchor depth (l). When the ratio a/R is greater than 2, the individual anchors in a group have an efficiency of 100 %. However, it should not be extrapolated below an a/R ratio of 0.4 since group tests were not conducted below this ratio. For the purpose of practical applications, the efficiency (β) of an individual bar in group anchored foundations can be approximated by the following linear equation:

$$\beta = 0.375\left(\frac{a}{l}\right) + 0.625 \quad for\ 0.2 \le \frac{a}{l} \langle 1.0 \qquad (2\ a)$$

$$\beta = 1.0 \quad for\ \frac{a}{l} \ge 1.0 \qquad (2\ b)$$

where a is the anchor spacing and l is the anchor depth. Note that the above linear efficiency equation uses an anchor depth (l) instead of R on the basis of our inverted cone failure surface with its apex at the middle of anchor depth.

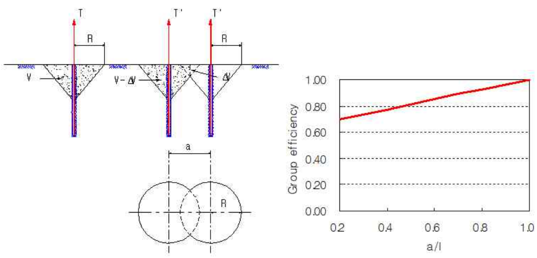


Fig. 5 roup effect of frictional anchor

4. Conclusions

A review of some recent full-scale and laboratory tests carried out for application in structural foundations was presented. From these test results, the uplift capacities and failure modes were evaluated on anchor foundations in various in situ rock masses in several regions in Korea. In particular, a number of group anchor tests demonstrated the practical applicability of rock anchor foundations.

It is believed that the proposed design procedure can be applied to similar anchored structures where pull-out tensile force is considered to be the dominant load.

Acknowledgments

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