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Installation and Use of Epoxy-Grouted Rock Anchors for Skyline Logging in Southeast Alaska

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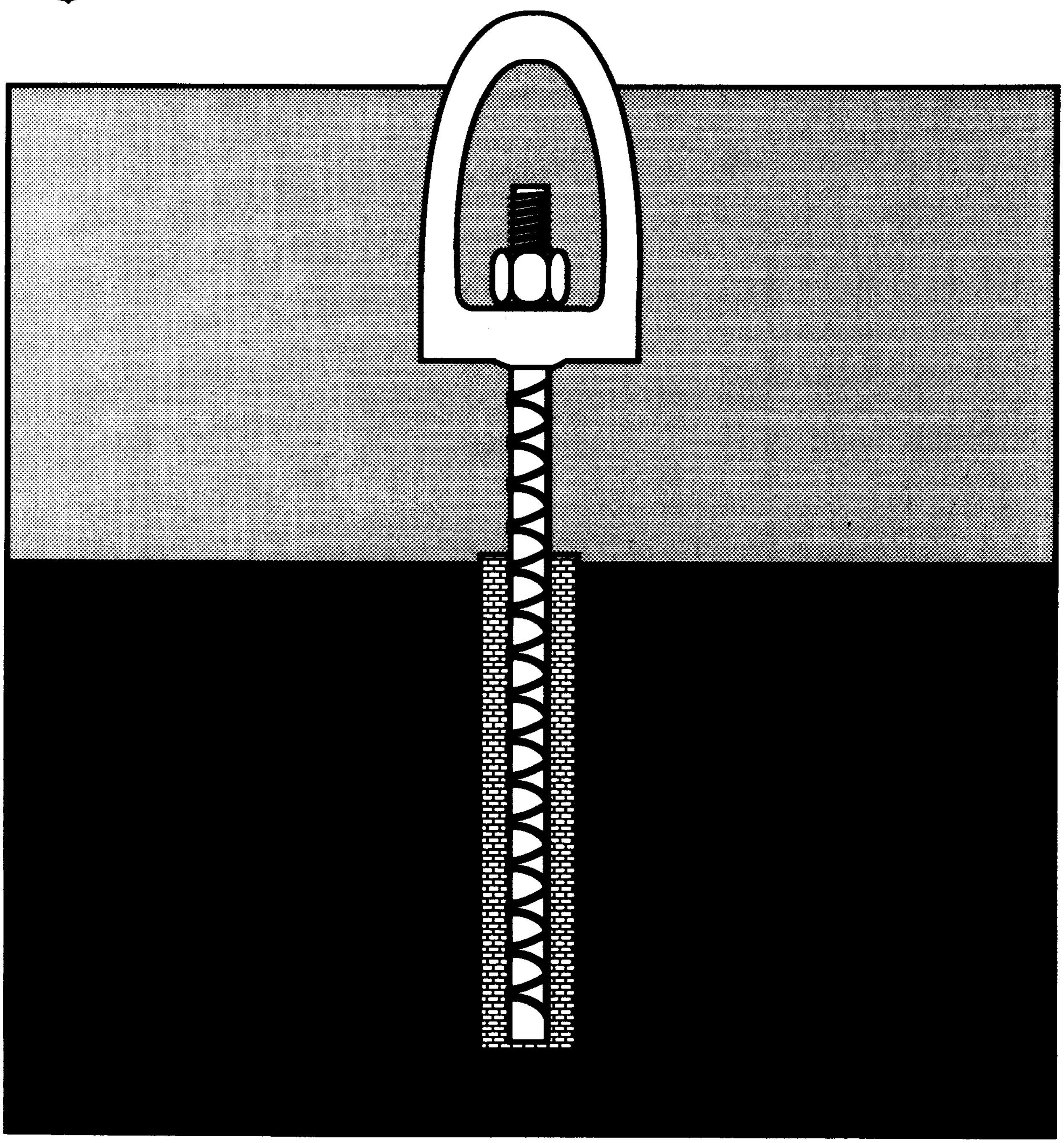

Introduction

Southeast Alaska has extensive areas of old-growth spruce (*Picea* sp.) and hemlock (*Tsuga* sp.) forest characterized by shallow root systems, a high water table, near-surface bedrock, and weak soils. These conditions result in a lack of reliable anchor points (such as trees and stumps) for attaching yarding tower guylines and tail-hold cables. Much of the terrain is too steep for most ground-based logging methods except for skylines, for which high-capacity anchor points are critical. Access to potential anchor sites is limited by the slopes and lack of roads. Helicopter and balloon logging is possible, but the high costs involved and limiting weather conditions make widespread application of these techniques impractical. This situation led the USDA Forest Service in Alaska, in conjunction with Oregon State University, College of Engineering, to conduct field tests on rockbolt anchors to learn if sufficient holding capacity for skyline anchorages can be developed, with reasonable anchor length, by using simple installation procedures at remote sites.

Field Test Results

The Forest Service conducted research on Wrangell Island to determine the load carrying capacity of epoxy-grouted rock anchors, which may serve in place of stump anchorages for guylines used in cable logging systems.[1] The elements of a typical grouted rock anchor are shown in figure 1. The information generated by this research provided guidance for designing anchors for field applications, such as those shown in figure 2. In the research, load tests were run on 44 full size anchors to determine their ultimate capacity (load required to pull out the anchor). In the tests, the load was always aligned with the axis of the anchor. Anchors were installed in two different rock types commonly found at logging sites in southeast Alaska. Each rock type displayed a substantial degree of weathering and fracturing and represented some of the poorest quality rock available for cable anchor purposes in southeast Alaska. The results of the tests provided quantitative information on the lowest available capacity for the wide range of rock conditions anticipated regionally. The research showed that ultimate capacity was mainly dependent on anchor length and rock hardness or strength.

Figure 3 presents the results of the research for the rock conditions tested. The poorest rock was a moderately to severely weathered phyllite schist. In appearance, the rock was medium to fine grained, fragmented, and platy. Breakage was along bedding planes ranging from 1/8 of an inch to 3 inches apart. In the field, rock with this strength will show a crater (depression created by pushing away rock from the point of impact) when its surface is struck with a sharp hammer blow. The other rock in the anchor tests was a weathered diorite gneiss. This rock was medium to coarse grained and salt and pepper gray in color. Joint and fracture surfaces were 2 to 3 inches apart and occasionally were separated by 1/2 inch to 2 inches of voids or soil fillings. In the field, rock with this strength will be dented (not cratered) by a hammer blow. The harder gneiss produced greater anchor capacity than the schist. The relations shown in figure 3 represent 95-percent prediction limits for ultimate anchor capacity for the anchors tested. This means that 95 percent of anchors installed in similar rock should have an ultimate capacity greater than that indicated in figure 3. Refer to the appendix for a presentation on the development of figure 3 from the test data.

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[1] Henry, V.T.; Cole, P.L.; Schroeder, W.L. 1989. Capacity of epoxy-grouted rock anchors for use in cable logging applications in southeast Alaska. Report. On file with: USDA Forest Service, Pacific Northwest Research Station, Forestry Sciences Laboratory, 2770 Sherwood Lane, Suite 2A, Juneau, AK 99801.

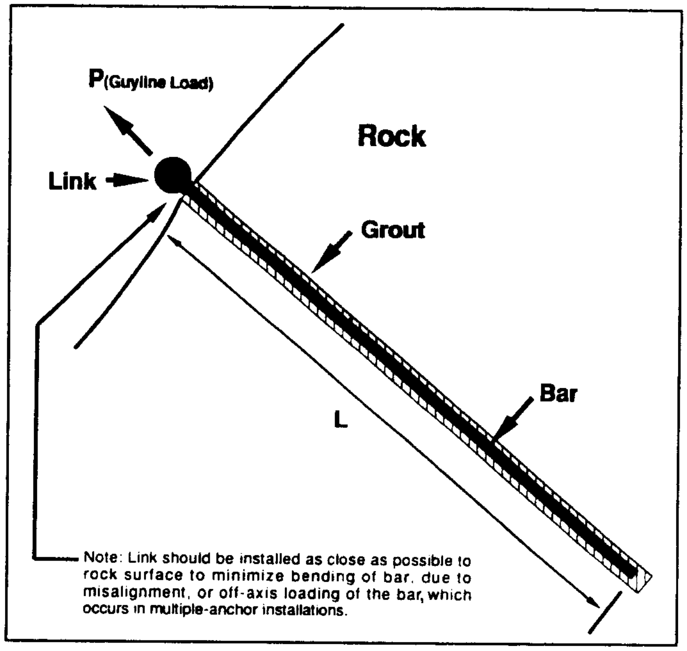


Figure 1—Elements of a grouted rock anchor. P is total guyline force or "load." A "link" is the mechanical connection between the guyline and the anchor. (One kip equals 1,000 pounds.)

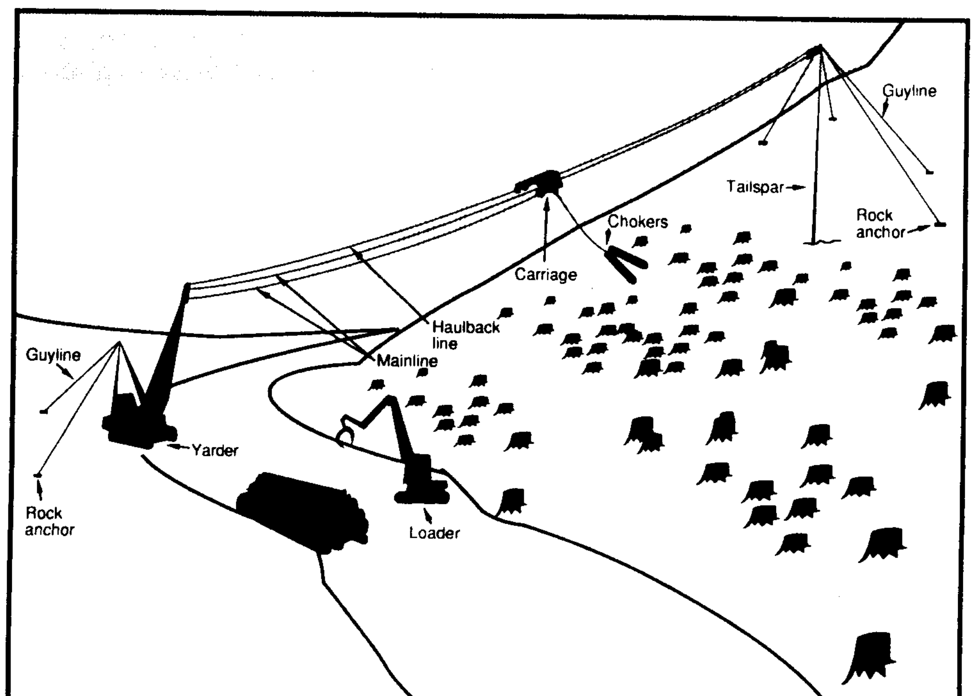


Figure 2—Typical rigging for skyline logging showing cable attachment locations where rock anchors can be effective.

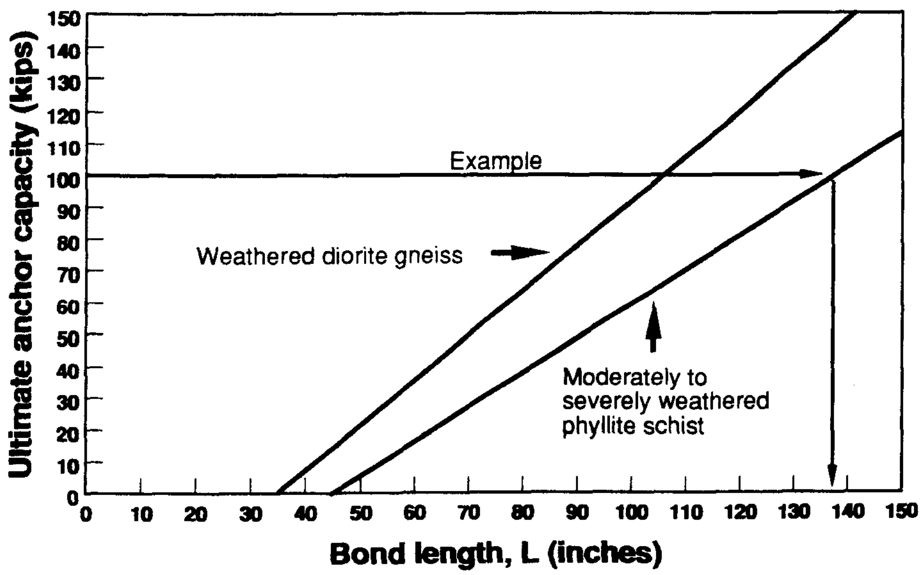


Figure 3—Ultimate anchor capacity as a function of grouted bond length in rock. This figure is based on results of anchor tests in rocks commonly found on Wrangell Island, southeast Alaska, and should be used with care at other localities where extensive weathering, fracturing, and discontinuous rock units are encountered.

Anchor Selection and Design Criteria

Design practice for anchors usually includes provisions for uncertainties by incorporating a factor of safety. One definition of factor of safety (FS) is,

$$FS = \frac{\text{ultimate anchor capacity}}{\text{actual anchor load}} . \qquad (1)$$

If the actual maximum load in a guyline, P, (see fig. 1) were equal to the ultimate anchor capacity, then from equation (1), the factor of safety would be one. In other words, the anchor would hold the actual maximum load and no more.

Figure 3 represents a 95-percent prediction limit for anchor capacity, based on the field test data. To use this data for design of anchors, unknown or variable factors, such as rock conditions, poor construction of anchors, poor installation procedure, and overloads, must be accounted for. A simple approach is to choose an anchor length that will hold more load than the actual maximum capacity from the testing program. In practice, this is done by using a factor of safety greater than one in equation (1). Where reliable but limited amounts of load test data are available, this factor of safety should be in the range of 1.5 to 2. The data presented in figure 3 and developed by Forest Service Research are reliable and were derived from a relatively large number of tests on some of the poorest rock in southeast Alaska. Because of this, and because figure 3 represents the lower bound of capacity (rather than an average), use of a factor of safety of 1.5 to 2 should be safe.

Anchor Length Selection From Rock Strength Estimates

The rock strength estimates developed by this research may be used to safely design anchors, even though rock conditions are not precisely known, because they were derived for some of the poorest rock types thought to exist in southeast Alaska. Anchors in other rock types usually will have greater actual holding capacity than that indicated here. When these data are used for anchor design, however, they should be used with caution and informed judgment, as local rock conditions could differ considerably from those in which the load tests were conducted. With these limitations, the results shown in figure 3 may be used to determine anchor length whenever the maximum actual load of a line to be anchored is known. The following steps show how figure 3 may be used to choose an anchor length for a 50-kip (50,000 pounds) guyline load, if the anchor is to be installed in phyllite schist.

1. Determine the maximum actual guyline load, P (see fig. 1), to be resisted by the anchor. This is called the design load.

2. Choose a factor of safety for anchor length selection. A minimum value of 2 is recommended. This means that the anchor length selected will have the capacity to carry twice the maximum guyline load.

3. Multiply the design load by the factor of safety to determine the required ultimate capacity of the anchor. For a design load of 50 kips and a factor of safety of 2, the required ultimate capacity is 2 • 50 = 100 kips, or 100,000 pounds.

4. Determine the required grouted length of the anchor. For this example, with a required ultimate capacity of 100 kips, the procedure is illustrated in figure 3. The required grouted length, L, is 138 inches (11.5 feet).

Anchor Length Selection Using Cable Size

This approach is based on the idea that the anchor should be able to resist a load at least as great as the breaking strength of the line to be anchored. It may be used if the size of the line to be anchored is known, and known to be adequate, but if the maximum actual load in the line is not known.

Table 1 provides breaking strength for two widely used wire rope types, and the length of anchor (from fig. 3) in moderately to severely weathered phyllite schist necessary to resist a load equal to the breaking strength. For instance, a 1-1/8-inch 6 • 19-IWRC (Independent Wire Rope Core) plow steel line should break at a minimum load of 98,200 pounds. An anchor length of 143 inches will resist pull-out under an equivalent load. The example of anchor length selection using rock strength estimates shows that an anchor length, L, of about 138 inches is required to hold as much load as this line can exert. In other words, the two approaches give similar results for the same ultimate capacity. Suppliers of wire rope normally recommend a factor of safety (breaking strength divided by actual load) of 5 for their product, but the logging industry commonly uses a factor of safety of 3. Thus, the line and the anchor in this example should actually be loaded to no more than about 20,000 to 33,000 pounds. This means that the factor of safety of the anchor from equation (1) will be between 5 and 3, or the same as that used for the line. If this approach is used, skyline systems that load cable guys to near breaking strength will have anchors operating closer to failure. If factors of safety of 3 to 5 are used in cable selection, this approach will result in longer anchors than would be selected by using figure 3 and the procedure described in the previous section.

Anchor Bars and Connections

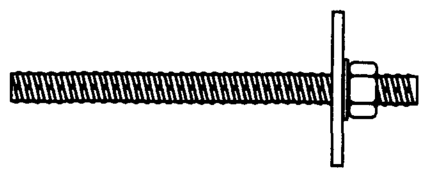
Threaded bar is usually preferable for making anchor bars (see fig. 4) because it is easier to connect to a cable than is regular deformed reinforcing steel. Use of a threaded bar allows the connection to be made with a nut and D-ring (see fig. 4C), which is simple to do in the field. An unthreaded bar requires a welded connection between the clevis and the bar that will develop the full required capacity, and it is difficult to fit and weld such connections in the field.

The term "yield strength" is used in the discussion that follows to describe the capacity of threaded bars. Yield strength is the load that a bar will carry at the point of first

Table 1—Nominal breaking strength of 6 • 19 IWRC wire rope and corresponding anchor length in phyllite schist[a]

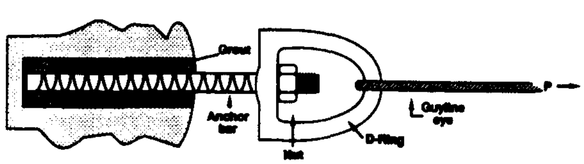
Rope diameter	Plow steel breaking strength	Rock anchor length	Improved plow steel breaking strength	Rock anchor length
Inches	*Pounds*	*Inches*	*Pounds*	*Inches*
1/2	20,000	65	23,000	68
5/8	31,200	76	35,800	81
3/4	44,400	90	51,200	96
7/8	60,200	105	69,200	114
1	78,200	123	89,800	135
1-1/8	98,200	143	113,000	158
1-1/4	120,800	166	138,800	184
1-3/8	145,200	190	167,000	212
1-1/2	172,000	217	197,800	243
1-5/8	200,000	245	230,000	275
1-3/4	232,000	277	266,000	311

[a] Breaking strength per Macwhyte Wire Rope Co. (1984).



A. Threaded bar with upset threads

B. Threaded bar with machined threads



C. Example connection to threaded bar

Figure 4—Examples of threaded bars used for anchors: (A) threaded bar with upset threads (Dywidag Systems International 1986); (B) threaded bar with machine threads (Williams Form Engineering Corp. 1988); and (C) common D-ring connection or "link" between guyline and threaded bar.

yielding. Below this load, it is elastic (returns to original length upon unloading). At greater loads it will deform permanently. The ultimate strength of anchor bars, or load at which the bar actually breaks, is greater than the yield strength.

Information included in this document on anchor bar strength is for commonly used ordinary grade steel. Steels with higher and lower strengths are available; the user should verify strength before design and installation. Information on alternate steels and connections can be obtained from manufacturers and suppliers.

Threaded bar with upset threads—Table 2 gives yield strengths for a range of commonly available threadbar sizes from one manufacturer. An illustration is provided in figure 4A. In this instance, the threads are formed by a process that does not reduce the nominal bar diameter. A bar should be selected that has a yield strength equal to or greater than the breaking strength of the line to be anchored. For instance, table 1 shows that a 1-1/8-inch 6 • 19 -IWRC plow steel line has a breaking strength of 98,200 pounds. From table 3, this would require a no.14 grade 60 threadbar to form a suitable anchor. In this case, the bar, with a yield strength of 135,000 pounds, will have much greater capacity than the line.

Threaded bar with rolled or machined threads—Threads for a retaining nut may be provided by a manufacturer or be cut in bar stock in a machine shop. In both cases, the cutting of the threads reduces the effective size of the bar and, therefore, its capacity to carry a load. Table 3 shows capacities of common threaded reinforcing bar furnished by one manufacturer. The bar is illustrated in figure 4B. The yield strength of a no.14 bar from table 3 may be compared with yield strength of the no.14 bar (upset threads) in the previous example.

Unthreaded bar—It is common, in the logging industry, to use regular grade 60 reinforcing steel for anchor bars. It sometimes may be useful to select bars without threads, as the latter will have greater capacity then bars of the same nominal size but with rolled or machine threads. Yield strengths for ordinary unthreaded bars are given in table 4.

Guyline Connections

Guylines may be connected to anchor bars in several ways. A "link" is illustrated schematically in figure 1. One type of link, involving a D-ring and nut, is shown in figure 4C. There are many other possible ways of attaching a line to a threaded bar; for instance, one manufacturer of threaded rebar (table 3) supplies threaded eyebolts, which make a suitable attachment to substitute for the nut and D-ring shown in figure 4C. To design anchorages, the load carrying capacity of these linkages must be determined from manufacturers or suppliers, so that the link has a capacity at least as great as the guyline and the anchor bar. Connection of a guyline to unthreaded bar requires a welded connection and link. We recommend that the link and weld not be designed and fabricated in the field. A standard design may be adopted and shop manufactured. The design should be prepared by a registered professional engineer to suit the required capacity of the line and anchor.

Installation of Anchors

Based on experience and knowledge gained during performance of this study, we recommend the following steps for installation of rock anchors:

1. Select a site for each anchor where the best quality rock is available. Try to avoid deeply weathered or intensely jointed or broken rock. The anchor should be embedded in rock for the entire length indicated from figure 3, and any length needed for

Table 2—Common ASTM A615 (grade 60) threadbar sizes for anchor rods

Rebar size designation	Nominal diameter	Yield strength
Number	*Inches*	*Pounds*
6	0.750	26,400
7	.875	36,000
8	1.000	47,400
9	1.128	60,000
10	1.270	76,200
11	1.410	93,600
14	1.693	135,000
18	2.257	240,000

Source: Dywidag Systems International 1986.

Table 3—Common ASTM A615 (grade 60) threaded rebar sizes for anchor rods

Rebar size designation	Thread size	Minimum yield strength through threads
Number	*Inches*	*Pounds*
4	1/2	8,500
5	5/8	13,000
6	3/4	20,000
7	7/8	27,000
8	1	36,000
9	1-1/8	45,000
10	1-1/4	58,000
11	1-3/8	73,000
14	1-3/4	114,000
18	2	159,000

Source: Williams Form Engineering Corp. 1989.

Table 4—Unthreaded ASTM A615 (grade 60) reinforcing bars

Rebar size designation	Nominal diameter	Yield strength
Number	*Inches*	*Pounds*
6	0.750	26,400
7	.875	36,000
8	1.000	47,400
9	1.128	60,000
10	1.270	76,200
11	1.410	93,600
14	1.693	135,000
18	2.257	240,000

connectors should be added. The rock strength selected for design should extend a distance at least equal to the anchor length in all directions from the anchor location. Care should be taken here, because the lateral extent of the rock may be masked by overburden soils. Rock depth will be determined when the hole for the anchor is drilled.

2. Drill the smallest size hole in the rock that will accept the anchor bar. The axis of the hole must align with the direction of the cable guy to be anchored.

3. Grout the anchor bar in the hole in accordance with the grout manufacturer's recommendations.

Service

Inspect all anchors in the installation daily during yarding operations to determine that no movement has occurred. Figure 5 shows some examples of potential failure modes and reduced anchor capacity.

Off-Axis Loading and Multiple Anchors

Off-Axis Loading

The research this document is based on did not address the case where the guyline and anchor bar are not aligned. Some of the test data indicated, however, that if there was misalignment, application of load would cause bending and yielding of the bar above the grouted section, until alignment of the bar and the guy was achieved. The grouted section would, of course, remain misaligned. Analysis further suggested that in this case, for an otherwise properly designed anchor system, the weak location would be where the bar bends. The bar would yield at a lower load than its yield strength if it were axially loaded. For a given line load, an off-axis anchor therefore will require a larger bar than one that is axially loaded.

Minor misalignments can be tolerated in these systems if the anchor bar is selected conservatively (larger than required for axial load). It is possible to calculate the required oversize if the line load and link location on the bar relative to the grouted anchor are known. It is beyond the scope of this document to illustrate these calculations here. Two general rules may be given, however:

1. The anchor bar and the guyline to be anchored always should be aligned as closely as possible.

2. The linkage should be placed as closely as possible to the rock surface (see fig. 1) to minimize bending of the anchor bar.

Multiple Anchors

It may not be possible, for large guyline loads, to develop a single anchor of adequate capacity. Multiple anchors are one solution to this problem. A multiple anchor system employing blocks to equalize line tensions is illustrated schematically in figure 6. The guyline force, P, is divided into tether forces P1 through P4, each of which is provided with its own anchor. It is difficult to align the tethers and their anchors in an actual installation. Thus, multiple anchor installations typically result in the same problems as the misaligned single anchors discussed above. It therefore is recommended that a registered professional engineer be retained to design multiple anchorage systems.

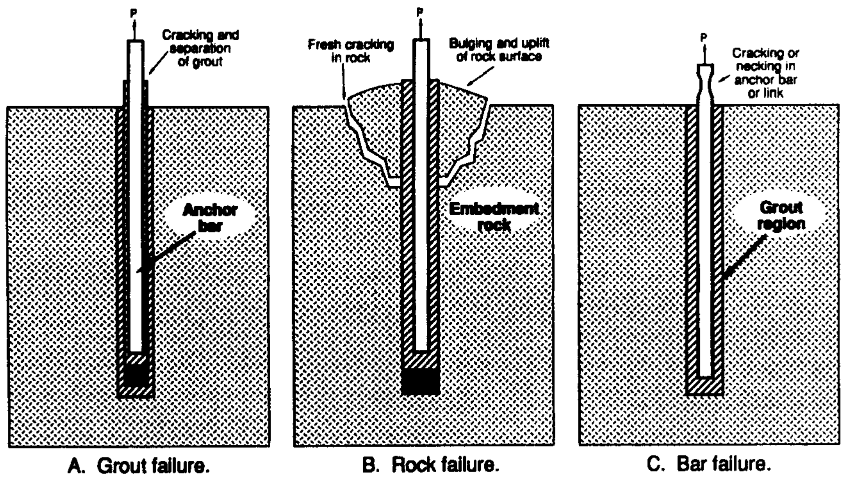


Figure 5—Three common failure modes for rock anchors: (A) failure of the grout; (B) failure of the rock mass; and (C) failure of the anchor bar.

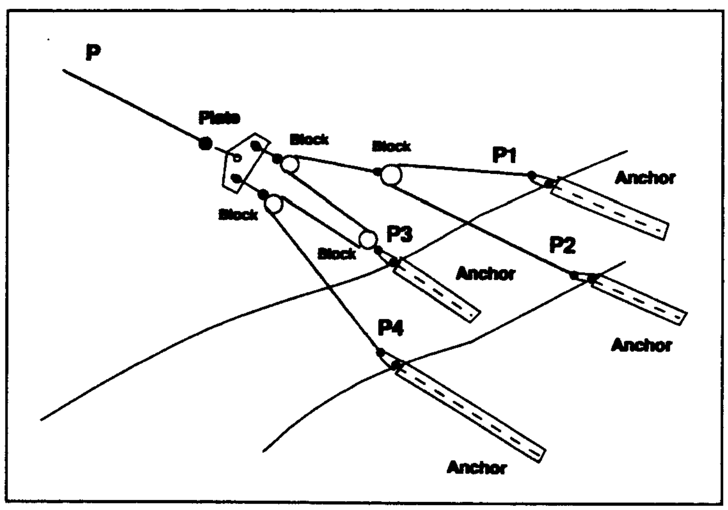


Figure 6—Example of a multiple anchor system for large guyline loads employing blocks to equalize line tensions. P is total guyline force. P1-P4 are tether forces.

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Appendix

Analysis for Design

Grouted rock anchors fail by one mode, or a combination of the following modes (see fig. 5):

1. Failure of the grout itself or failure at the contact between the grout and rock or between the grout and anchor bar.

2. Failure of the rock mass.

3. Failure of the anchor bar.

Anchor design must take into account each of the above possibilities and evaluate each part of the anchor system (rock, grout, bar, and connections) to reveal the weakest link in that system. The properties of the anchor bar, connections, and grout are usually well defined, but those of the rock are not.

The anchor pull tests performed in the field studies exhibited failures in the rock mass or at the contact between grout and rock. The rock (phyllite/schist and diorite gneiss) therefore was considered the weakest component of the anchor system. Linear regression analysis (least squares fit) was used to establish the linear relation between ultimate anchor capacity and bonded anchor length for the rock types tested. For design purposes, the lower 95-percent prediction limits were determined for each rock type (phyllite/schist and diorite gneiss) and are presented in figure 7 as prediction lines A and B along with the actual test results. Lines A and B in figure 7 correspond to the limits shown in figure 3. The 95-percent prediction limit is a statistical estimation in the form of a line or curve that bounds 95 percent of a set of data (Rice 1988).

The lines on figure 7 are the lower bound of 95 percent of the data for ultimate anchor capacity for any given bond length. The prediction lines can be used for design purposes, but caution should be used when the quality of the rock is questionable (low intact strength and intense weathering and jointing). Prediction lines A and B in figure 7 can be used on most phyllite/schist and diorite gneiss in southeast Alaska with a Unified Rock Classification (Williamson and Kuhn 1987) of at least D for intact strength and D for degree of weathering. Figure 7 also shows that the prediction relations are reliable only for anchors with bonded lengths greater than 45 inches for anchors in phyllite/schist and greater than 37 inches for anchors in diorite gneiss. Anchors should not be installed with bonded lengths shorter than these values.

Use of the 95-percent prediction lines in figure 7 may not be appropriate for field installations without application of an additional factor of safety. This may be accomplished simply by selecting a required ultimate anchor capacity that is greater then the actual anticipated load. For instance, for a 50-kip required capacity in gneiss, figure 7 suggests a minimum length of 70 inches. A factor of safety of 1.5 would be achieved by specifying a bond length of 88 inches (75 kips), and a factor of safety of 2.0 would be achieved by a bond length of 103 inches (100 kips).

Ultimate anchor bond strength is another way to present anchor capacity. From figure 7 the ultimate bond strength of the phyllite/schist and diorite gneiss are 150 psi (pounds per square inch) and 209 psi (pounds per square inch), respectively, as determined by equation (1). These values agree well with bond strength values for metamorphic rocks given by Littlejohn and Bruce (1975):

$$\tau = \left(\frac{P}{L}\right)\left(\frac{1}{\pi d}\right), \quad (2)$$

where

P = ultimate anchor capacity, from figure 7;

d = nominal hole diameter ≅ 2.5 inches;

L = bonded anchor length, from figure 7; and

τ = bond strength of rock/grout/bar system.

By using this approach for design, an appropriate factor of safety should be applied to the ultimate bond strength values derived from equation (1) and the test data, and only bond lengths in excess of 45 inches and 37 inches should be used to calculate capacity of anchors installed in phyllite/schist and diorite gneiss, respectively.

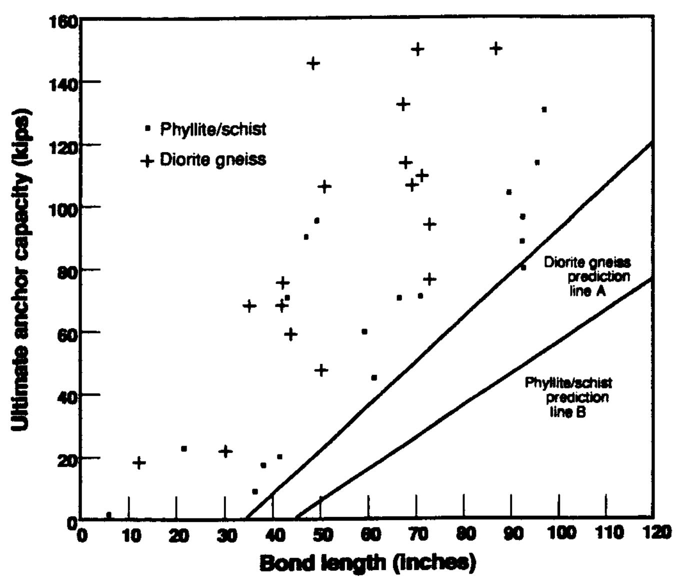

Figure 7—A plot of the ultimate anchor capacity versus bonded length for 34 test sites in phyllite/schist and diorite gneiss on Wrangell Island, southeast Alaska. Lines A and B were obtained by linear regression analysis and represent the lower 95-percent prediction limits for rock types tested. Anchor capacity is extrapolated above 110 kips.