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ANCHORING IN ROCK

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by

J. F. Wadsworth

ABSTRACT

This report is a study of the state-of-the-art of anchoring in rock. Included are summaries of the performance of various propellant-actuated embedment anchors, rock bolts, and grouted anchors. Sections are included describing the characteristics and properties of coral and basalt seafloors, and the phenomena associated with rock penetration.

Design guidelines for various seafloors are given based upon existing knowledge and experience. In coral or soft rock seafloors, the embedment anchor should be used; in hard, competent rock, rock bolts should be used in shallow water, drilled-in anchors should be used for larger anchors to intermediate depths, and deadweight anchors are the most likely choice in deep water.

There is a need for a more viable anchor for hard rock seafloors. Land tests of the embedment anchor in hard rock are recommended to evaluate it as a rock anchor. Interim projectile selection guidelines are given.

General guidelines based on operational requirements, available support, and seafloor conditions should be developed in handbook form to aid the designer in choosing an anchor for rock seafloors.

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INTRODUCTION

The propellant-actuated embedment anchors are nearing true operational usefulness as evidenced by their use for temporary and permanent moorings in the Santa Barbara Channel, Midway Island, and Diego Garcia. Thus far, most of the anchors installed for working purposes have been in coral seafloors, but it is anticipated that future users will require embedment in even harder seafloor materials, such as basalt.

The mechanisms of dynamic penetration and the subsequent holding capacities of anchor projectiles in rock is not well understood. This report is the result of a literature search on the subject of dynamic penetration of rock and the holding capacities of rock anchors.

METHODS OF ANCHORING IN ROCK

Embedment Anchors

Embedment anchors operate by generating high pressure in a gun barrel to drive an anchor projectile into the seafloor. To date, most of the embedment anchors developed make use of a solid propellant charge to supply the needed pressure. Actual tests of propellant-actuated or "explosive" anchors in coral and basalt seafloors have been conducted by the U. S. Army and Navy. The results of these tests are summarized in Table 1.

MERDC XM200 and XM50 Anchor. The earliest coral tests were conducted by the U. S. Army Mobility Engineering Research and Development Center (MERDC) with the XM50 and XM200 anchors (Figures 1 and 2) at Key West, Florida, during 1963 (Mayo, 1973). The anchor projectiles for these anchors are flat steel plates. A short ogive nose section at the center of the leading edge is used to initiate penetration. The downhaul connection is to the center of the plate. These anchors have keying flaps at the trailing edge to initiate keying into a horizontal position. This keying will occur in sediments; in rock the flaps will probably help to wedge the fluke in the penetration hole.

A total of nine tests were conducted in coral. Of these, two tests were performed with the XM50 anchor, and both showed good penetration and resisted loads higher than the 50,000-pound rated holding capacity. The XM200, rated at 200,000-pound holding capacity, was more extensively tested with less satisfactory results. Although penetration depths varied within a fairly close range, the holding capacities varied widely, from 60,000 pounds to 220,000 pounds. Indeed, the highest load sustained was by the anchor with least penetration and lowest projectile velocity. However, there is no information on the material properties of the coral at each site, so conclusions on the relationship of velocity and penetration to holding capacity would not be meaningful.

SUPSALV Anchor. The SUPSALV anchor was designed and built by Aerojet-General and then developed by the Civil Engineering Laboratory (CEL) under the sponsorship of the U. S. Navy Supervisor of Salvage

Table 1. Embedment Anchor Tests in Coral and Basalt Seafloors

Test No.	Date	Test Site	Seafloor	Water Depth (ft)	Projectile Velocity (ft/sec)	Projectile Weight (lbs)	Energy (10^6ft-lbs)	Penetration (ft)	Holding Capacity (kips)	Remarks
MERDC XM50										
3	5/21/63	Key West	Coral	40	-*	400	-	17	65	
4	5/21/63			42	-	400		22	80	
MERDC XM200										
1	5/07/63	Key West	Coral	45	452	1250	127.7	17	145	Keying flap failure, load line at 11° angle
2	5/06/63			46	410		105.1	18	60	Keying flap failure, load line at 11° angle
3	5/10/63			47	452		127.7	19	100	Keying flap failure, load line at 11° angle
4	5/13/63		Mud/Coral	46	318		63.2	21	215	Keying flap failure, load line at 45° angle
5	5/16/63			38	410		105.1	28	115	Keying flap failure, load line at 11° angle
6	5/17/63			39	452		127.7	31	125	Keying flap failure, load line at 11° angle
7	5/23/63		Coral	45	318		63.2	10	220	Keying flap failure, load line perpendicular
SUPSALV Anchor										
12	5/28/68	Key West	Fringe Reef Coral σ_c=1500-2500 psi	52	-	-	-	8	68	SUPSALV anchors were loaded horizontally unless penetration was considered insufficient
13	7/08/68			50	-	2000	-	7	120	
14	7/09/68		Surface Texture Rough to Moderate Basalt	50	-	2000	-	11	128-136	Horizontal pull, finally extracted by 10 hours of vertical pull with 30-60 kips
15	12/12/68	Anacapa Island		55	-	2000	-	3.6	45-50	Vertical pull
16	12/13/68		σ_c=16,400 psi Schmidt N=32 γ=163 pcf	52	-	2000	-	4.2	64	Vertical pull Vertical pull
17	12/15/68			56	-	1650	-	5.1	168	
22	4/02/68	S.Coast, Oahu	Coral σ_c=1500-2500 psi Smooth Surface, Scattered Sand	50	-	2000	-	9	65	Anchors 23 and 24 used as reaction for pull on 25, cable broke, while anchors remained embedded
23	4/03/68			60	-	2000	-	11	75+	
24	4/04/68			48	-	2000	-	11.4	75+	
25	4/04/68			48	-	2000	-	11.4	150+	
28	8/23/69	Cobb Seamount	Basalt σ_c=20,000 psi	120	-	1650	-	-		
CEL 100K Anchor										
P1	5/22/75	Diego Garcia	Lagoon Coral Heads and Coral Sand	70-50	425	1500	135.5	35	110+	All anchors proof tested to 110 K without failure in direct uplift
P2	5/23/75				410		126.1	26	110+	
P3	5/24/75				390		114.1	35	110+	
P4	5/24/75				390		114.1	35	110+	
P5	5/25/75				380		108.3	31	110+	
P6	5/25/75				380		108.3	35	110+	
P7	5/26/75				-		-	30	110+	
P8	5/27/75				395		117.0	30	150+	Proof tested to 150 K with no failure
P9	5/27/75				390		114.1	35	110+	110K proof load test
P10	5/27/75				-		-	31	110+	

*Dashes signify unknown values

Table 1. Embedment Anchor Tests in Coral and Basalt Seafloors (Continued)

Test No.	Date	Test Site	Seafloor	Water Depth (ft)	Projectile Velocity (ft/sec)	Projectile Weight (lbs)	Energy (10^6 ft-lbs)	Penetration (ft)	Holding Capacity (kips)	Remarks
CEL 100K Anchor (cont'd)										
T1	6/02/75	Diego Garcia	Bare Coral	70-50	385	1500	111.2	35	110+	
T2	6/02/75	↓	↓	↓	315	1900	94.3	28	165+	Surge loaded to 165 K with no failure
T3	6/02/75	↓	↓	↓	385	1500	111.2	28	110+	110 K proof load test
T4	6/03/75	↓	↓	↓	335	1900	106.6	28	110+	↓
T5	6/03/75	↓	↓	↓	335	↓	106.6	32	110+	↓
T6	6/03/75	↓	↓	↓	335	↓	106.6	30	110+	↓
T7	6/04/75	↓	↓	↓	330	↓	103.5	29	110+	↓
T8	6/04/75	↓	↓	↓	325	↓	100.3	30	110+	↓
CEL 20K Anchor										
11	3/26/73	Anacapa Island	Weathered Basalt	67	405	300	24.6	2	107	Except as noted, all loads in direct uplift
12	3/26/731	↓	↓	67	405	↓	↓	3	20	Embedded on steep slope
28	12/06/74	↓	Cemented Basalt	90	405	↓	↓	-	-	Projectile fired into sand pocket, no results
29	12/06/74	↓	Cobbles, Rubble	90	405	↓	↓	1	13?	Embedded in cemented rubble
43	9/15/75	Midway Island	Hard Coral	110	↓	↓	↓	3	30+	Estimated loads, modified sand projectile used loading line approximately 7?° from perpendicular
44	9/15/75	↓	Soft Coral	↓	↓	↓	↓	14	50	↓
45	9/16/75	↓	Soft Coral	↓	↓	↓	↓	10	30+	↓
CEL 10K Anchor										
1	9/14/75	Midway Island	Soft Coral	59	392	148	11.4	9	30	Estimated failure load during heavy weather, 70° load
2	9/14/75	↓	↓	59	380	173	12.5	14	30+	Estimated load, load line ~70° from perpendicular
3	9/14/75	↓	Hard Coral	110	392	148	11.4	2	None	Fluke damaged

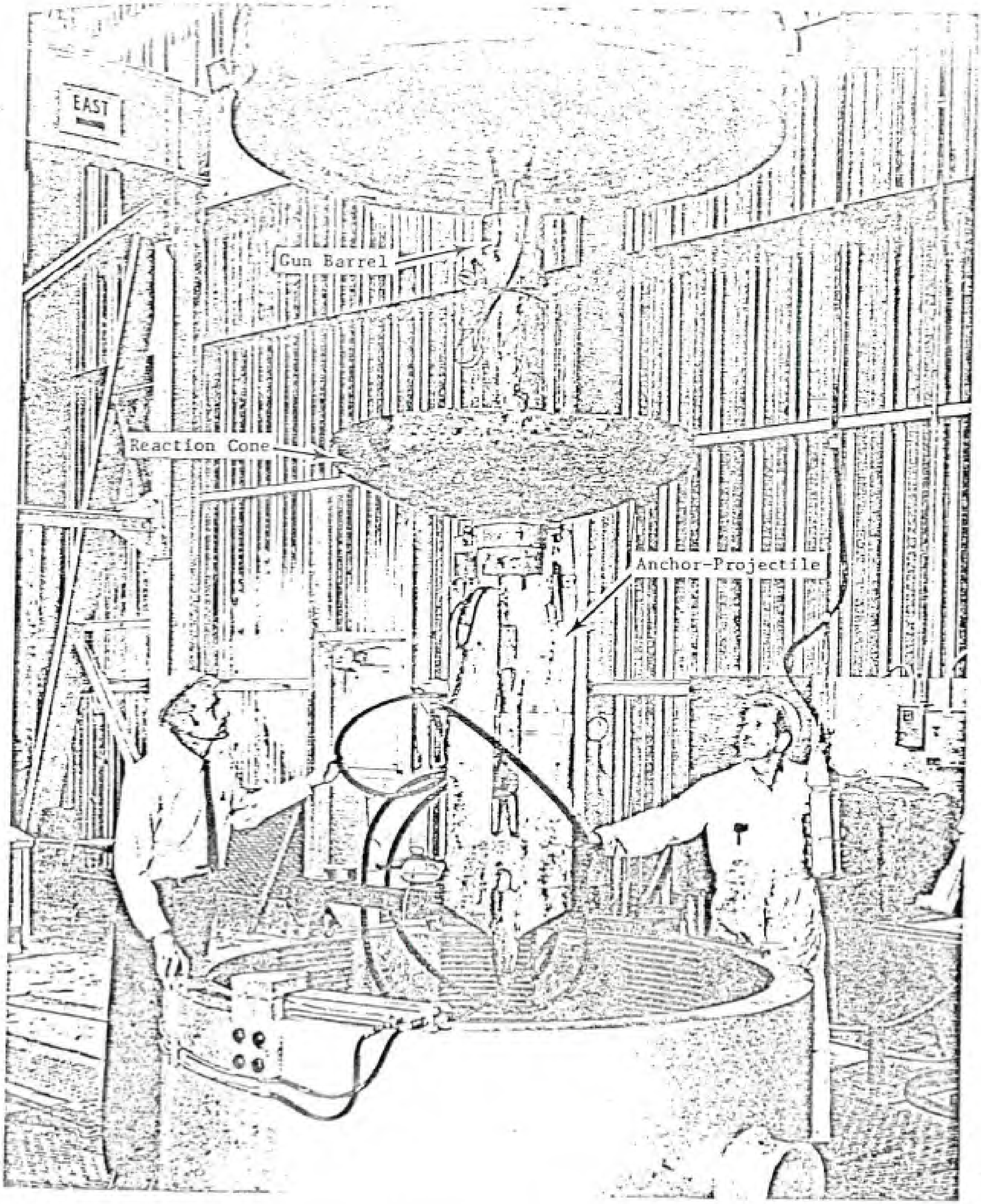


Figure 1. MERDC XM50 explosive anchor

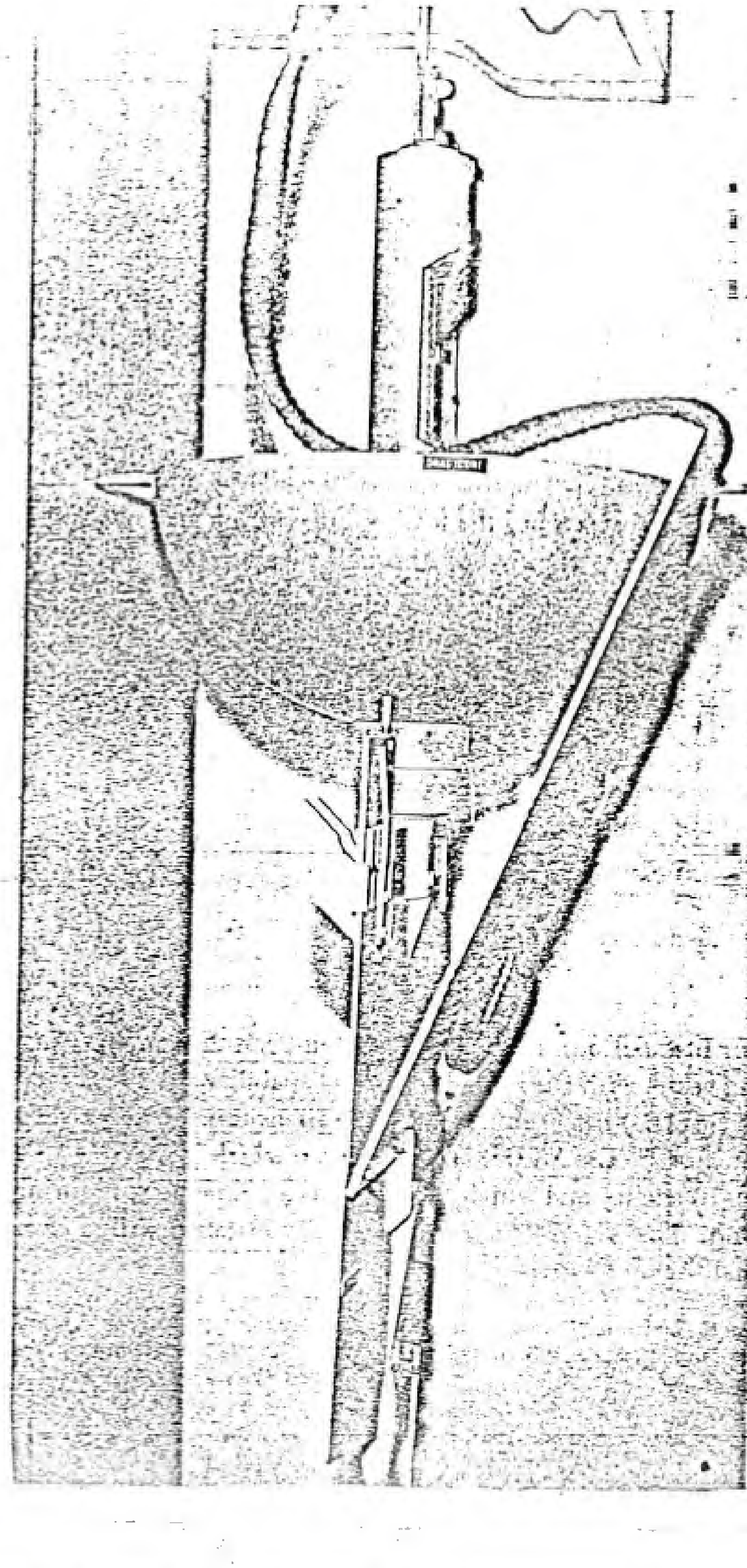


Figure 2. Cutaway view of the MERDC XM200 explosive anchor (from Mayo, 1973).

(SUPSALV). It was the first embedment anchor to have an anchor projectile specifically designed for use in coral. This anchor was tested during 1968-69 in coral at Key West, Florida, and Oahu, Hawaii; and in basalt at Anacapa Island, California, and the Cobb Seamount off the coast of Washington.

Modifications were made to the projectile during these tests to improve penetration and holding capacity. The changes in projectile shape which improved holding capacity included enlarging the coral projectile and adding serrations to the plate edges (Figure 3). As both modifications were carried out simultaneously, the influence of each modification upon holding capacity cannot be separated. The revised coral projectile was used for the first of two basalt tests, but was modified for the third test by cutting the plates to a more pointed arrowhead shape (Figure 4). This configuration tripled the holding capacity; however, the charge weight was also increased. The increased capacity could be due as much to the increase in projectile velocity as to the change in shape. The same projectile was later tested in basalt at Cobb Seamount, but it was damaged during penetration and failed structurally at a low load.

The results from the coral tests indicate that increased embedded side area and/or serrated edges increase the holding capacity of an embedment anchor. In the basalt tests, deeper penetration seemed to result in higher holding capacities, and increased penetration occurred when the projectile was tapered. It is interesting to note in Table 1 that tests 16 and 17 had almost the same side area embedded, yet test 17, with the sharply tapered nose, held almost three times the load of the coral projectile used in test 16. However, the drawback of using a sharply pointed nose was shown by the Cobb Seamount test. It appears that the nose should be pointed to maximize holding capacity, but this will require a very strong material for the anchor projectile.

<u>CEL 100K Anchor</u>. This anchor, formerly called the SUPSALV anchor, was renamed after modifications to conform to the nomenclature of the other Navy embedment anchors, the CEL 20K and CEL 10K anchors. The 100K gives the nominal holding capacity of the anchor, 100,000 pounds.

This anchor was used at Diego Garcia to emplace permanent and temporary tanker moorings (Figure 5). The flukes used were the quick-keying type developed for anchoring in sediments. This type of fluke was used because the anchor sites were primarily soft coral overlain with coral sand. It was thought that this design would allow the anchor to develop its full holding capacity in either the sand or coral. After emplacement, each anchor was proof tested to 110,000 pounds. On some anchors, movement was noted, indicating keying, while others did not move at all, indicating a solid embedment.

The Diego Garcia results show that a sediment type of fluke is desirable in soft coral seafloors. Flukes like the SUPSALV coral projectile depend upon frictional resistance to direct uplift because its axis is in vertical orientation and the load is applied along the axis. If the material is weak and brittle, an anchor that keys under load to a horizontal position provides greater bearing resistance. If the material is too hard to permit keying, there is still plenty of area to

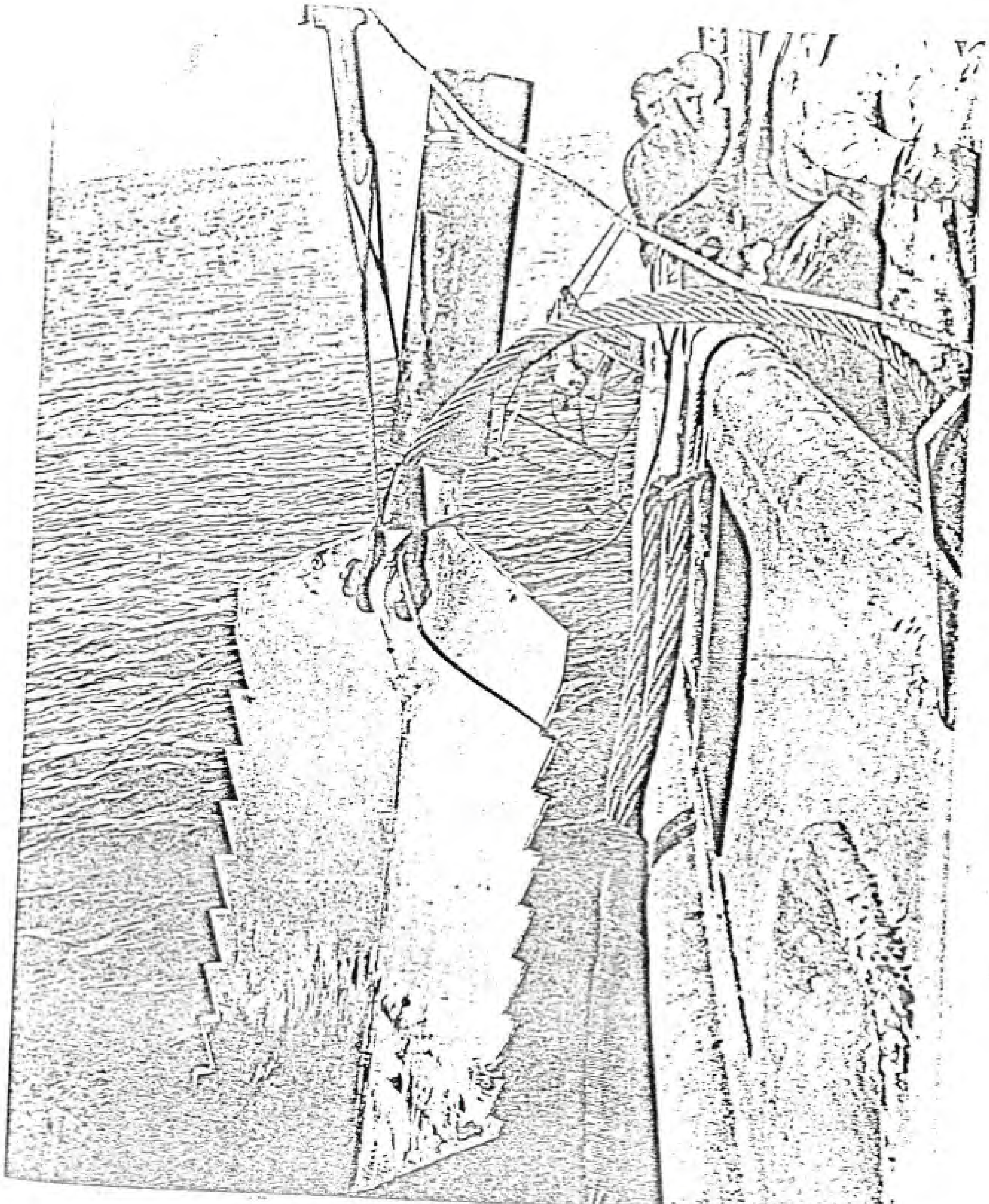


Figure 3. SUPSALV coral anchor projectile with piston.

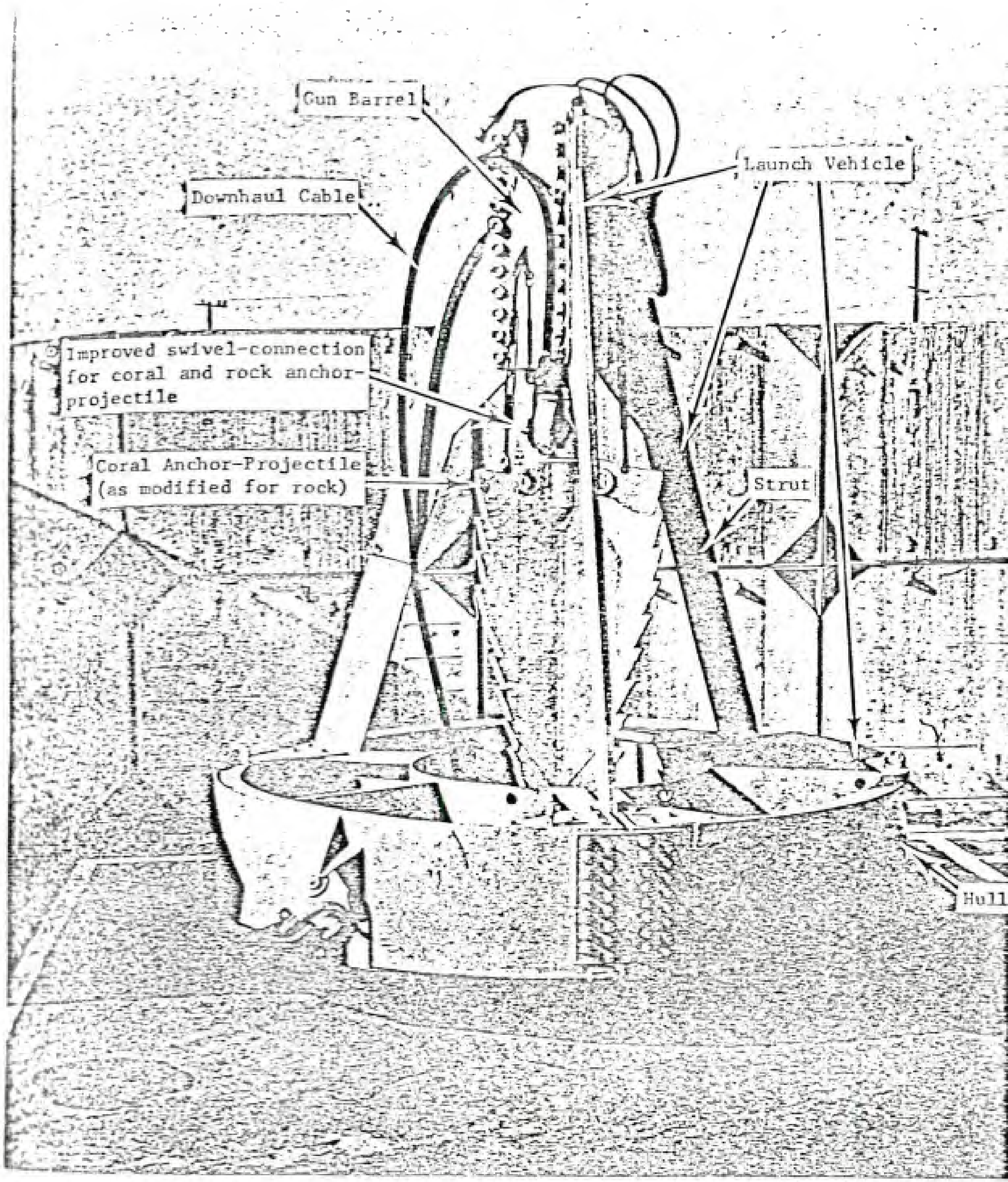


Figure 4. SUPSALV anchor assembly with coral projectile modified for rock.

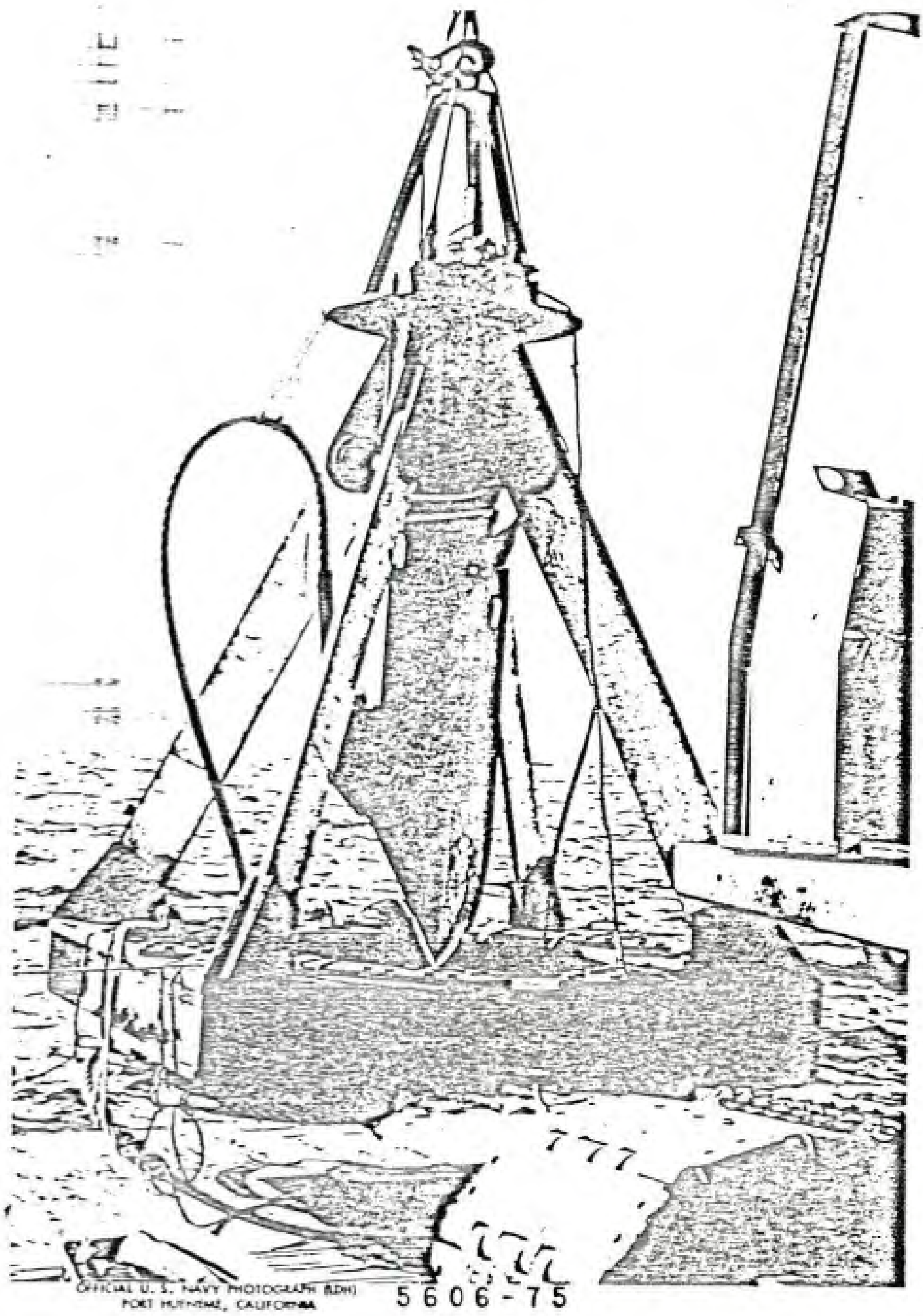


Figure 5 CEL 100 K Anchor

develop frictional pullout resistance. There is a sacrifice in penetration depth when using the sediment fluke in coral. This is a concern in harder seafloors, but at Diego Garcia adequate penetration (~30 feet) in the soft coral was achieved with the sediment fluke.

CEL 20K Anchor. The rock projectile designed for use with the CEL 20K anchor (Figure 6) incorporates some of the features deemed desirable from earlier experiences. The body is a solid high-strength alloy bar with a conical nose blunted to a 90-degree (1.6 radians) cone at the tip to prevent hooking or bending. Three alloy steel plates are welded to the bar to give a dart-shaped projectile. A three-fin fluke is more desirable than other shapes because it affords increased moment resistance to randomly directed loads. The downhaul cable is eccentrically connected to one of the fins and protective fairings are added to prevent damage to the connecting socket.

This anchor has been tested only four times with the rock projectile. The first two tests were in weathered basalt, the first test holding 107,000 pounds and the second 20,000 pounds. The second test was complicated by penetration into the side of a slope. About 4 feet of rock was spalled off the face of the slope before the anchor finally embedded 3 feet into the rock.

The next two tests were not successful. The bottom for these tests was composed of cemented cobbles and rubble with large sand pockets. The first projectile broke the downhaul cable when fired. It is thought that the projectile hit a sand pocket and "over penetrated". When it ran out of faked line the downhaul was snapped. The second test was partially successful. The projectile penetrated, but the piston unseated before full penetration, hit the back of a fin, and sheared off the fin's corner. The holding capacity was seriously reduced.

This projectile has never been used in coral.

The CEL 20K anchor was used at Midway Island to place three temporary anchors. Modified sand anchors of the quick-keying variety were used. Penetration ranged from 3 feet in very hard coral to 10-14 feet in softer coral. The flukes were not load tested; however, one was pulled out during a dragging operation that was estimated to have loaded the anchor to about 50,000 pounds. It was estimated that the other two anchors were never loaded more than 20,000 to 30,000 pounds. They were left in place at the end of the operation.

The results of the 20K anchor tests show the importance of selecting the proper anchor projectile and emplacement site. The rock projectile seems suitable for use in competent rock, but does not perform well in rubble or on steep slopes. The tests in coral show that the penetrability of coral will significantly vary over a relatively small area. The rock projectile could have been used in the very hard coral to attain greater penetration and probably greater holding capacity. The soft corals would call for the use of a projectile with a more positive keying action to increase bearing resistance to pullout.

CEL 10K Anchor. The 10K anchor is a new lightweight embedment anchor with a nominal holding capacity of 10,000 pounds (Figure 7). This anchor was taken to Midway for testing but was used for placing part of a temporary four-point moor.

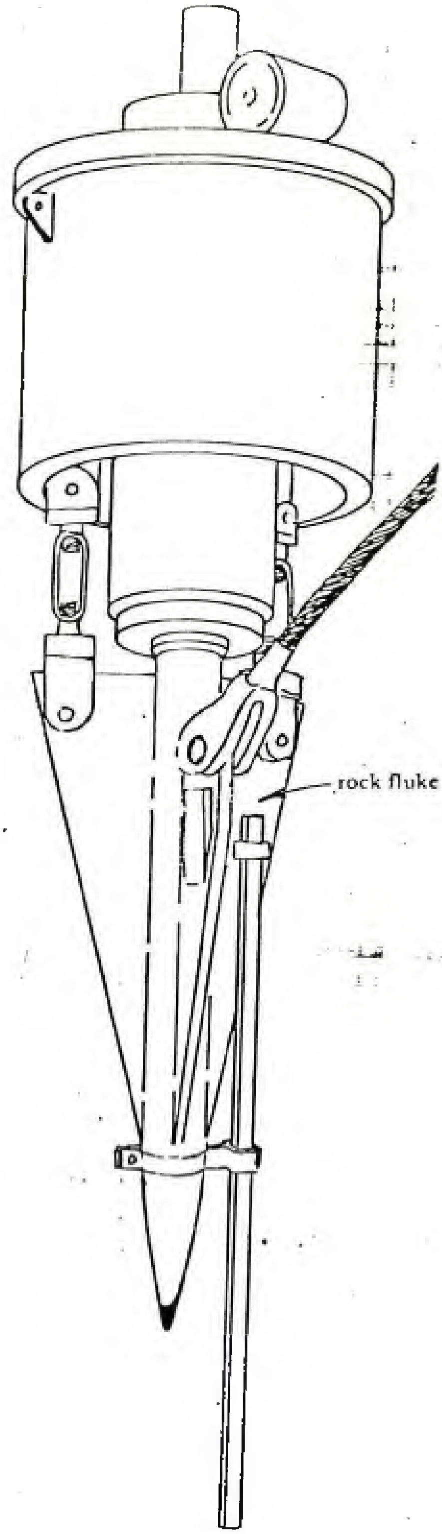


Figure 6. CEL 20K anchor assembly with rock fluke.

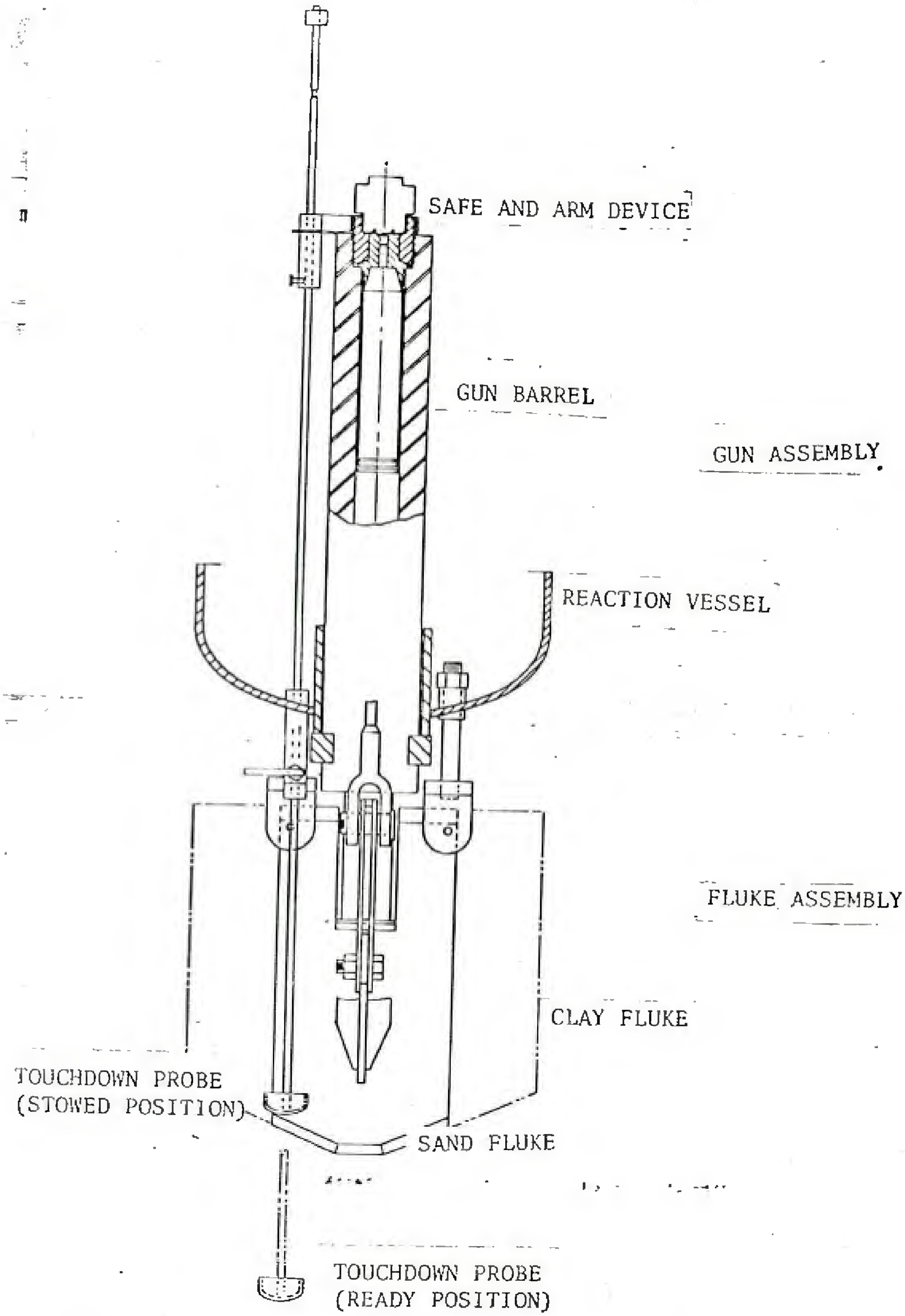


Figure 7. CEL 10K anchor assembly

Two types of projectiles were used, a 1-foot by 2-foot fluke and a 2-foot by 2-foot fluke, both similar in design to the CEL 20K anchor projectiles. Neither projectile was designed for use in coral, but both were slightly modified for use at Midway. Two anchors embedded satisfactorily; a 1-foot by 2-foot fluke penetrating 14 feet and a 2-foot by 2-foot fluke penetrating 9 feet. A third anchor, 1-foot by 2-foot, penetrated but was severely damaged and was easily pulled out by a 35-foot workboat. The coral apparently was too hard for the sediment projectiles so the CEL 20K anchors were used at this site. The two embedded flukes were used as the stern anchors of a four-point moor for an ARS class salvage vessel. The 1-foot by 2-foot fluke pulled out during heavy weather at an estimated load of 30,000 pounds. The 2-foot by 2-foot fluke is still in place, having resisted loads estimated at 15,000 pounds.

These results also show that the sediment type of fluke may be effective in soft coral seafloors, but they are relatively ineffective in harder corals and rocks. The difference in coral properties found within the mooring space of a relatively small ship points out the variability of conditions that can be expected in coral.

Rock Bolts

One of the most commonly used methods of anchoring in rock is rock bolting. Rock bolts are used in mines and tunnels to reinforce rock walls that have been de-stressed by the removal of the supporting rock during excavation. The rock bolts serve as a post-tensioned system to reapply compressive stress to the surface layer of rock.

The rock bolt develops holding power by expanding a collar against the rock wall of a drilled hole. The expanded collar sets up a compressive stress between the rock and collar. Pullout resistance is achieved by the friction of the expanded bolt collar against the rock wall of the drilled hole. Rock bolts do not develop bearing resistance to pullout due to the small expanded area of the bolt collar.

Rock bolts are most effective in hard,high-strength rock where full collar stress can be developed. Weaker rocks crush as the collar expands and little compressive stress and, thus, little frictional resistance to pullout is developed.

The U. S. Army Corps of Engineers conducted a series of investigations on rock bolt reinforcement in which pullout tests were conducted on various sizes of bolts anchored in quartz monzonite, a hard competent rock (Table 2)(Distefano and Boldan, 1965). Pullout tests on large #14 rock bolts indicated holding capacities above 160,000 pounds. Failure was due to material failure of the bolt, not failure of the rock. Tests with a #11 rock bolt indicated an anchor failure load of 76,000 to 84,000 pounds. Several #8 rock bolts were installed in badly fractured rock. As might be expected, the failure loads were extremely varied, ranging from 2,000 to 38,000 pounds. In each of the tests above, the bolts were anchored deep enough to mobilize full strength of the rock. The #14 bolts were 6 feet long, the #11 bolts were 16 feet and 6 feet long, and #8 bolts were 4 feet, 6 feet, and 8 feet long. These lengths seemed to have less influence on the holding capacity than the condition of the rock at the expansion shell.

Table 2. Holding Capacity of Large Rock Bolts in Quartz Monzonite (after Distefano and Boldan, 1965)

Bolt Size No.	Bar Diameter (in)	Expansion Shell Diameter (in)	Length (ft)	Holding Capacity (kips)	Cause of Failure
14	1 3/4	3	6	160+	Material failure of bolt threads
11	1 3/8	2 1/4	6 and 16	76-84	Rock failure
8	1	1 5/8	4,6, and 8	2-38	Badly fractured rock

The recent development of a diver-operated underwater rock drill at CEL (Figure 8) has allowed the use of rock bolts as an anchorage for submarine cables and small craft. Various types of rock bolts (Figure 9) were tested underwater during development of the rock drill (Brackett and Parisi,1975). It was found that rock bolts are effective anchors in competent rock, with holding capacities increasing with greater bolt length and diameter and higher compressive strength in the rocks. The size of the bolt used is limited to the size of the hole that can be drilled. The CEL rock drill is most efficient when drilling 1-inch-diameter holes and can easily drill to an 18-inch depth. The tests show, however, that the full strength of a 1-inch-diameter bolt can be mobilized in a hole 6 inches deep; deeper holes are not really necessary. For bolts in the 1- to 1 1/2-inch-diameter range, a depth of 12 inches is usually enough to develop full strength. Tests indicate single bolt pullout loads as high as 45,000 pounds for a 1-inch-diameter bolt.

A new, larger rock drill is now under development at CEL which allows drilling of up to 4-inch-diameter holes to at least 4 to 6 feet.

High-capacity moors can be obtained by securing padeyes with several bolts (Figure 10). Experimental results using three 1/2-inch-diameter stud bolts show the resulting holding capacity to be about 90 percent of the sum of the individual rock bolt strengths with the bolts spaced at about 5 1/2 inches center to center. In this particular experiment, the individual bolt strengths were 10,000 pounds, and the padeye failed at 27,000 pounds.

At present, the use of rock bolts is limited to diver depth as installation is manual. However, the bolts could be emplaced at greater depths by using a submersible.

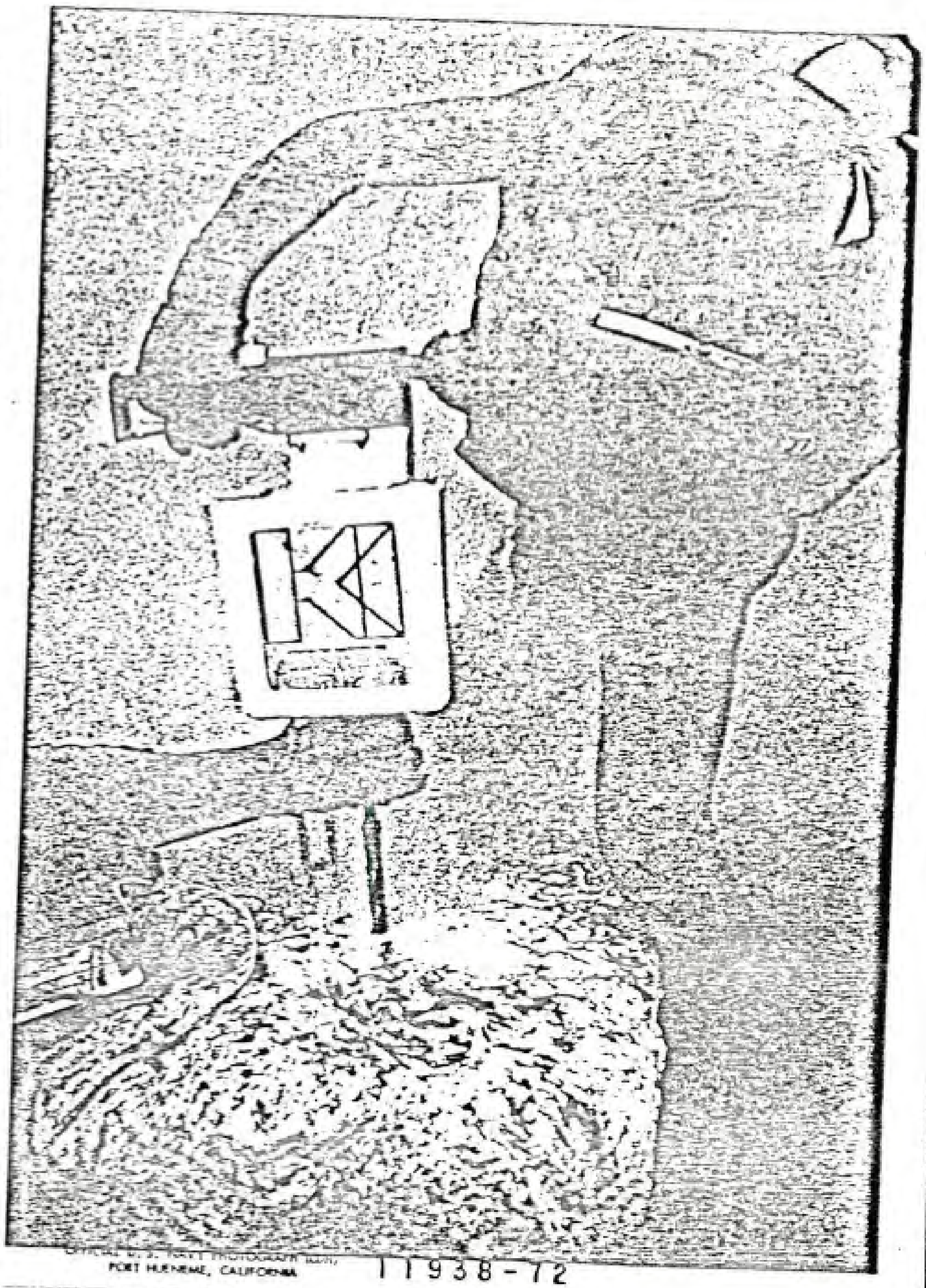
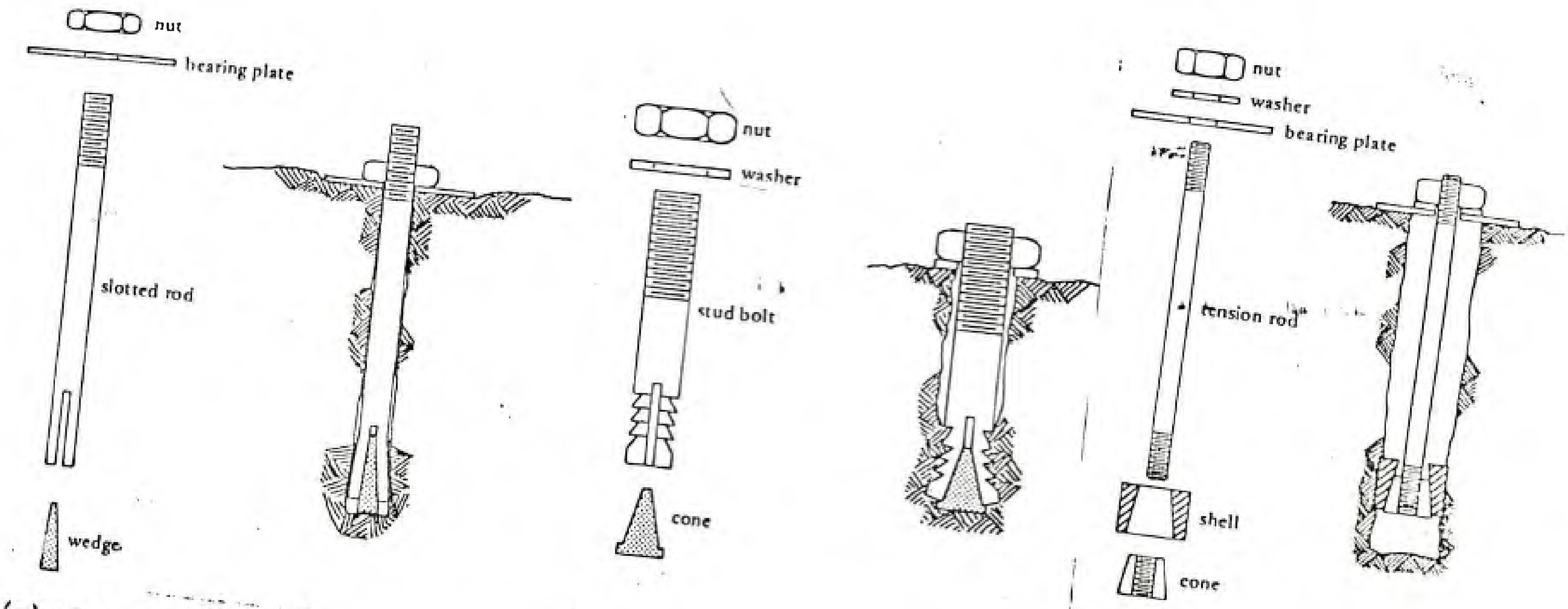


Figure 8. CEL hand-held hydraulic rock-drill for use by divers.

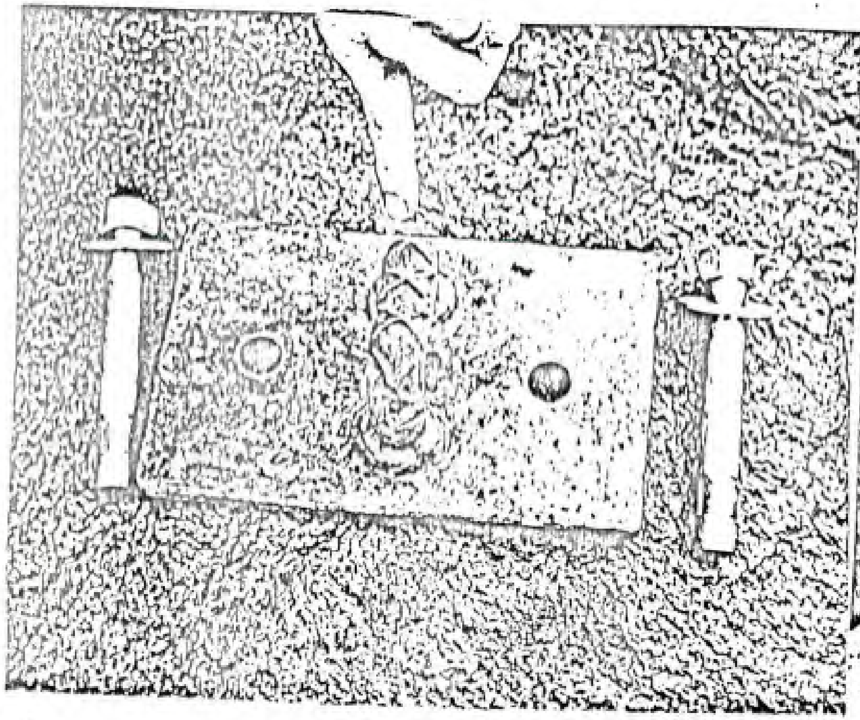


(a) Slot and wedge bolt.

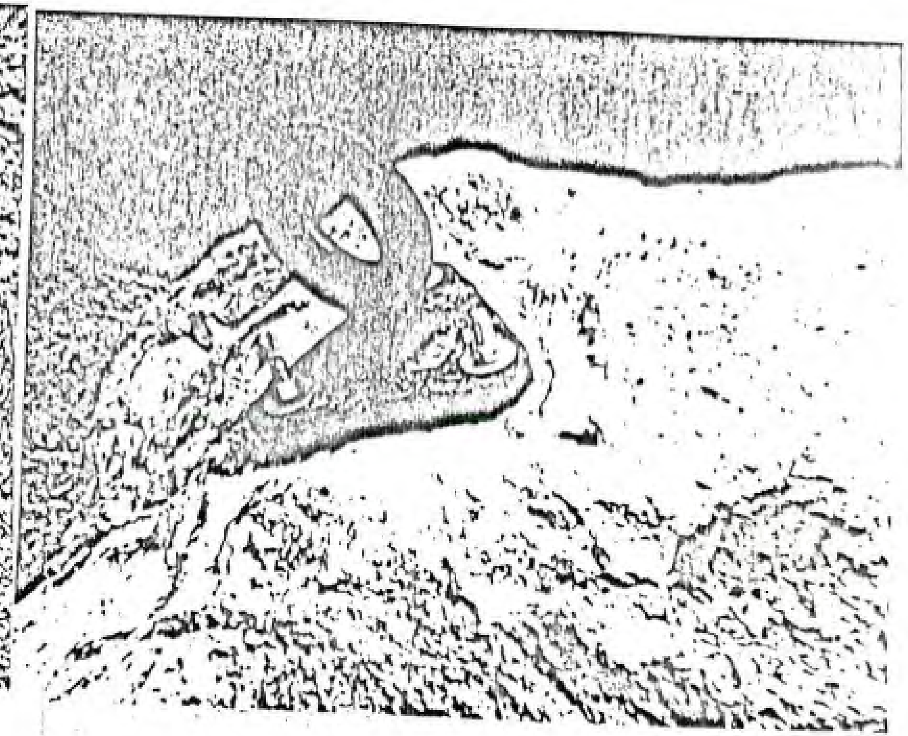
(b) Cone and stud anchor.

(c) Torque-set anchor.

Figure 9. The three basic types of rock bolts.



(a) Two-bolt padeye for ship mooring.



(b) Three-bolt padeye for ship mooring.

Figure 10. Rock bolt padeyes for high capacity moorings.

The results show that rock bolts are suitable for use only in competent rock, the harder the better. Soft rock and fractured rock do not allow full development of the rock bolt strength.

Grouted Anchorages

Grouted bolts, cables, or bars are more consistent than mechanical anchorages in developing high holding capacities in terrestrial applications. This is especially true in badly fractured or low strength rock that is not amenable to the use of rock bolts.

Grouted anchors are basically deformed bars or cables placed in a drill hole which is backfilled with grout or resin. Various companies have different configurations of the anchoring cable and different methods of drilling the hole (Figure 11). The grout is selected to develop enough bond strength with the rock interface to prevent pullout of the grout plug. The anchor rod or cable is sized to allow enough bond area with the grout to develop the ultimate tensile strength of the anchor. For example, about 4 feet of grouted length is necessary to develop the ultimate tensile strength of a #11 bar (1 3/8-inch diameter). This would give a holding capacity of 93,600 pounds. Grouting is advantageous in that the bar may be secured, even in badly fractured rock or low-strength rock.

There is offshore technology available for placing large drilled-in and grouted anchors. Piles and caissons have been placed into sockets drilled into bedrock and then backfilled using the tremie process in depths of 170 feet. The oil industry has used drilled-in anchors to moor exploratory drill rigs. The anchor is placed by drilling a hole, inserting chain, and backfilling with grout. A 9-inch hole can be drilled in 20,000 feet of water. The AEC has developed a large-diameter rock bit with which holes to 42 inches in diameter have been drilled.

There are problems when attempting to drill into exposed rock. In most drilling, the sediment cover provides directional stability to the drill bit; on exposed rock the drill bit will wander over the surface. The use of a drilled-in anchor on exposed rock seafloors is, thus, limited to the depth at which directional stability can be provided to the bit through the use of a stiff drill string or bottom template.

The difficulties of using grouted anchors underwater on a smaller scale are related to the difficulties of placing grout underwater. An experimental grout-dispensing system for use by divers (Figure 12) has been developed (Parisi and Brackett, 1974), but further improvements in this system are needed to insure placement of quality grout in the volumes required. This system could be modified for use by a submersible to allow placement of grouted anchors at greater depths.

ROCK SEAFLOORS

The types of rock found on the seafloor consist of both continental and oceanic rocks. Continental-type rocks are exposed along the continental margins in areas where currents are fast enough to prevent

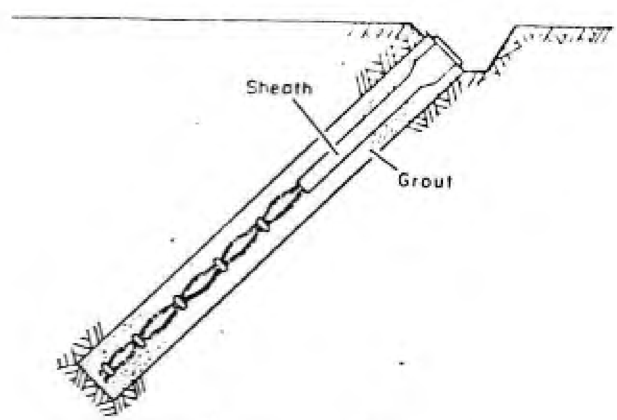


Figure 11a. Configuration of VSL anchor (after Ground Engineering, 1968).

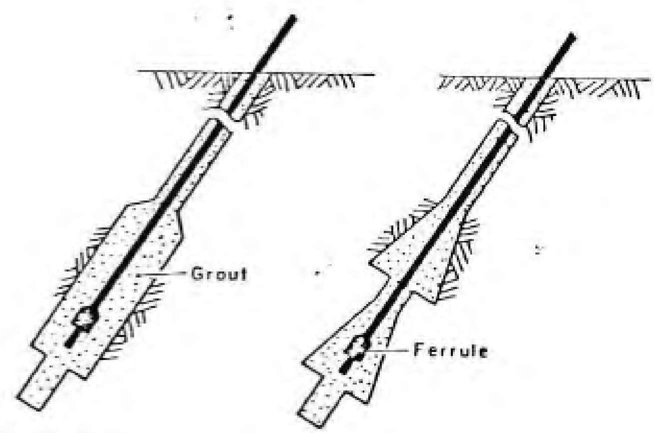


Figure 11b. Configuration of Universal Anchorage Company grouted anchors(after Ground Engineering 1968)

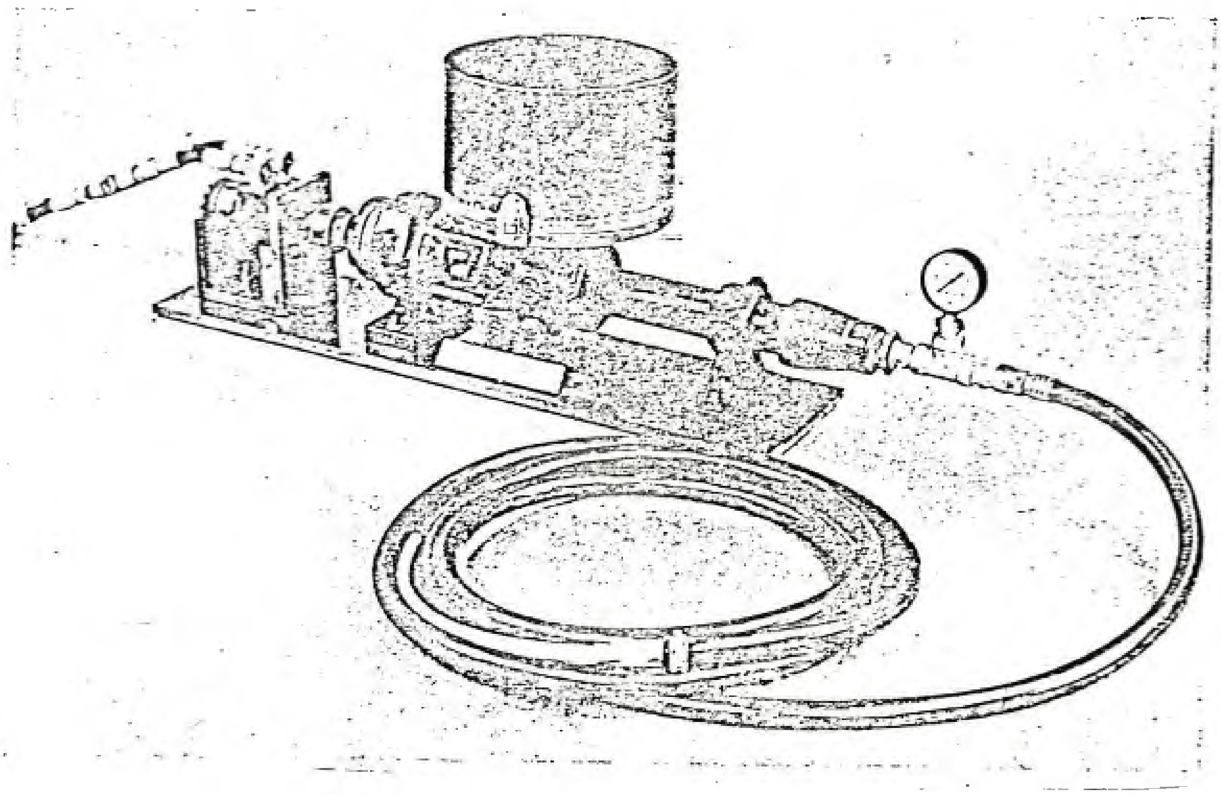


Figure 12. Experimental underwater grout-dispensing system.

sediment deposition or to erode pre-existing sediments. Typical examples are submarine canyons, tidal straits, and rocky, wave-swept shorelines. Glacial debris may also be found on the seafloor in the form of moraines or ice-rafted boulders.

The materials which form most oceanic topographic rock features are composed of coral and basalt. Figure 13 shows the distribution of these features (seamounts, islands, atolls, and guyots) around the oceans of the earth.

Basalts

Most of the rock found in the oceanic regions is extrusive basalt. Extrusive basalt is formed by lava extruding from the earth due to terrestrial and submarine volcanic activity. The rock is exposed in areas of past or present volcanic or tectonic activity such as the Mid-Atlantic Ridge, seamounts, and guyots. The type of basalt composing each feature may differ widely in physical characteristics.

Seamounts are volcanoes which never built up enough to reach the sea surface. They are constructed mainly of fluid lava emptying from summit rifts and craters but lack the large summit caldera and ash cones typical of subaerial volcanoes (Menard, 1964). Observations from Project FAMOUS (French-American Oceanographic Underwater Survey) indicate a similar mode of formation along the median valley of the Mid-Atlantic Ridge (Heirtzler and Bryan, 1975).

Guyots, volcanic islands, and atolls are similarly formed but these features are submarine volcanoes which reached the surface and are now in various stages of growth, erosion, uplift, or drowning. Guyots are volcanoes which reached the surface, were eroded by the waves, and then were drowned or subsided to their present depth. Atolls are islands or guyots which submerged slowly enough to allow a fringing coral reef to build upward as it sank.

Many varied forms of lava extrusions have been observed (Figure 14). They consist largely of pillows or tubes, either filled or more rarely hollow, depending on the history of the lava flow. Photographs of flow surfaces around the base of the island of Hawaii at 10,000 feet and 13,500 feet show a predominance of bulbous forms (12 to 27 inches across) that appear to be the typical configuration of a submarine basalt known as pillows. It could also be the ropy surface of a lava form known as entrail pahoehoe (McDonald, 1967).

Pillows observed on land have glassy surfaces caused by the sudden cooling of the outside layer of lava by contact with water or saturated sediment. The interior of the pillow has a radial structure and appears to have cooled from the surface toward the center with a slightly greater abundance of vesicles (if present) along the upper edge of the pillow. Dredge samples of submarine pillows are usually broken in wedge-shaped fragments due to the radial structure and will sometimes pop and break into pieces when brought to the surface due to the expanding gas in the vesicles.

Normally, lava is filled with dissolving gases which come out of solution to form bubbles in the hardening lava. This is what gives

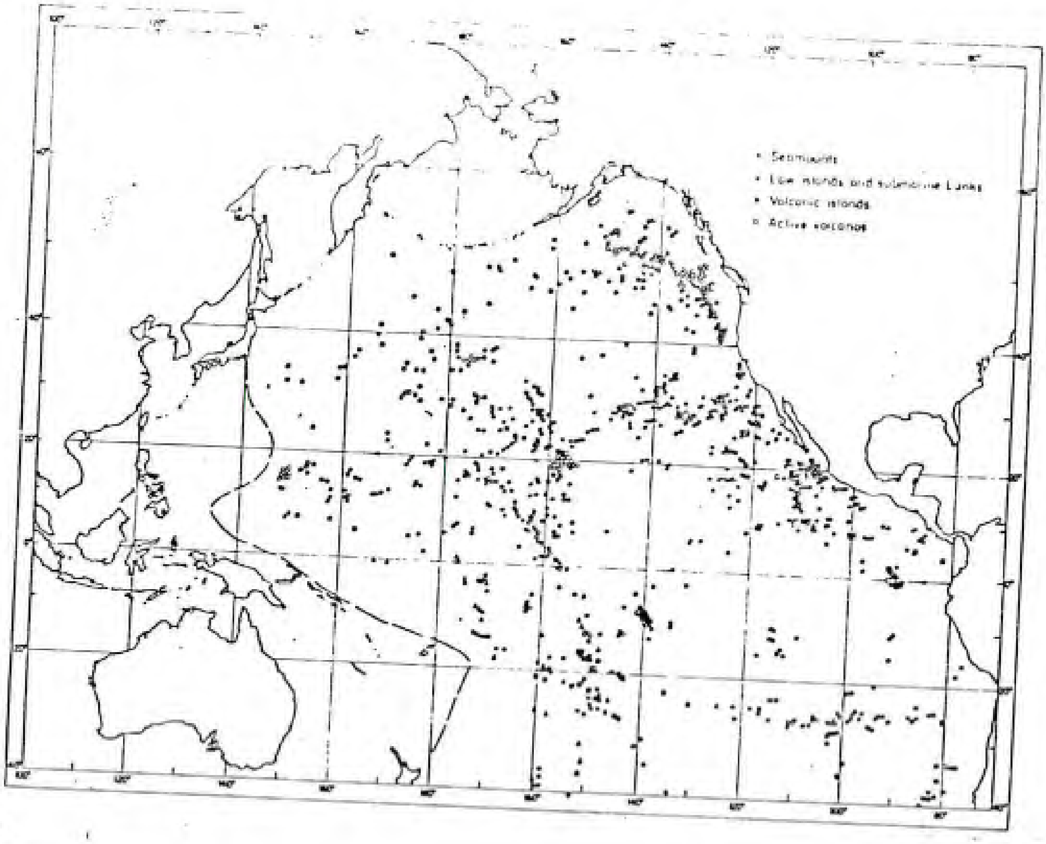


Figure 13a.

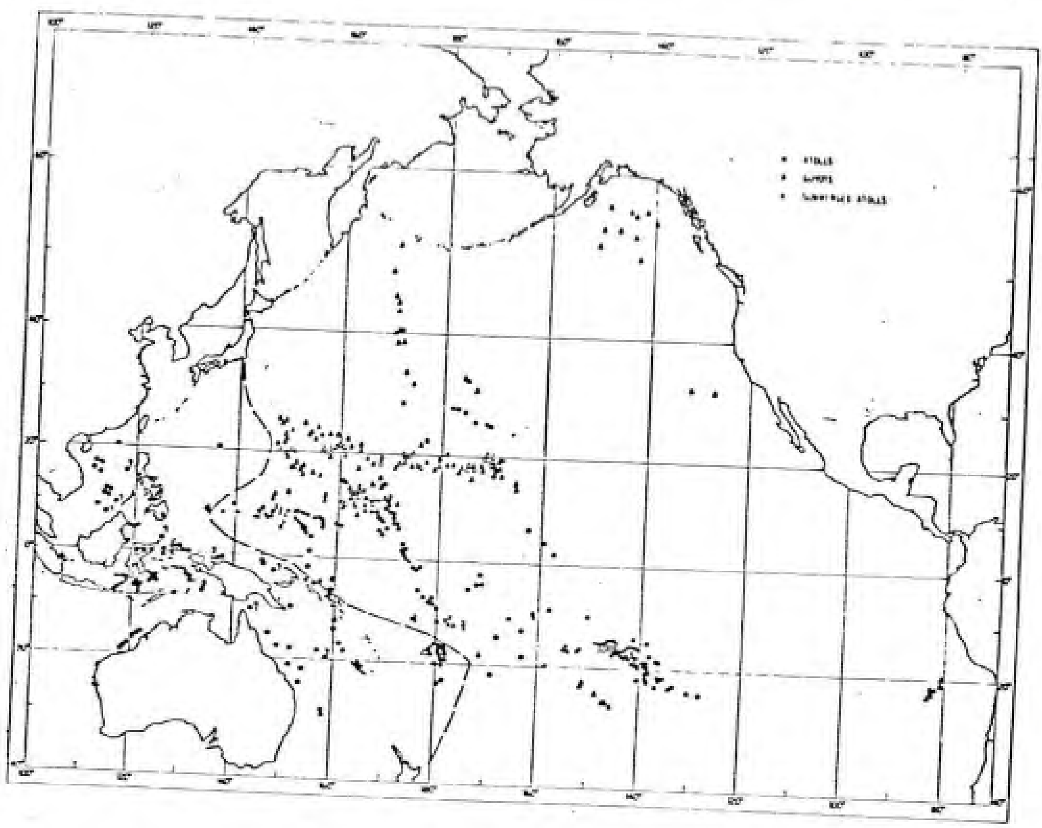


Figure 13b

Figure 13a & b. Distribution of seamounts, volcanic islands, low islands, atolls, and guyots in the Pacific Basin (from Menard and Ladd, 1963).

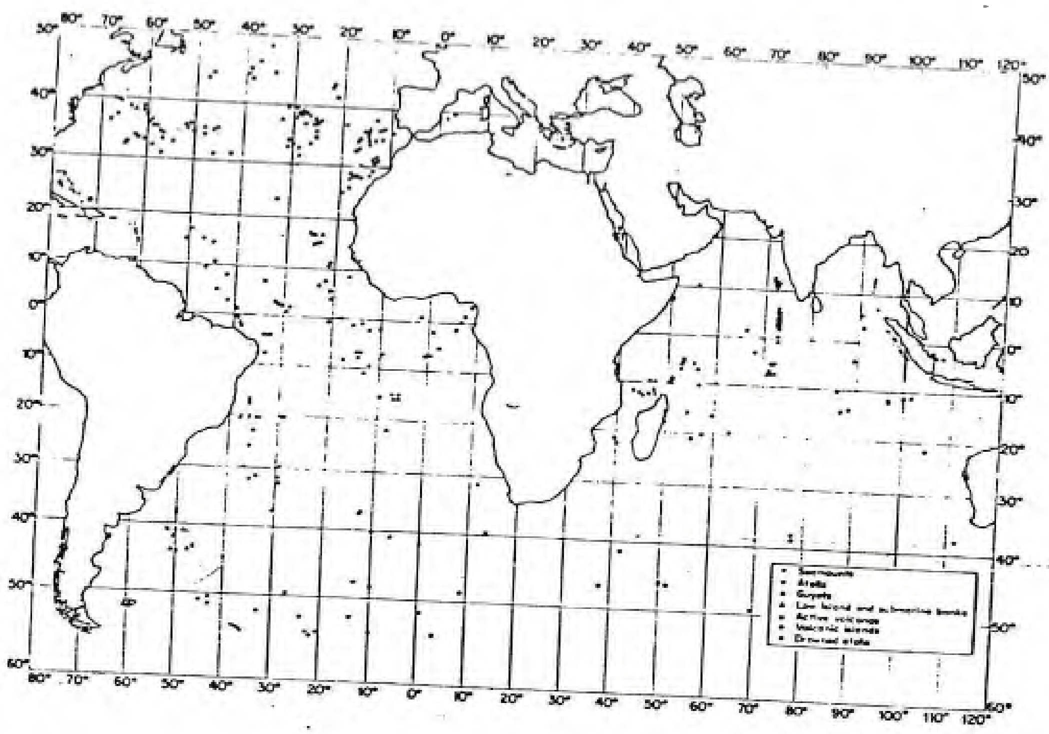


Figure 13c. Distribution of seamounts, volcanic islands, low islands, atolls, and guyots in the world exclusive of the Pacific (from Menard and Ladd, 1963).

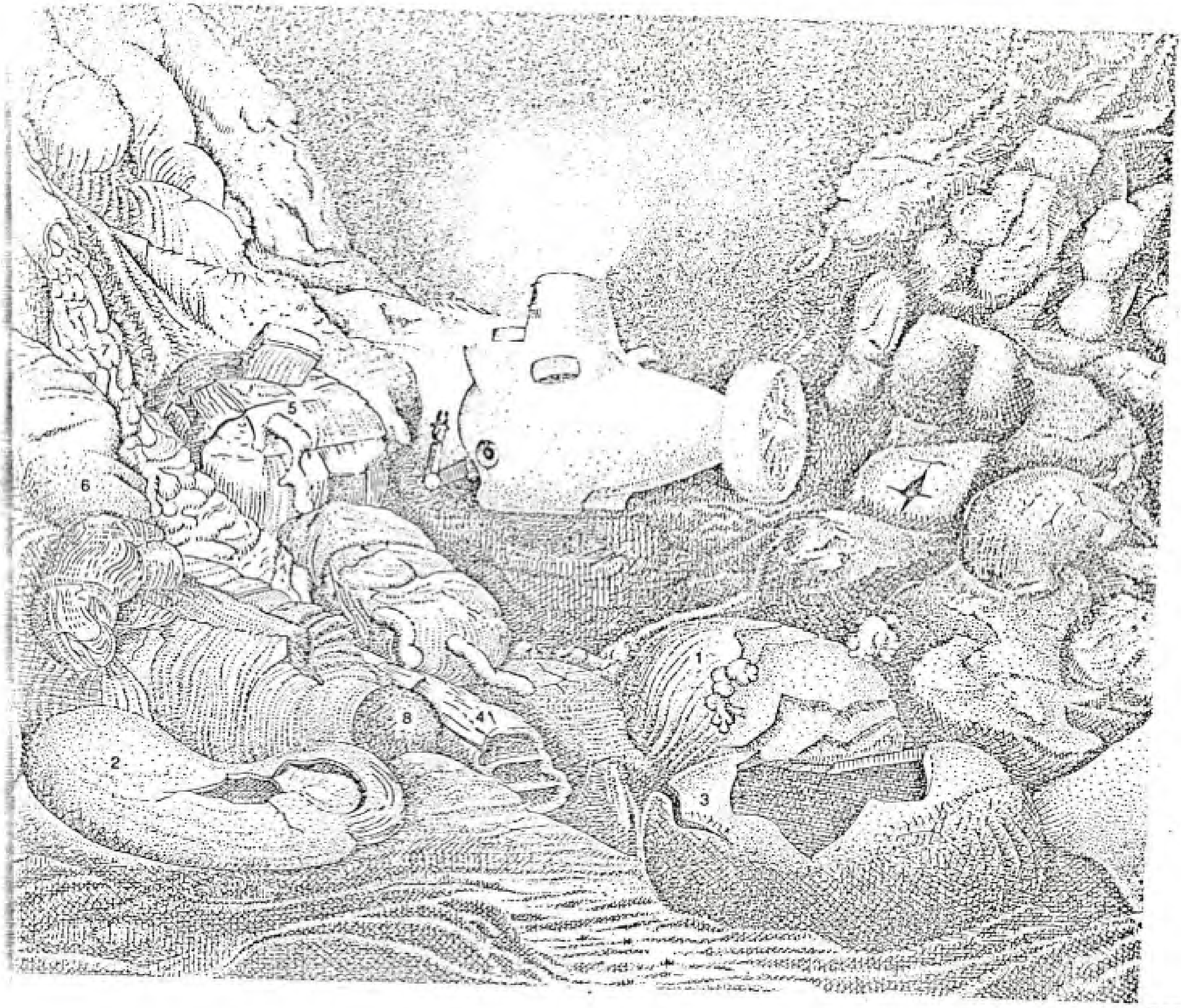


Figure 14

E SUBMARINE LANDSCAPE of lava extrusions was ob- the intersection of two lava-flow fronts in the Mid-Atlantic drawing is based on a sketch of the scene made by one of rs (Bryan), who served as one of the observers aboard the ring its dives into the Mid-Atlantic Rift. The numbers ome of the lava forms that were observed: (*1*) bulbous ith knobby budding, (*2*) a flattened pillow formed by the inage of lava while the skin was still plastic, (*3*) a hollow blister pillow formed by the drainage of lava after the skin had solidified, (*4*) a hollow layered lava tube formed by temporary halts in a falling lava level, (*5*) a bulbous pillow with a "trapdoor" and toothpaste budding, (*6*) an elongate pillow, typical of a lava extrusion on a steep slope, (*7*) a breccia cascade, formed on very steep slopes where the lower end of an elongate pillow has ruptured, releasing a cascade of fluid lava, and (*8*) an elongate pillow swelling into a bulbous form along a longitudinal spreading crack.

From "The Floor of the Mid-Atlantic Rift" by J. R. Heirtzler and W.B. Bryan.

subaerial flows their vesicularity. However, deep water flows are subject to high hydrostatic pressure which prevents separation and expansion of the dissolved gases. The result is that below about 6,500 feet the basalt should be nonvesicular. For basalts found in shallow water, the characteristics will be about the same as subaerial flows. Above 2,500 feet, 10 percent of the vesicles have an average diameter greater than 0.5 mm and the rock has a specific gravity of 2.8. As the depth increases, the vesicles become smaller and fewer; at 13,000 feet vesicles are rare, and the specific gravity is about 3.0.

There are circumstances under which vesicular basalts are found at much greater depths than expected. Basalt with a subaerial degree of vesicularity have been found on guyots as deep as 6,500 feet. These basalts were probably formed when the guyot was at least 3,300 feet nearer the surface. Of the guyots samples, all appear to be drowned volcanic islands with truncated tops sprinkled with erosional debris such as basalt cobbles and pebbles, and sometimes with low coral banks. Therefore, the type of rock to be found on guyots is unpredictable without sampling. The rock may be formed at or near the surface, eroded, overgrown with coral, and then submerged. Similar events may occur in the history of the other volcanic topographic features.

The mechanical properties of basalt vary widely due to the diversity of physical characteristics in basalts. The vesicularity will vary, depending on the depth at which the basalt was formed, and the crystal structure will vary according to the composition of the original lava and the rate at which it was cooled. Once the rock has formed, it is subject to chemical and physical weathering that will reduce its strength. In addition to this variation in material properties, the rock mass may differ in the amount of jointing and fracturing from one site to the next.

Examples of this variation in properties are available. Samples of basalt taken during SUPSALV anchor tests in basalt at Anacapa Island, California, indicated a Schmidt N hardness of 32, a unit weight of 163 pcf (2.6 g/cm^3) and an unconfined compressive strength of 16,400 psi. Later tests at Cobb Seamount yielded samples with a 20,000 psi compressive strength (Smith, 1971). Tests of a basalt with a hyalophitic texture, indicative of submarine or subglacial formation, show a compressive strength of 45,400 psi, a tangent modulus of 11.3×10^6 psi, and a unit weight of 176.9 pcf (Deere and Miller, 1966).

No tests have been conducted to determine how an embedment anchor will perform in pillow basalt, and few tests have been conducted to evaluate performance in other basalts. Due to the variability in strength and structure of submarine basalts, a bottom survey and accurate positioning may be required to ensure proper placement.

A more reliable method of anchoring would be to use rock bolts or grouted rods to tie down a padeye. The mooring line could then be secured to the padeye. Beyond diver depth this would require the use of a manned or remote-controlled submersible, but submersibles may be necessary to carry out a bottom survey, even if embedment anchors were to be used.

Coral

Coral is extremely variable material, and the name is used loosely to identify a variety of calcareous materials. Actual coral forms are recently-produced reef materials formed by the secretion of a calcareous exoskeleton by coral polyps and certain algaes. This form of "coral" is usually very porous due to the gaps left between formations by the inter-growth of coral heads and the cementation of fragments.

The central mass of older reefs may be composed of limestone. This is a relatively hard rock formed by the dissolution and precipitation of the calcium and magnesium carbonates produced by coral forms into a more dense, crystalline material. Limestone is usually found where coral or shell deposits have been exposed to fresh water. This could occur where a reef has emerged from the sea due to uplift or due to a lowering sea level or where fresh water springs occur underwater. It may also be found at former coral sand beaches that have been submerged. Fresh water dissolves the coral and becomes saturated with calcium carbonate. When this saturated solution comes in contact with sea water, the calcium carbonate is precipitated, forming limestone and cementing together any surrounding particles.

A third type of coral, known as "cascajo" (pronounced kas-ká-ho), comprises the various forms of partially cemented coral debris. High-quality cascajo consists of limestone fragments left by old, emerged reefs, while lower quality forms include lagoon sediments and reef debris.

Typical properties of corals, as determined by Duke, 1949, indicate an unconfined compressive strength of 6,000 psi and bulk specific gravity of 2.6 for limestone, 1.7 to 2.2 for reef coral, and 1.8 to 2.4 for lagoon coral. Samples of reef coral taken at anchor test sites (Smith, 1971) indicated compressive strength of 1,600 to 2,500 psi.

Coral is a good material for embedment anchoring. Much higher holding capacities are obtainable than are possible in sediments, and so the general efficiency of embedment anchors is improved in coral. Maximizing this efficiency will require more than one type of projectile due to the variability of strength and hardness found in coral.

Rock bolts are not able to develop much holding capacity in coral due to the low compressive strength of the material. When load is applied to the bolt, it simply crushes the sides of the drill hole and slips out. Installation is also complicated by the presence of large voids in the coral which prevents proper expansion of the bolt collar. Rock bolts may be used for lower capacity applications; for example, they have been used to tie down a submarine cable at Barking Sands, Kauai. A solid limestone seafloor would allow higher capacity to be developed.

Grouted anchors run into problems due to the porosity of coral. Large void spaces can make it impossible to properly fill the drill hole with grout. The grout may simply flow into surrounding voids or it may cause the grout to dissolve through over-dilution by the surrounding water. The results would be better in solid limestone seafloor where a reliable high-capacity grouted anchor could be emplaced.

ROCK PENETRATION

Impact and Penetration Phenomena

A considerable amount of research into the behavior of rock under impact loading has been conducted in order to understand and improve drilling and mine excavation techniques. These studies have concentrated on developing models and failure criteria for rock under dynamic loading.

Early tests were conducted by dropping weighted chisels from various heights onto limestone samples and monitoring the bit force versus bit displacement during penetration (Pennington, 1954). It was found that, for all impact energies, an initial pulse was produced of the same force over the same depth of penetration (Figure 15). Impacts with insufficient energy to surpass the proportional limit of this initial peak (point B on Figure 16) were rebounded, while those with greater energies continue to penetrate. Thus, it is apparent that there is a critical energy required to initiate penetration by a given projectile in a given material.

Pennington found that this initial loading peak could be broken down into four regions (Figure 16). The first region, 0-A, is characterized by the crushing of surface irregularities. Thereafter, the bit force is transmitted through a thin layer of crushed rock. The second region, A-B, is a region of elastic response accompanied by some surface damage. At point B, the proportional limit, the rock has reached its crushing strength, and a wedge of crushed, compacted rock forms under the nose of the bit. The peak of the curve, point C, represents the point at which sufficient stress has been transferred into the surrounding rock to cause rupture, and the rock chips away, leaving the wedge of crushed rock under the bit nose unsupported. The wedge crumbles, and the bit force declines until the bit again contacts solid rock. Later studies have confirmed these failure mechanisms and have shown that continued penetration will result in a series of repeated chipping actions (Figure 17)(from Dutta, 1972).

Rock penetration tests in limestone performed by Sandia Laboratories (Patterson, 1972) confirm the same failure mechanisms by the presence of three distinct failure zones observed in the rock surrounding the penetrator (Figure 18). Zone 1 consisted of comminuted rock adjacent to the body and about one-half to one body diameter thick. This was the remains of the crushed rock wedge which is pushed aside as chipping occurs. Zone 2 consisted of the chipped rock; it was a layer of brecciated rock about one body diameter thick. This zone is the remnants of the rock chipping which follows the crushing. The third zone was a shear deformation zone about two body diameters thick. All the zone dimensions were constant from the rock surface to the tail of the projectile; at that point the zone boundaries tapered to meet at a point approximately 1 inch below the tip of the nose section.

The post-penetration state of the rock shows that most of the resistance of the projectile to direct uplift will be developed at the nose section. Here the zone of broken rock is smallest so more of the elastic strain in the deformed rock of zone 3 can be transmitted directly to the projectile and, thus, increase frictional resistance to pullout.

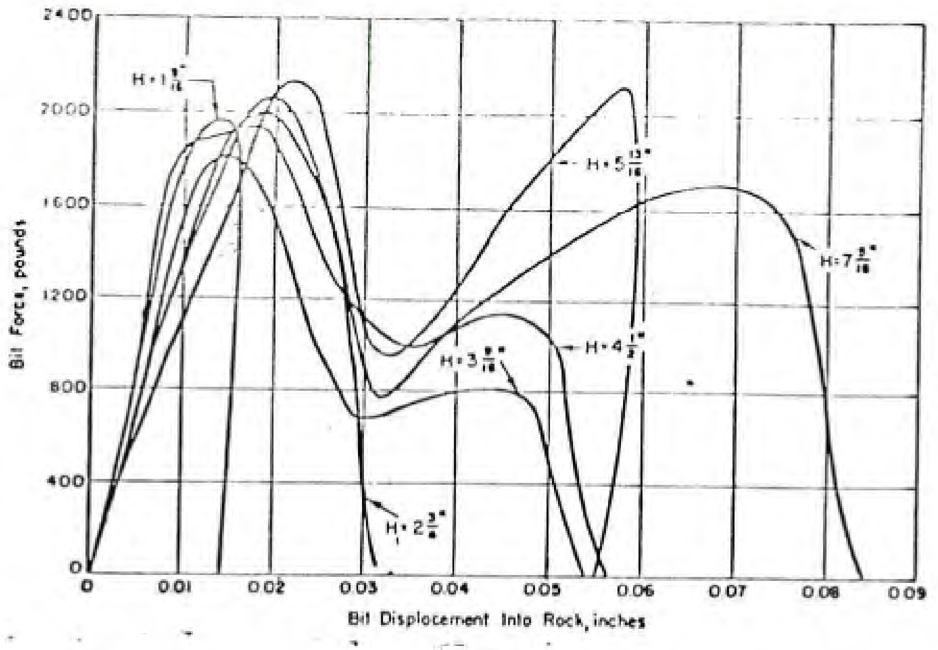


Figure 15. Bit force as a function of displacement of bit into rock for several single blow drops from various heights (from Pennington, 1954).

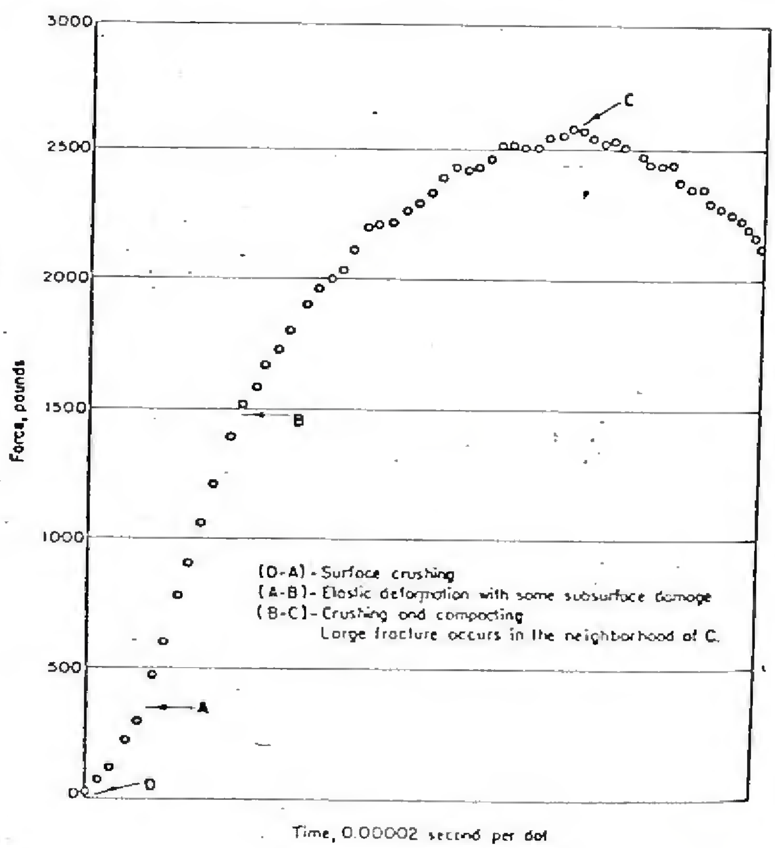


Figure 16. Expanded first force pulse of a force waveform for a single blow (from Pennington, 1954).

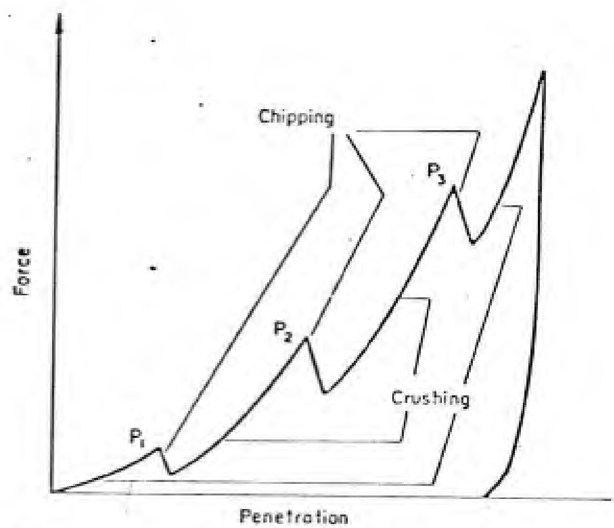


Figure 17. Shape of experimental force-penetration curves (from Dutta, 1972).

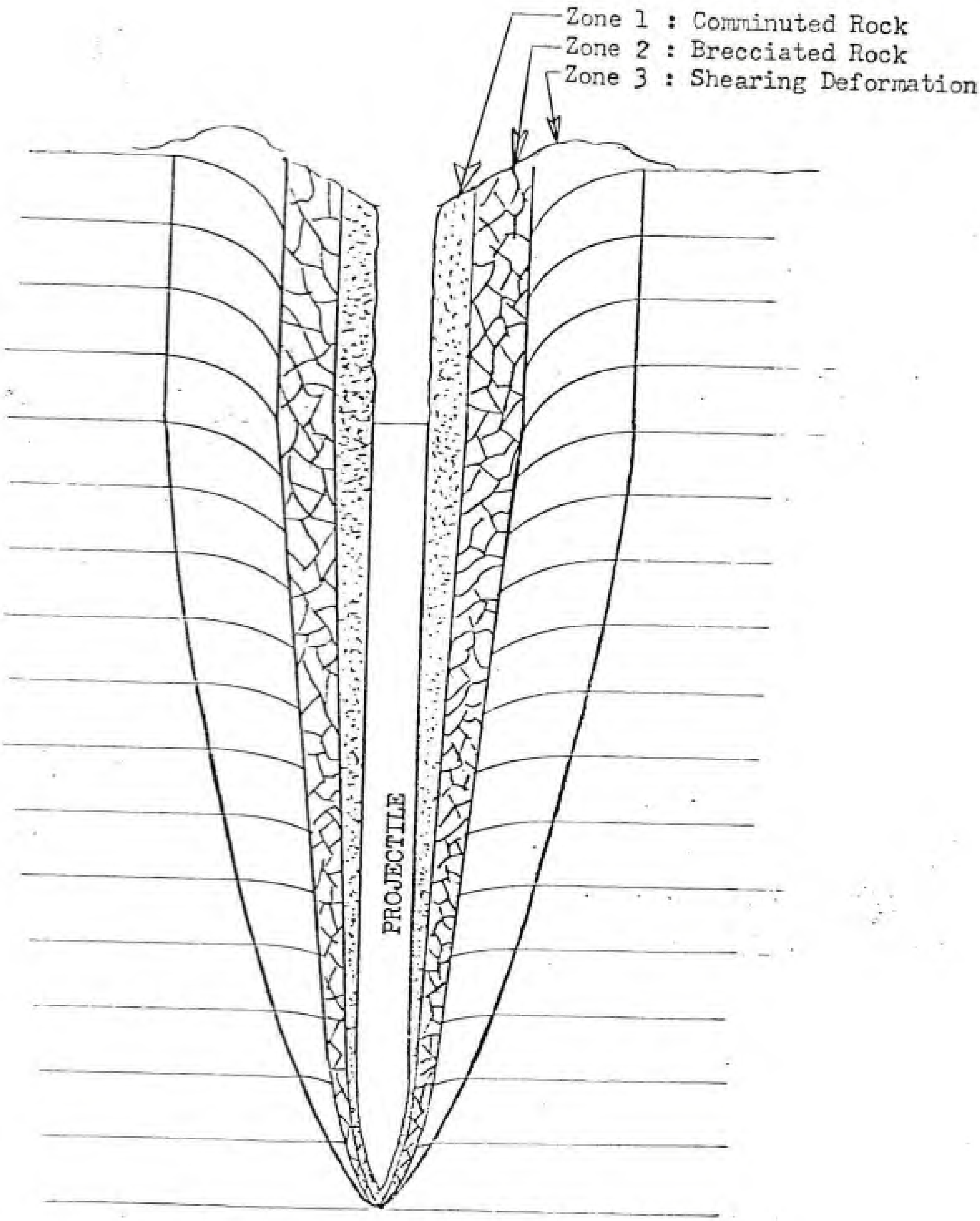


Figure 18. Damaged rock zones around a projectile embedded in rock.

The best projectile shape for an anchor would be a shape which minimizes the chipping zone.

Three of the Sandia penetrometers were completely rebounded from the rock after having penetrated as much as 5.5 feet. Remembering that during penetration considerable energy is expended in elastic deformation of the rock prior to crushing, some rebound should not be surprising. Any projectile that expends its last energy during the elastic phase of a chipping action will be subject to some rebound. This can be seen in Figure 17 and the H = 5 13/16 inches curve of Figure 15. The amount of rebound experienced in the Sandia Laboratory tests, however, seems excessive, considering the low and medium static compressive strengths and low Young's modulus of the rock tested. The excessive elastic behavior of the rock must therefore be due to the dynamic loading.

The dynamic loading conditions which cause complete rebound of the projectile from the hole must be avoided when using embedment anchors. Such rebound will make the anchor useless. In order to understand why such excessive rebound would occur, we must look further into impact phenomena and dynamic loading of rock. More recent studies of impact phenomena have concentrated on explaining the impact failure process and, in particular, why rocks exhibit higher dynamic strength than static strengths. Primary differences between stress pulse loading (impact) and static loading were summarized by Singh and Hartman, 1961. They state that the crack propagation velocity is generally far lower than the stress pulse velocity; thus, the cracks do not have time to grow before the pulse has traveled through the rock and the stress has been removed. This results in an apparent increase in compressive strength. In addition, the pulse affects only a small portion of the solid, and fractures may develop in one region without affecting the rock outside that region. Compression pulses may also be reflected from a surface or joint to give a tension pulse, and when the pulse meets a boundary obliquely, both distortional and dilational pulses are produced. The interference and superposition of such pulses may cause very complex stress distributions and fractures through the rock. The net effect of all this behavior can cause even ductile materials to behave like brittle materials when dynamically loaded.

Thus, rebound of a projectile can occur if the dynamic compressive strength of the rock is greater than the stress that can be applied by the projectile. This is probably what caused the rebound of the Sandia penetrometers. The projectile initially penetrated but expended energy as it did so. Finally, the projectile reached a point where it could cause considerable elastic deformation but did not have enough energy to cause further failure of the rock. Then the rock rebounded, pushing the penetrometer out of the hole.

The foundation of a dynamic fracture criterion that is sensitive to strain rate, absolute temperature, and confining pressure has been carried out, based upon dynamic tests on Dresser basalt (Lindholm et al., 1974). This basalt is a metamorphosed basalt unlike oceanic basalts, but the density and strength are within the same range as those expected in the oceanic basalts.

At ocean temperatures, their results showed that the minimum stress required to initiate fracture is sensitive to the strain rate and pressure (Figures 19 and 20). The higher the strain rate and confining pressure, the greater the compressive failure strength, and, thus, the greater the energy input required to assure penetration. Therefore, a rock in 20,000 feet of water is more difficult to penetrate than an identical one near the ocean surface, due to an almost 60 percent increase in compressive strength. In considering strain rates, this means that, if two projectiles have the same frontal and side area and the same kinetic energy, but different velocities, the slower projectile will penetrate further.

Penetration Prediction

Penetration Equations. The basic equation used for penetration depth of a low-velocity projectile in a low-strength material is based upon simply momentum transfer (Maurer and Rinehart, 1960).

$$P = \frac{K_1}{a} M (V_o - V_c) \qquad (1)$$

where
- P = depths of penetration
- K_1 = constant dependent upon mechanical properties of the target and projectile materials
- a = frontal area
- M = projectile mass
- V_o = terminal impact velocity
- V_c = critical impact velocity

The critical impact velocity, V_c, varies with the material hardness. The terminal impact velocity, V_o, must be greater than V_c to penetrate the target material. The depth of penetration depends on the momentum in excess of the critical value necessary to initiate penetration.

At higher projectile velocities, a more suitable penetration equation is given by

$$P = K_2 \frac{M}{a} \ln (1 + K_3 V_o^2) \qquad (2)$$

where K_2 and K_3 are constants. This is the general form of the Poncelet equation developed in 1829. Most rock penetration equations since that time have been devoted to modification of the equation and finding satisfactory values for the constants K_2 and K_3. One of the most used penetration equations,

$$P = \frac{W}{a} K \log_{10} \left(1 + \frac{V_o^2}{215{,}000}\right) \qquad (3)$$

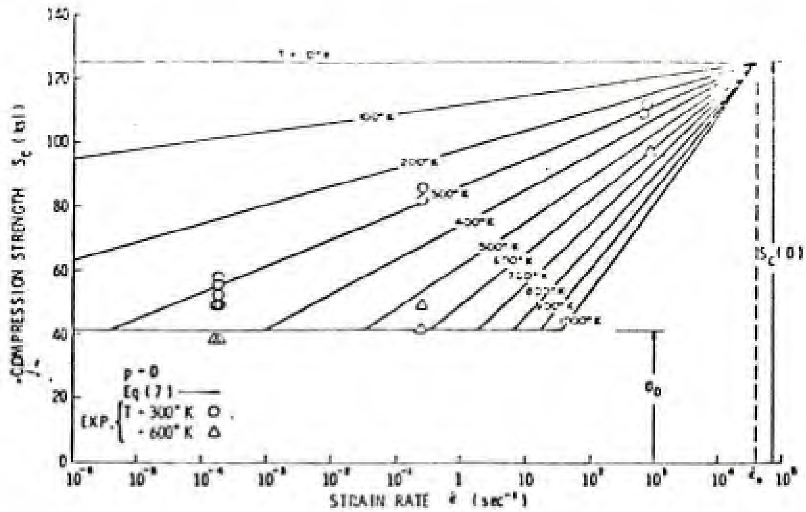


Figure 19. Compression strength as plotted as a function of strain rate and temperature (from Lindholm, et al., 1974).

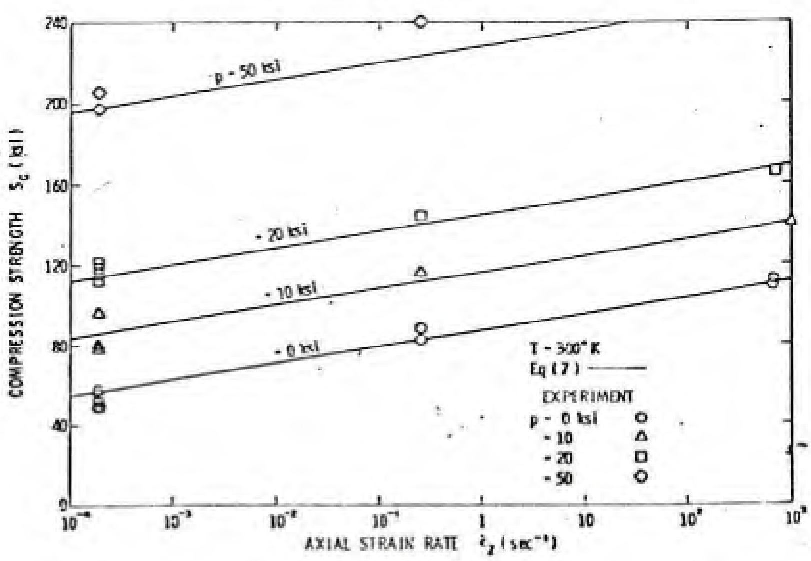


Figure 20. Effect of confining pressure on the strain-rate dependence of the compression strength. (from Lindholm, et. al., 1974)

where a = frontal area, in^2
V_0 = impact velocity, ft/sec
W = projectile weight, lb
K = constant depending on target material

was developed by the U. S. Army Corps of Engineers to predict bomb penetration.

The most extensive and well-documented studies of rock penetration have been conducted by Sandia Laboratories. They tested air-deliverable penetrometers in both soils and rock. On the basis of those tests, an empirical equation was developed (Young, 1967) to predict the depth of penetration. Two equations were found:

$$P = 0.53\ SN\ \left(\frac{W}{A}\right)^{1/2} \ln\left(1 + 2V^2 10^{-5}\right) \qquad V < 200 \text{ fps} \qquad (4a)$$

$$P = 0.0031\ SN\ \left(\frac{W}{A}\right)^{1/2} (V - 100) \qquad V \geq 200 \text{ fps} \qquad (4b)$$

where S = soil or rock coefficient
N = nose shape coefficient
A = frontal area, in^2
W = projectile weight, lb
V = projectile velocity, ft/sec

Further tests of the penetrometers were conducted in rock (Patterson, 1972) and Young's equations were found to correlate best with the test results. In this work, rock coefficients were found to vary between 1 and 3.

For predicting penetration of embedment anchors, the best equation at present is Equation (3), the modified Poncelet equation. Young's equation, (4b), is of interest because of its inclusion of a nose-shape coefficient. Unfortunately, embedment anchor shapes are completely different from the shapes which have been previously investigated. A series of test firings of anchor shapes into a rock for which S, Young's soil or rock constant, has already been determined would be necessary to determine N, the nose shape constant. In addition, Young's equation does not seem to follow conclusions that, for projectiles of the same shape, frontal area, and kinetic energy, heavier and slower projectiles will penetrate further.

Selecting Rock Constants. The development of methods for predicting projectile penetration in rocks can follow one of two paths, an empirical solution or an analytical solution. To date, complete understanding of the variables involved has prevented a totally analytical solution, and so empirical constants are used even in analytical treatments of penetration. These constants are most often selected to represent the soil properties and the shape and material properties of the projectile. Problems arise when constants are used to represent rock properties, because the dynamic properties differ significantly from the static properties, the basis upon which most soil constants are selected.

Table 3. Calculated Rock Constants for Coral Using U. S. Army Corps of Engineers Poncelet Equation

Test No.	Projectile Velocity (ft/sec)	W/A (lbs/in^2)	Penetration (ft)	Calculated K	Test Site
MERDC XM50					
3	400 (est)	8.24	17	8.5	Key West, FL
4	400 (est)	8.24	22	11.1	
MERDC XM200					
1	452	7.1	17	8.3	
2	410		18	10.1	
3	452		19	9.2	
4	318		21	17.7	
5	410		28	15.8	
6	452		31	15.1	
7	318		10	8.4	
CEL 100K					
P1	425	15.1	35	8.8	Diego Garcia
P2	410		26	6.9	
P3	390		35	10.0	
P4	390		35	10.0	
P5	380		31	9.2	
P6	380		35	10.4	
P7	-		30	-	
P8	395		30	8.4	
P9	390		35	10.0	
P10	-		31	-	
T1	385		35	10.2	
T2	315	17.5	28	9.7	
T3	385	15.1	28	8.1	
T4	335	17.5	28	8.8	
T5	335		32	10.0	
T6	335		30	9.4	
T7	330		29	9.3	
T8	325		30	9.9	
CEL 20K					
43	405	8.75	3	1.4	Midway Island
44	405	8.75	14	6.5	
45	405	8.75	10	4.6	
CEL 10K					
1	392	4.7	7	8.2	
2	380	4.7	14	13.3	

Table 4. Deere and Miller's Engineering Classification for Intact Rock

I. On Basis of Strength

Class	Description	Uniaxial Compressive Strength, lb/in^2
A	Very High Strength	Over 32,000
B	High Strength	16,000-32,000
C	Medium Strength	8,000-16,000
D	Low Strength	4,000-8,000
E	Very Low Strength	Less than 4,000

II. On Basis of Modulus Ratio

Class	Description	Modulus Ratio[1]
H	High Modulus Ratio	Over 500
-	Average Modulus Ratio	200-500
L	Low Modulus Ratio	Less than 200

Classify rock as B, BH, BL, etc.

[1] Modulus Ratio = E_t/σ_a(ult.)

where E_t = tangent modulus at 50% ultimate strength

σ_a = uniaxial compressive strength

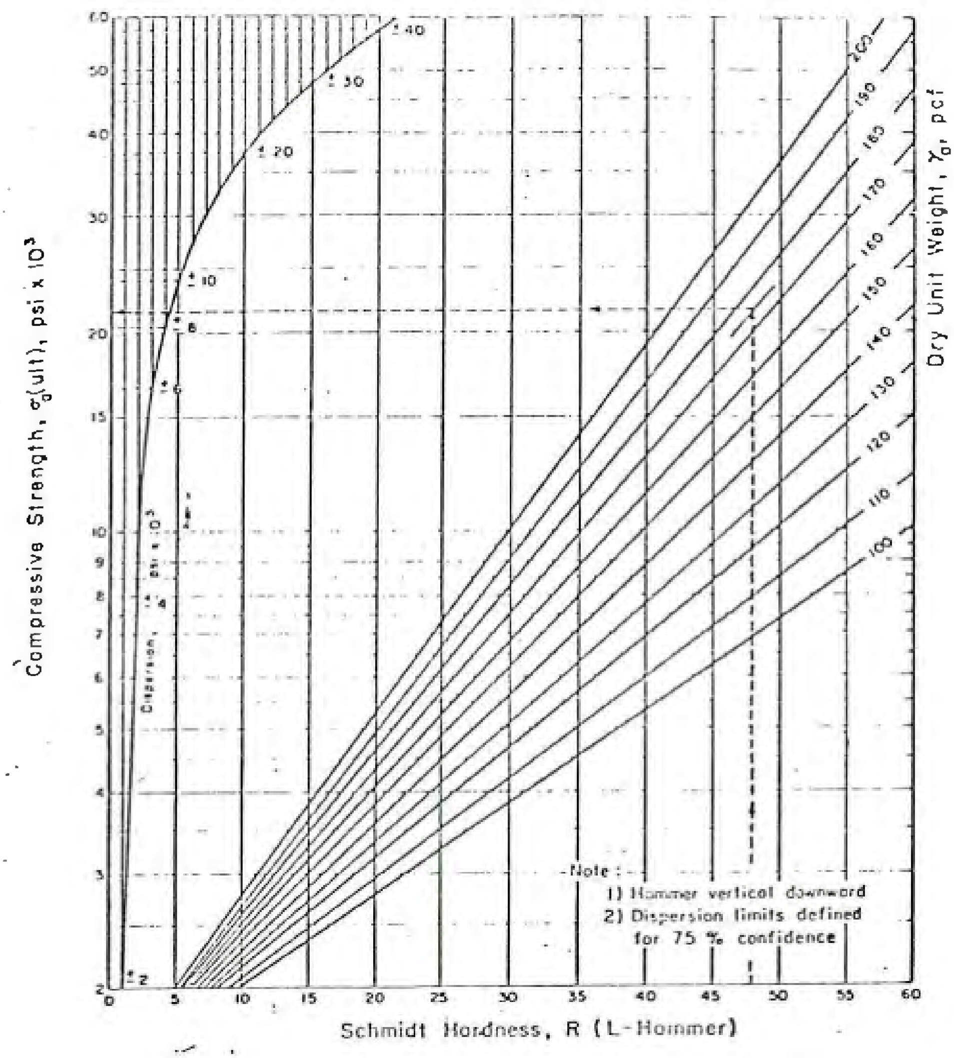


Figure 21. Nomograph for determining compressive strength of rock using Schmidt hardness and dry unit weight (from Deere and Miller, 1966).

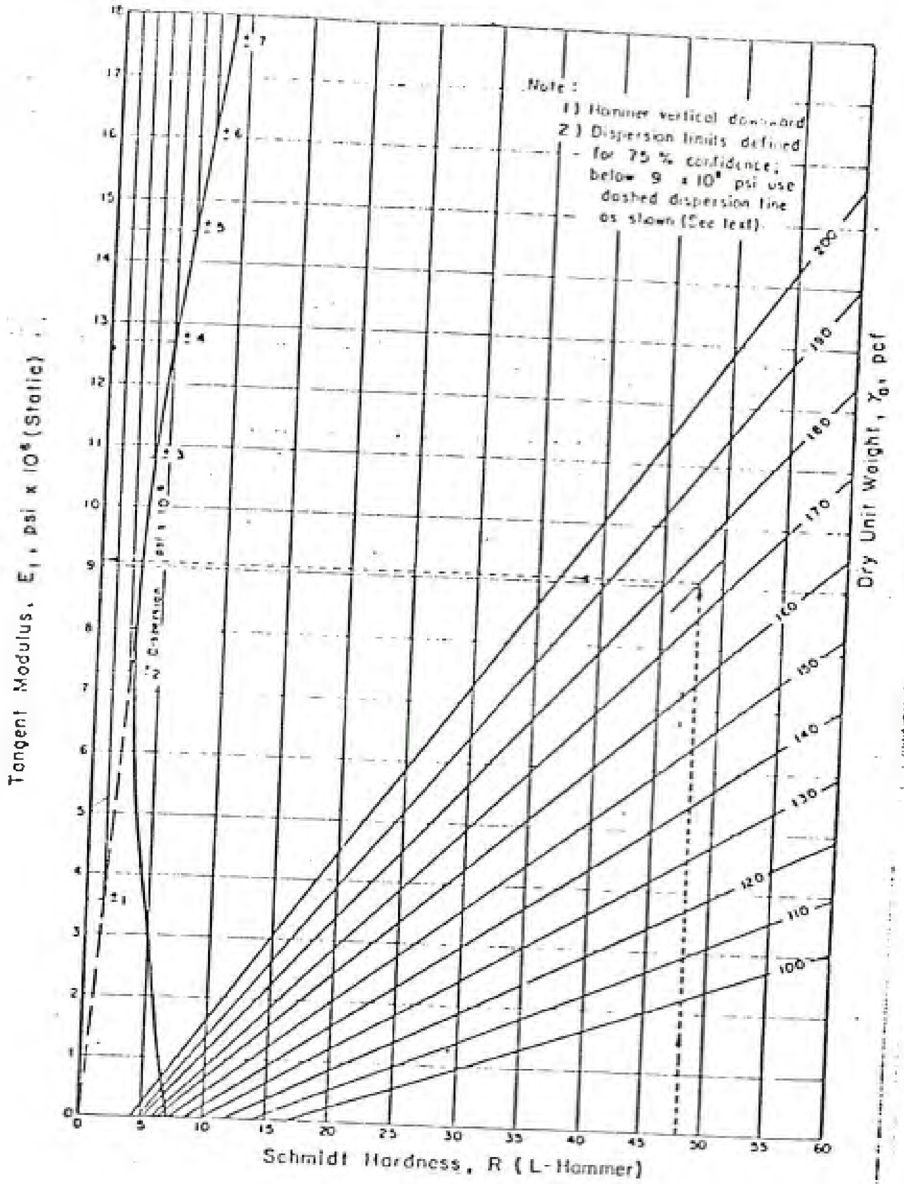


Figure 22. Nomograph for determining rock modulus using Schmidt hardness and dry unit weight (from Deere and Miller, 1966).

capacity based upon graphical relationships developed between bolt size and type, rock compressive strength, and embedment depth (Brackett and Parisi, 1975). These relationships are too extensive to present here; however, they are available from the referenced work. Since seafloor rock bolts must be installed by divers, inspection and sampling of the rock is possible at each site. The knowledge gained from such surveys facilitates estimations of the holding capacity.

Estimation of the holding capacity of a properly grouted anchor rod is also possible, provided the bond strength of the grout to the anchor rod or bar is known. Using the American Concrete Institute (ACI) standards for ultimate strength design (ACI, 1963), the bond stress, u_u, for normal deformed reinforcing bar (ASTM A 305) shall not exceed:

$$\frac{9.5 \sqrt{f_c'}}{D} \quad \text{nor } 800 \text{ psi}$$

where f_c' = compressive strength of grout (psi)
D = nominal diameter of bars (in)

Using the allowable ultimate bond stress, u_u, and the bar dimensions, the holding capacity, HC, may be calculated as:

$$HC = u_u \pi D L$$

where L = grouted length of bar.

By properly sizing the bar length, the holding capacity can be increased to the yield strength of the bar. By adding a plate or ferrule to the bar, the anchor can mobilize bearing strength to replace bond strength. In this manner, the bar may be shortened. The bond strength of the grout with the drill hole wall would become the limiting factor, as the anchor would tend to pull the grout plug out of the drilled hole.

The real variable in estimating the capacity of grouted anchors is the compressive strength, f_c', of the grout. Properly placed grout could have an f_c' of 5,000 psi; poorly placed grout may be completely useless. In addition to placement, the age of the grout will influence the compressive strength. The strength of concrete and grout gradually increases with age, rapidly for the first seven to ten days, then at a slower rate reaching a practical maximum after twenty-eight days. Studies of subaqueous concrete (Lorman, 1970) show that subaqueous concrete cures slower than concrete poured and cured under standard conditions. Thus, the use of grouted anchors may require considerable time for curing before full strength is developed.

Predicting the holding capacity of an embedment anchor is a more complicated matter. Holding capacity for a vertically-loaded anchor is obtained by friction between the fluke and the rock. Uplift alone is considered for two reasons. Following embedment, the downhaul cable leads directly to the surface through the penetration path; therefore, even if a load is applied at an angle, the rock will prevent the cable from pulling at any angle but up. Direct uplift is also used because it will be the worst case of loading.

However, little work has been done to determine the state of stress between the rock and the embedded projectile. It is the amount of induced stress that will determine the holding capacity in hard, elastic rock with a high RQD. During embedment, the rock is elastically deformed. At the end of embedment, the rock will strive to recover elastically and, depending on the strain rate during loading, viscoelastically. This recovery places a compressive stress upon the projectile. The holding capacity will be determined by this compressive stress, the stressed area of the anchor, and the coefficient of friction between the anchor and rock. Some adhesive effects may also occur. The outer skin of the Sandia penetrometers melted during penetration in rock, adhering to the surface of the penetration hole (Colp, 1968).

Thus, an equation can be formulated to determine the holding capacity (HC) as:

$$HC = \mu f'_c A_1 + aA_2$$

where μ = coefficient of friction
f'_c = compressive stress
A_1 = stressed area
a = unit adhesion
A_2 = adhesive area

The actual solution of this equation cannot be carried out at this time due to the lack of quantitative data from previous rock tests. An investigation of the rock before and after penetration is needed to determine the change in stress, possible changes in mechanical properties, and the separate effects of friction and adhesion.

If the rock is badly fractured or is too weak to resist loads through frictional resistance, the anchors in their present configuration are designed to rotate, digging the front and back ends of the fluke plate into the rock. However, there may be a tendency for the anchor to slide right back out the slot made during penetration. The MERDC anchor has small keying flaps at the trailing edge of the fluke to initiate rotation. In weak rock, this sort of device would dig into the rock with more dependability than a system relying solely on eccentric loading. It is interesting to note that,on every MERDC anchor recovered after tests in coral, the keying flaps had failed. This may indicate that the anchors were held in place largely by the action of the keying flaps and that when the flaps failed the anchor failed. The determination of holding capacity in soft or badly fractured rock will be extremely difficult. The holding capacity will vary widely, depending on how the anchor keys and upon the depth of rock where it finally does key. An estimation of holding capacity can then be made if the state of the rock above the keyed anchor is known. This solution would be similar to the determination of the holding capacity of the anchor in sediments.

RECOMMENDATIONS

Site Survey

1. The determination of geotechnical properties are of primary importance in selecting a method of anchoring. Samples of the seafloor rock should be taken for laboratory testing. If laboratory testing is not possible, the determination of Schmidt hardness and dry unit weight can be performed in the field and used to classify the rock based upon Deere and Miller's classification system.

Present methods of determining geotechnical properties of seafloor rock are limited to testing grab samples. Core drilling is possible but too expensive to be warranted in most cases. Improved methods of quickly determining the in-situ properties of seafloor rock would greatly aid the anchor selection process and improve the confidence of holding capacity predictions.

2. If an embedment anchor is to be used, it is recommended that the anchor site be accurately mapped and marked or visually inspected by divers or television camera to insure that no bathymetric hazards such as ledges or extreme slopes are present which could prevent proper embedment.

Anchor Selection

Following the classification of the seafloor rock, the choice of an anchoring method would be made. The final choice would depend largely on the operational requirements, but the following recommendations are based solely on the state-of-the-art capabilities of the anchors.

1. For coralline and soft rock seafloors, the embedment anchor is the most desirable method of anchoring.

2. For competent, hard rock in shallow water, the rock bolt is the most desirable method of anchoring. Installation is by divers and, so, is depth-limited.

3. For hard rock seafloors beyond diving depth, the drilled-in and grouted anchor is the best method of anchoring. The capability of drilling sockets into hard, exposed rock is also depth-limited. The exact depth limit would be difficult to define, but, in general, the drilled-in anchor technique is not practicable in ocean depths greater than 200 m.

4. For hard rock seafloors in depths beyond the capabilities of the rock bolt and drilled-in anchor techniques, the deadweight anchor is the only proven alternative. The limitations of the deadweight method are due to its low efficiency in resisting horizontal loads and its high instability on a slope.

Interim Projectile Selection Guidelines

The limitations of the various methods of anchoring in deep, exposed hard rock demonstrate the need for an effective rock anchor. The embedment anchor has been very effective in shallow-water basalt when the anchoring site was properly chosen. Land testing of the embedment anchor in typical oceanic basalts would allow an evaluation of these anchors as an alternative to more expensive or less reliable anchoring methods.

Land testing of the embedment anchor in rock is to be conducted in FY77 to determine what factors affect holding capacity. Until testing can be conducted to determine the aforementioned factors, a general guide of which embedment anchor projectile to use in various types of rocks may be outlined as follows.

1. In hard elastic rock, such as basalt, the use of the 20K anchor rock projectile should continue with some possible modifications in the fluke fin plates and downhaul connection. Shorter, lower aspect ratio fins are recommended to increase penetration. The original fins were provided for moment resistance, but smaller fins should adequately serve that purpose while reducing frontal area. The eccentric downhaul connection will be most useful when the projectile is completely embedded; however, no projectile fired into competent basalt thus far has done so. By narrowing the fins, some eccentricity is sacrificed but penetration is increased.

2. As the rock becomes weaker and less competent, the size of the fins should be increased. The present configuration should be suitable for weathered basalts and hard coral limestones.

3. In the weakest rocks, such as reef coral, a sand anchor projectile should be modified for use by increasing the nose taper, adding a keying flap or spur to the trailing end, and eliminating the connecting links to the downhaul by direct connection to the projectile as is done in the rock projectile.

Anchor Guideline Development

Future development of the various types of rock anchors will extend the capabilities of each anchor and will provide further alternatives to the mooring designer. General guidelines for anchoring in rock should be developed which will allow a rational procedure for choosing an anchoring system based upon the operational needs, the support available for installation, and the seafloor conditions.

REFERENCES

American Concrete Institute (1963). Building code requirements for reinforced concrete, ACI 318-63, Detroit, Michigan.

Attewell, P. B. (1962). Response of rocks to high velocity impact, Institution of Mining and Metallurgy, Transactions, London, Vol. 71, pp. 705-724.

Brackett, R. L., and Parisi, A. M., LT, CEC, USN (1975). Hand-held hydraulic rock drill and seafloor fasteners for use by divers, Civil Engineering Laboratory,Technical Report R-824, Port Hueneme, California.

Colp, J. L. (1968). Terradynamics: a study of projectile penetration of natural earth materials, Sandia Laboratories, SC-DR-68-215, Albuquerque, New Mexico.

Deere, D. U. (1964). Technical description of rock cores for engineering purposes, Rock Mechanics and Engineering Geology, Vol. 1, No. 1, pp. 17-22.

Deere D. U., and Miller, R. P. (1966). Engineering classification and index properties for intact rocks, Air Force Weapons Laboratory,Technical Report No. AFWL-TR-65-116, Kirtland AFB, New Mexico.

Distefano, C., and Boldan, P. (1965). Investigation of rock bolt anchorages in quartz monzonite rock-piledriver project, Omaha District U. S. Army Corps of Engineers, Rock Bolt Research Technical Report No. 1, Omaha, Nebraska.

Duke, M. C. (1944). Engineering properties of coral reef materials, Proceedings of American Society for Testing Materials, Vol. 49, pp. 964-976.

Dutta, P. K. (1972). A theory of percussive drill bit penetration, International Journal of Rock Mechanics and Mining Sciences, Vol. 9, No. 4, July 1972, pp. 543-567.

Heirtzler, J. R., and Bryan, W. B. (1975). The floor of the mid-Atlantic rift, Scientific American, Vol. 233, No. 2, August 1975.

Lindholm, U. S., Yeakley, L. W., and Nagy, A. (1974). The dynamic strength and fracture properties of Dresser basalt, International journal of Rock Mechanics and Mining Sciences, Vol. 11, No. 8, August 1974.

Lorman, W. R. (1970). In-situ strength of subaqueous concrete, Naval Civil Engineering Laboratory,Technical Report R-673, Port Hueneme, California.

MacDonald, G. A. (1967). Forms and structures of extrusive basaltic rocks, Basalts, edited by H. H. Hess and A. Poldervaart, Wiley Interscience, New York.

Maurer, W. C., and Rinehart, J. S. (1960). Impact crater formation in rock, Journal of Applied Physics, Vol. 31, No. 7, July 1960.

Mayo, H. C. (1973). Explosive embedment anchors for ship mooring, U. S. Army Mobility Equipment Research and Development Center, Report 2078, Fort Belvoir, Virginia.

Menard, H. W. (1964). Marine Geology of the Pacific, McGraw Hill, San Francisco, California.

Menard and Ladd (1963). Oceanic islands, seamounts, guyots, and atolls, The Sea, Vol. 3, edited by M. N. Hill, Wiley Interscience, New York.

Parisi, A. M., and Brackett, R. L. (1974). Development, test, and evaluation of an underwater grout-dispensing system for use by divers, Civil Engineering Laboratory, Technical Note N-1347, Port Hueneme, California.

Patterson, W. J. (1968). Physical properties and classification of seven types of rock targets, Sandia Laboratories, SC-TM-68-621, Albuquerque, New Mexico.

Patterson, W. J. (1972). Penetration of in-situ rock by air deliverable penetrators, Rapid Penetration of Terrestrial Materials, Proceedings of Conference at Texas A&M University, edited by John L. Colp, February 1972.

Pennington, J. V. (1974). Rock failure in percussion, Petroleum Engineer, Vol. 26, No. 5, May 1954.

Singh, M. M., and Hartman, H. L. (1961). Hypothesis for the mechanism of rock failure under impact, Proceedings of the 4th Symposium on Rock Mechanics, College of Mineral Industries, Pennsylvania State University, University Park, Pennsylvania.

Smith, J. E. (1971). Explosive anchor for salvage operations - progress and status, Naval Civil Engineering Laboratory, Technical Note N-1186, Port Hueneme, California.

Young, C. W. (1967). The development of empirical equations for predicting depth of an earth-penetrating projectile, Sandia Laboratories, SC-DR-67-60, Albuquerque, New Mexico.

BIBLIOGRAPHY

Appel, F. C., and Gatley, W. S. (1961). Ratio of loading effects in chisel impact, Proceedings of the 4th Symposium on Rock Mechanics, College of Mineral Industries, Pennsylvania State University, University Park, Pennsylvania.

Byerle, J. D. (1967). Theory of friction based on brittle facture, Journal of Applied Physics, Vol. 38, No. 7, June 1967.

Byerle, J. D. (1968). Brittle-ductile transition in rocks, Journal of Geophysical Research, Vol. 73, No. 14, July 1968.

Christians, J. A., and Meisburger, E. P. (1967). Development of multileg mooring system: phase A - explosive embedment anchor, U. S. Army Mobility Equipment Research and Development Center, Report 1909-A, Fort Belvoir, Virginia.

Christman, D. R., and Gehring, J. F. (1966). Analysis of high velocity projectile penetration mechanics, Journal of Applied Physics, Vol. 37, No. 4, March 1966.

Deere, D. U., Merritt, A. H., and Coon, R. F. (1969). Engineering classification of in-situ rock, Air Force Weapons Laboratory, Technical Report No. AFWL-TR-67-144, Kirtland AFB, New Mexico.

Farmer, J. W., and Atterwell, P. B. (1965). Rock penetration by high velocity water jet, International Journal of Rock Mechanics and Mining Sciences, Vol. 2, No. 2, pp. 135-153.

Kovacs, A., et al. (1975). On the theory of ground anchors, U. S. Army Cold Regions Research and Engineering Laboratory, Technical Report 258, Hanover, New Hampshire.

Lew, T. K. (1973). Deformation behavior and fracture mechanisms of rocks, Naval Civil Engineering Laboratory, Technical Report R-788, Port Hueneme, California.

Lindquist, R. G. (1974). Underground tests of the REAM method of rock fragmentation for high-speed tunneling, Proceedings of 1974 Rapid Excavation and Tunneling Conference, San Francisco, California.

MacDonald, K., Luyendyk, B. P., Mudie, J. D., and Speiss, F. W. (1975). Near bottom geophysical study of the mid-Atlantic ridge median valley near latitude 37°N: preliminary observations, University of California, San Diego, Marine Physical Laboratory, Scripps Institution of Oceanography, MPL-U-105/74, San Diego, California.

McCutchen, W. R. (1949). The behavior of rocks and rock masses in relation to military geology, Quarterly of the Colorado School of Mines, Vol. 44, No. 1, January 1949.

Rinehart, J. S. (1960). On fractures caused by explosions and impacts, Quarterly of the Colorado School of Mines, Vol. 55, No. 4, October 1960.

Schmid, W. C. (1969). Penetration of objects into the ocean bottom (the state of the art), Report to Naval Civil Engineering Laboratory under Contract No. N62399-68-C-0044, CR 69.030, Port Hueneme, California.

Shockey, D. A., et al. (1974). Fragmentation of rock under dynamic loads, International Journal of Rock Mechanics and Mining Sciences, Vol. 11, No. 8, August 1974.

Suh, S. L., Goldsmith, W., Sachman, J. L., and Taylor, R. L. (1974). Impact on a transversly anistrophic half-space, International Journal of Rock Mechanics and Mining Sciences, Vol. 11, No. 11, pp. 413-421.

Taylor, R. J., and Beard, R. M. (1973). Propellant actuated deep water anchor: interim report, Naval Civil Engineering Laboratory, Technical Note N-1282, Port Hueneme, California.

Tiseo, B. (1969). A laboratory apparatus for the determination of the anchorage capacity of rock bolts, U. S. Army Corps of Engineers, Missouri River Division Laboratory, Technical Report MRDL-2-69, Omaha, Nebraska.

Valent, P. J., Lee, H. J., True, D. G., and Malloy R. J. (1973). Feasibility study: sub-seafloor emplacement of nuclear waste, Naval Civil Engineering Laboratory (Port Hueneme, California) Report to Batelle Memorial Institute (Richland, Washington), AEC Purchase Order No. RL-73-6375, July 1973.

Young, C. W. (1966). Low velocity penetration study, Sandia Laboratories, SC-RR-66-118A, Albuquerque, New Mexico.

Young, C. W. (1967). Low velocity penetration study: second phase, Sandia Laboratories, SC-DR-66-543A, Albuquerque, New Mexico.

English to S.I. Conversion Factors

To Convert	To	Multiply By
inches (in)	millimeters (mm)	25.40
inches (in)	meters (m)	0.0254
feet (ft)	millimeters (mm)	305
feet (ft)	meters (m)	0.305
pounds (lb)	kilograms (kg)	0.453
pound-force (lbf)	newtons (N)	4.45
kips (K)	kilonewtons (kN)	4.45
foot-pounds (ft-lbf)	joules (J)	1.357
pounds per square inch (psi)	kilopascals (kPa)	6.9
pounds per cubic foot (pcf)	grams per cubic centimeter (g/cm^3)	0.016
feet per second (fps or ft/sec)	meters per second (m/s)	0.305

Note: Care should be taken in converting the penetration equations (Equations (1)-(4)), as the constants may be used only with English units. The best procedure to convert is to solve in English units and then convert the answer to S.I.