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FOUNDATIONS FOR SMALL SEAFLOOR INSTALLATIONS

By

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ABSTRACT

This report presents a procedure for the design of foundations for small non-manrated and non-strategic seafloor installations. These procedures are applicable only to installations with dimensions less than 15 feet and submerged weight less than 4000 pounds. They do not require a detailed analysis of the prospective site and are applicable to all seafloor sites, except those located on slopes greater than 10° and those in areas of rapid sediment accumulation, such as off mouths of large rivers. The report includes analyses of vertical and lateral loading and load resistance, tiedowns, use of materials, and foundation emplacement. Several typical foundation types and special features are described. Two example design problems are included to illustrate the design procedures.

CONTENTS

INTRODUCTION

Objective

The purpose of this report is to present foundation design procedures for small unmanned seafloor installations. This category includes instrument or sensor platforms, test structures, in situ test devices, and other totally submerged non-strategic structures having maximum dimensions of less than 15 feet, submerged weights of less than 4,000 pounds, and which are placed on the seafloor as a single unit. These guidelines are for structures placed in water depths greater than 30 feet, and for periods of from several hours to several years.

The information presented here is for those who must design and emplace such structures and whose background does not include a familiarity with seafloor foundation design principles. This information is not intended as, nor is it a reasonable substitute for, a complete foundation analysis which is required for structures of higher monetary or strategic value or of larger size or weight. Foundation analysis for such structures is discussed elsewhere (see, for example, References 1 through 10).

Background

Seafloor foundations can be separated into a number of categories (Reference 2) which range from small non-strategic structures, which are relatively insensitive to tilting, to large man-rated structures which are quite sensitive to very slight differential movement. There currently is no report available which is intended to provide guidance for the case of the small, non-strategic structures. These installations do not typically merit extensive site surveys or comprehensive foundation analyses. However, this group includes the vast majority of all installations currently being placed on the seafloor.

Of the hundreds of such foundations which are known to have been placed, a number have not performed in a totally satisfactory manner.[3] One of the primary causes for these problems was a lack of knowledge, at the time, concerning seafloor foundation design considerations and principles.[3] This report is intended to help satisfy the need for guidance in designing foundations for these installations.

Approach

A logical sequence is involved in designing a foundation. This report will generally follow this same sequence. The report begins with a discussion and description of seafloor foundations in order to provide a background for the reader and to establish certain definitions.

The first step in the design procedure is to define the installation in terms useful to the foundation engineer and to determine what "satisfactory performance" actually is in quantitative terms. The second step is the determination of the influential site properties and the evaluation of each. These first two steps are combined in this report under the section, "Design Conditions". This is followed by the sections covering actual design procedure, which is an iterative process in which a foundation configuration is selected and sized and then is checked for such parameters as adequate bearing capacity and resistance to lateral forces and overturning. An inadequacy in any one parameter requires a change in the size or configuration and a reiteration. The emplacement technique, which is discussed in a subsequent section, is considered in the same manner, since a large number of foundation performance difficulties have resulted from improper emplacement.[3] This report deals with only one class of installation; thus, a number of assumptions are made concerning the design considerations, site properties, and their interrelationships. These assumptions are necessary because little data will typically be available concerning sites for structures in this category. In all cases, the assumptions should be reasonable or conservative as "worst likely" situations are assumed. This approach also serves the purpose of simplifying the design procedure.

FOUNDATIONS

Physical Characteristics

The foundation is that portion of an installation which transfers the loads of the structure to the seafloor. Its design is largely dependent upon the properties of the underlying soil, or rock, and other environmental factors such as topography and bottom currents. Virtually all foundations for structures fitting into the category covered by this report, should utilize some form of a footing foundation. Figure 1 illustrates a multiple spread footing foundation. This particular foundation utilizes three footings, a determinate number which ensures even distribution of the load among the individual footings, and also has a reasonably large distance between the three to increase stability against overturning. The ball and socket joints allow the individual footing to articulate and thus to conform to the topography giving a more uniform distribution of bearing pressures on the seafloor.

The important physical characteristics of this system (structure and foundation) are illustrated in Figure 1. This particular foundation is designed for a soil bottom (sand, mud, or ooze) rather than rock. A foundation for rock could have a quite different appearance (see Figure 2i); however, it would have similar physical characteristics of importance. The symbols used to describe these characteristics are listed below and are defined on the List of Symbols at the back of the report. The numerical values for each are as follows:

W_{sub} = 3900 pounds

z_{csw} = 4 feet

W_{dry} = 5600 pounds

A_1 = 60.6 square feet

z_1 = 4.13 feet

A_v = 155.8 square feet

r_h = 6.69 feet

z_h = 11.42 feet

r_1 = 7.71 feet

r_2 = 7.42 feet

r_3 = 7.42 feet

r_{min} = 7.42 feet

$B_1 = B_2 = B_3$ = 6.0 feet

$A_1 = A_2 = A_3$ = 28.3 square feet

d_e = 0.25 feet

These parameters, all of which are determined during the foundation design procedure, have a major influence on the final performance of the installation. The nature of these influences, and thus, the logical selection of these parameters during the design phase, is discussed in the subsequent section.

Typical Configurations

A number of typical foundations which have been used on the seafloor are illustrated in Figure 2. The crossed strip footing (see Figure 2a) is used to support a small vertical instrument. The relatively wide spread of the foundation and its large weight (greater than that of the instrument) improve stability against overturning. The downturned leg of the angles acts as a keying edge and prevents any lateral movement. When this edge is embedded completely, the effective thickness of the footing is very small (less than 1/2 inch) and scour should not be a problem. The foundation is made of standard structural steel welded together. All members are oversized to allow for corrosion. The foundation is isolated by plastic from the instrument in order to prevent a galvanic couple and the associated accelerated corrosion.

The ring strip footing illustrated in Figure 2b is made of a plastic pipe (polyvinylchloride - PVC) with aluminum struts and has a steel anchor at the base. This large concentrated weight at the center of the base, the low weight of the plastic and aluminum, and the wide spread of the ring footing (12-foot diameter) all contribute to the stability against overturning once it is emplaced. In the case of this particular structure, the lifting point is only 3-1/2 inches above the center of the submerged weight (z_{csw} = 30.53 inches and z_h = 34.0 inches). This and the fact that the vertical projected area (A_v) is fairly large and the total submerged weight is small (W_{sub} = 360 pounds), increase the possibility of large rotations during lowering, and thus the chance of the structure being landed on its side, unless extreme care and a very slow lowering rate are used.

The multiple rigid spread footing foundation shown in Figure 2c utilizes a reasonable spread on the individual foundation elements and a fairly large submerged weight (W_{sub} = 4000 pounds) to resist overturning moments due to current drag. However, an inclined seafloor would cause problems for a system with a large height to the center of submerged weight (z_{csw} = 12 feet) such as this. Also, the rigid footings (maximum articulation, a = 0) would not be desirable if the microtopography, or surface roughness of the seafloor, were large. This system is made of welded standard structural steel which is painted.

Figures 2d, 2e, and 2f illustrate a single spread footing with a keying edge, the embedding length of which, d_e, is 4 inches. Figure 2d shows this footing elevated on dunnage on dry land. The holes in the upper surface of the footing are vents to allow the water entrapped by the keying edge to escape as the edge embeds. Figure 2e shows the same footing properly deployed on the seafloor in 600 feet of water. In this case, the footing is totally embedded except for the structural backing. This embedment reduces the possibility of scour caused by bottom currents or by the marine organisms that always seem to be attracted to installations deployed on the seafloor. Figure 2f is a later version of the same footing design. This version has a reduced effective footing edge thickness, t_e, because the outer structural angle has been eliminated (by using heavier members elsewhere). These spread footings are designed with plastic (PVC) in contact with the soil where corrosion rates are high and with welded aluminum, oversized to allow for corrosion, as the structural backing.

Figure 2g is a large spread footing (B = 14 feet) made of heavily reinforced concrete. A welded steel keying edge (with d_e = 10 inches) surrounds the perimeter. The horizontal edges of this footing are contoured in an attempt to streamline the footing to bottom currents and thus to lessen the chance of scour and undermining. Concrete was used in this case to provide additional weight to properly embed the keying edge.

Figure 2h illustrates a structure supported on two strip footings (each having B = 2 feet and L = 12 feet) made of aluminum. This structure is designed for recovery from deep water after deployment for up to several years. The footings are therefore attached with magnesium bolts, which are designed to corrode away rather rapidly upon exposure to the sea water so that the footings are not attached at the time of recovery, thus avoiding the problem of breakout associated with all soils except clean sands, gravels, and rock.

The last illustration on Figure 2 shows a multiple stubby pile foundation designed for a rock bottom. A small footing is built into, but located just above, the point of each pile. This is in case the rock is weak or covered by sand and the pile would have a tendency to penetrate. In either of these situations, these plates would act as small spread footings and prevent excessive penetration. Under normal circumstances, three such piles would be desirable to ensure equal loading of each, and thus prevent rocking of the foundation.

The nine foundations illustrated in Figure 2 are representative of the types of configurations now in use for seafloor foundations. A few of the nine illustrated are designed to support larger and heavier installations. Thus, some are larger or slightly more complex than may be required for the smaller of the installations being covered by this report. The design considerations would, however, be basically the same.

DESIGN

Design Conditions

The foundation design process starts with inputs from three areas: (a) data on the structure to be supported; (b) information describing the site; and (c) information on the limitations of the planned emplacement technique. The design process itself is basically an iterative process, or in many applications simply a trial and error procedure, in which a foundation is selected and sized and then checked for adequacy. If it falls short in any regard, it is modified and checked again until a satisfactory design results.

The input information on the structure includes the structure's dry and submerged weights (W_{sd} and W_{sw}), its maximum dimensions (x_{max}, y_{max}, z_{max}) and its maximum lateral projected area (A_l). These values will, of course, be different from those for the final installation which will include the foundation.

Knowledge of the sites for structures in this category usually consists only of the geographic location, the water depth, and the geologic province--including general slope. Information describing the geologic province (examples of geologic provinces include deep ocean basin, seamount top, and continental shelf) should indicate whether rock or sediment will be encountered on the seafloor. This determination has a large effect upon the foundation design. However, if it cannot be made, the foundation should be designed for sediment. Such a foundation will usually suffice for a rock bottom whereas the reverse situation would not be satisfactory. Knowledge of the general slope of the site, such as from bathymetric charts or surveys, is needed for calculations concerning overturning. Foundations in this category should not be located on slopes steeper than 10 degrees. Foundations for such slopes are possible; however, they require more detailed site investigations and analyses, which are beyond the scope of this report.

The expected emplacement capability influences the foundation design process primarily by limiting the spread of the foundation elements. Larger spread is desirable to increase stability against overturning; however, this generally increases the minimum lateral clearance radius (r_h), the vertical projected area (A_v), and the installation's weights (W_{dry} and W_{sub}), all of which may be limited by the available emplacement capability (ship, boom length, line working load, and winch capacity).

A foundation consisting of three articulated spread footings (similar to Figure 1a) is often the most desirable configuration, although not the least expensive. Either circular or square individual spread footings can be used. If the seafloor is expected to be relatively smooth (small microtopography and surface roughness) then a ring footing (similar to Figure 2b) or two strip footings (Figure 2h) would be equally satisfactory and simpler to fabricate. A single spread footing is usually less desirable because it offers less resistance to overturning in comparison to other types. However, it can often be the best selection from both deployment and ease of fabrication standpoints.

In designing the actual configuration it is desirable to locate the structure as low on the foundation as practical and to center it relative to the foundation element or elements, as nearly as possible with respect to submerged weight, mass, and resultant force from current drag. These three are most often not at the same point. In fact, it is typically difficult to determine these three precisely; however, an awareness of this consideration during the early design stages will usually improve the end product immensely. Once the general configuration has been determined, the individual elements can be sized.

Bearing Capacity

The bearing capacity of a seafloor soil is dependent upon the type and size of footing used and upon the engineering properties of the soil profile. These properties, normally determined through laboratory analysis of core samples or in situ testing, will often not be evaluated for the exact sites for structures in the category covered by this report. For this reason the available data representing the worst likely situations (Reference 11 includes an excellent collection of this type of data in unanalyzed form) was analyzed to determine a minimum likely soil strength profile. The worst likely situation was taken as the weak and compressible cohesive soils typical of the deep ocean floor and shallower basins and bays. The minimum likely was that strength profile which was exceeded by 90 percent of the data (95 percent or larger would have been desirable; however, there is not sufficient data to make the results of its use significant).

The foundation elements are sized on the basis of the total vertical force applied to the soil by the installation (primarily due to the submerged weight) or that portion applied to the individual foundation element (1/3 or 1/2 of the total in the most simplified cases with no lateral forces). The allowable loads which can be supported by strip footings and spread footings are shown on Figure 3, parts (a) and (b), respectively. These curves are based in part on bearing capacity equations and relationships from References 12, 13, and 14. The values from Figure 3 contain an adequate factor of safety and may, therefore, be used directly in the foundation design. These values are minimums applicable to all the world oceans except areas of very rapid deposition such as near the mouths of large rivers. Such areas require more detailed analysis (see for example Reference 1). When the bottom material is known to be a clean sand (such as that found on most beaches) the values of allowable bearing load may be doubled for all footings.

In sizing the foundation elements on the first trial, it is advisable to use twice the vertical force attributable to the submerged weight alone. This is to account roughly for the effects of overturning due to current drag and other factors. In the subsequent section on "Overturning" this approximation is checked and corrected. If the seafloor is known to be rock, bearing capacity, as such, will not be a problem. However, the overall dimensions of such a foundation or its elements may be sized as if it were for a clean sand bottom.

Lateral Forces

Lateral forces and effective lateral forces on an installation are caused, respectively, by current or surge drag forces and by the effect of a sloping seafloor. Drag forces are related to the maximum expected peak flow velocity, the area exposed to this flow, and the shape of the structure or structural members making up this area. Two worst likely situations are assumed here. For most locations in water depths greater than 400 feet, a peak velocity of 2.3 feet per second is assumed. For shallower water and for bay entrances, narrow passages, seamount tops, and similar areas of higher velocity, 5.1 feet per second is assumed. Resulting simplified relationships for calculating maximum drag force, based on References 15 and 16 are the following:

Most deeper water -

$$F_d \text{ (pounds)} = 12 * A_1 \text{ (square feet)} \tag{1}$$

Shallower water and areas of high currents -

$$F_d \text{ (pounds)} = 60 * A_1 \text{ (square feet)} \tag{2}$$

where A_1 is the maximum lateral projected area as defined earlier. For complex structures such as that shown in Figure 2h, A_1 can be taken simply as the entire area encompassed by the external members of the structure.

An installation located on a slope of inclination, i, relative to the horizontal, will have a component of its submerged weight acting parallel to the slope. This effective lateral force, F_i, is given by the following equation:

$$F_i = W_{sub} * \sin i \tag{3}$$

If the slope at the site is not known, either from bathymetric charts or a topographic survey, then a typical value for "i" can be assumed from the following list. Values are based partially on data from Shepard[17] and take into account typical surface roughness and microtopography.

Continental shelf (water depth less than 600 feet): i = 2 degrees

Continental slope (water depth between 600 feet and 4500 to 17,000 feet): i = 6 degrees

Deep ocean basins (water depth greater than 4500 to 17,000 feet): i = 2 degrees

For sites in the vicinity of seamounts, rises, hills, trenches, and similar seafloor features of large relief, generalizations concerning slopes can be dangerously misleading from a foundation design standpoint. The actual slope must be determined for each such individual site, and in many cases will be too steep for foundations described in this report.

In this design process the worst case is assumed: the effective lateral force resulting from the inclination of the foundation is assumed to act in the same direction as the drag force resulting from currents and surge. In order for the foundation to resist these lateral forces and to prevent the possibility of the installation skidding down the slope as a result, it is necessary that the installation, with or without keys, satisfy the following inequality

$$F_i + F_d \leq 0.5 * W_{sub} \tag{4}$$

If this is not the case, then the design should be changed (weight can be added or the area, A_1, reduced). Once inequality (4) is satisfied, it may still be necessary to consider the use of keys (see Figure 1b) on the base of the foundation. In general, it can be assumed that a properly proportioned foundation with no keys has an effective coefficient of friction of 0.1 with the soil. Thus, its lateral resistance, F_h, is given by the equation:

$$F_h \text{ (pounds)} = 0.1 * W_{sub} \text{ (pounds)} \tag{5}$$

If the bottom material is known to be clean sand or rock, a value of 0.3, rather than 0.1, may be used for the coefficient. If the sum of F_i and F_d is greater than F_h then keys must be used. For the general case (presumed not to be on rock or clean sand) perimeter keys should be made of material between 0.2 and 0.4 inches in thickness. Their required depth, d_e, is given by the equation:

$$d_e \text{ (feet)} = 0.17 + 0.05 * B \text{ (feet)} \tag{6}$$

When keys are used, the footing should have holes or other provisions to allow the water entrapped by the keying edge during emplacement to escape. If it is known that the footing will be placed on clean sand and that keys are required, the key depth, d_e, should be reduced to 1/3 the value calculated from Equation 6. On rock a number of pointed bars 1/2-inch in diameter extending 3 to 6 inches below the footing, spaced every 6 inches to 1 foot around its perimeter, and pointing slightly outward (15 to 30 degrees relative to vertical) make an effective design.

Overturning

Lateral forces can cause two additional problems, simple overturning of an installation or excessive bearing pressures on the downhill or downcurrent side of a structure. The maximum overturning moment, M_d, is given by the equation:

$$M_d \text{ (foot-pounds)} = z_1 \text{ (feet)} * F_d \text{ (pounds)} + z_{csw} \text{ (feet)} * F_i \text{ (pounds)} \qquad (7)$$

As a general rule the following inequality should be satisfied:

$$M_d \leq 1/3 * W_{sub} * r_{min}$$

where r_{min}: for a multiple spread footing foundation is the smallest of r_1, r_2, and r_3; for a strip footing it is the lesser of L/3 and D-B/2 (see legend on Figure 3); for a crossed strip footing it equals L/3; for a ring footing it equals 0.35 * D; and for a single spread footing it equals 0.29 * D. If this equation is not satisfied then consideration should be given to increasing r_{min} or reducing M_d by shortening the structure or by similar structural changes. In calculating the actual vertical force for which a footing must be designed, it is necessary to take into account the effect of this overturning moment. The following equations give the total vertical force, F, from which foundations, or foundation elements, may be sized.

For an individual footing of a foundation with three spread footings

$$F \text{ (pounds)} = 1/3 * W_{sub} + M_d \div r_{min} \qquad (8)$$

Force per foot on a ring footing

$$F \text{ (pounds per foot)} = W_{sub} \div L + 7 * M_d \div D^2 \qquad (9)$$

Force on two parallel strip footings

$$F \text{ (pounds per foot)} = W_{sub} \div (2 * L) + (2 * M_d) \div (r_{min} * L) \qquad (10)$$

Force on crossed strip footings

$$F \text{ (pounds per foot)} = W_{sub} \div (2 * L - B) + (12 * M_d) \div L^2 \qquad (11)$$

Force for a single spread footing foundation

$$F\ (\text{pounds}) = W_{sub} + (32 * M_d) \div B \tag{12}$$

The maximum force, F, calculated from the appropriate equation (8, 9, 10, 11, or 12 above) is used in conjunction with Figure 3 to size the footing. In all of these equations values of either "D" or "B" are required to solve the equation. Trial values of each should be selected, tested with the equation and Figure 3, and then adjusted and rechecked if necessary.

For foundations on rock the bearing elements may be open frames, crossed angles (such as in Figure 2a) or similar configurations. The overall dimensions may be sized as if one were designing a foundation on clean sand. Most rocky areas are rather irregular, and for this reason the following criterion is suggested:

$$r_{min} \geq (2 * M_d) \div W_{sub} \tag{13}$$

In general, the most satisfactory foundation configuration on rock is a tripodal arrangement of three foundation elements, sized and configured as discussed above. A ring footing configuration of large overall diameter, D, and small element width, B, has also been successfully used.

Tiedowns

In some cases, particularly in shallow water, it can be difficult to overcome the large lateral forces and the resulting overturning moments by simply adding weight. For such situations small tiedowns can be used to develop additional anchorage capacity. Where this approach is selected it is necessary to use either three or four tie-down points, each located near the outside edge or end of a foundation element or, for the case of a single element foundation, spaced evenly around the perimeter. The design capacity, F_a, required of each tiedown to resist the overturning moment, is given by the following equation:

$$F_a = M_d \div r_{min} \tag{14}$$

where r_{min} equals the minimum lateral distance from the CSW to a tiedown.

A number of such tiedown anchor configurations are illustrated in Figure 4. These are reasonable solutions only in diver depths since from a practical standpoint divers are required for their installation. Other work systems could be used at deeper locations; however, these are typically quite expensive compared to divers.

For locations on a relatively clean sand either a jetted-in anchor or a screw anchor is useful. The jetted-in anchor[18] shown in Figure 4a has a 6-inch fluke diameter. The shaft and hose diameter are usually dependent upon available equipment. Embedment of the fluke to a depth of 8 feet and rodding to densify the sand as it is backfilled will give a design pullout capacity of 500 pounds. To achieve this capacity, it is extremely important that the sand which is backfilled into the hole created by the jetting, be compacted to a dense state. Rodding (repetitiously thrusting and removing a 10-foot length of 3/4-inch-diameter pipe, or similar, into the sand as it is filled into the hole) is one practical means.

A second type of anchor which can be used in either sand or cohesive soils (clays, silts, muds, and oozes) is a screw anchor. Figure 4b shows a single-flight type which can be installed by a diver. When this 8-inch-diameter screw is embedded 4 feet in cohesive soils, a design capacity of 120 pounds is achieved. In sands a 4-inch-diameter can be used and only 2 feet of embedment is necessary to develop a 240-pound design capacity. In order to assure these capacities, these screw-in anchors should always be torqued to the diver's maximum capability, which may result in slightly larger penetrations at some locations. Higher capacities can be achieved by designing with larger diameter anchors and deeper embedment, however, the torque required during installation in either case would usually be beyond a diver's capability.

Another configuration which can be used in cohesive soils is a simple mini-pile, Figure 4c. A usable design is a 2-inch-diameter pipe, an open end is satisfactory, about 8 feet in length. This can be driven with an 18-inch-length of larger diameter pipe with a plate welded on one end. This drop hammer driver should weigh about 20 pounds. Simply driving the mini-pile until the penetration per blow is less than 1/4 inch will give a design capacity of 120 pounds. Penetrations will typically be 8 feet or less.

Explosive embedment anchors, Figure 4d, can be used where large capacities are required and other anchor types either singly or in groups, are not satisfactory. Reference 19 discusses their use and capabilities.

For a rock bottom, none of the previously mentioned configurations are directly applicable (one exception is the explosive embedment anchor, several variations of which have recently been developed for rock). One acceptable tiedown system on rock is the rock bolt, Figure 4e. Various types are available; however, most require the drilling of a hole in the rock with a diameter of from 1/4 to 1 inch and to a depth of from 2 to 10 inches. Design load capacities of these range from 100 to 2000 pounds when properly installed in sound rock. Values in coral may be somewhat less. Another technique, Figure 4f, involves the drilling of a 1 to 2-1/2 inch diameter hole from 6 to 48 inches into rock. A steel rod, such as a standard concrete reinforcing bar, is positioned in this hole which is then backfilled with a cement grout. Load capacities of up to 2000 pounds have been reported utilizing this technique. More detailed information on capacities, techniques, and equipment is available in Reference 20. Attention must be paid to prevention of corrosion, particularly in typically highly stressed rock bolts.

Foundation Materials

For structures in this category it is generally recommended that unusual materials be avoided and that standard structural steel, aluminum (Type 5086), concrete, or plastic (Polyvinylchloride - PVC) be used. Oversized members, from a stress analysis design standpoint, and large rigid connections are suggested. Where it is necessary to have a flexible joint, it should be encased in a flexible jacket and the jacket filled with oil and sealed. Many such joints requiring flexibility for only a short period of time may not require oil-filled jackets. For example, flexibility may be required only until the foundation is resting on the seafloor and the flexible joints have allowed individual footings to adjust to the local irregular topography. For this case, exposed flexible joints may be used, by the time corrosion or fouling locks up the joint, all required movement will have been completed.

A foundation made of standard structural steel with welded connections is a practical solution. Where additional weight is needed in the foundation, in order to resist lateral forces, concrete can be added. A dense, high-strength concrete, mixed from sulfate-resisting cement and sound aggregate is most reliable. The concrete should provide three inches of cover over any reinforcing used in order to minimize corrosion. Where weight must be reduced or minimized, a welded aluminum or plastic foundation is suggested. PVC plastic has very low weight in water and offers yield strengths as high as 9,000 pounds per square inch. It is however more expensive than aluminum.

Both steel and aluminum (Type 5086 only) are recommended because both have fairly uniform rates of corrosion. Protective coatings can be used; however, these are typically scratched during routine handling and deployment. Any scratched area will be subject to local accelerated corrosion. In general, it is recommended that oversized sections (thicknesses increased by 1/4 to 1/2 inch over what is required by the stress analysis) be used, and corrosion be allowed. Cathodic protection is helpful, but it must be properly designed. Contact of dissimilar metals cannot be allowed, except in the case of sacrificial anodes. If such connections are otherwise necessary, the two materials must be isolated from each other with plastics or similar materials. Corrosion rates for metals in direct contact with the seafloor are often accelerated by a factor of two or three.

In some design situations, it may be necessary to reduce the in-water weight of an installation below what is possible by careful selection of materials. For these cases, buoyancy can be added. In deep water either syntactic foam or glass spheres can be used. The latter are usually less expensive; however, if several are required in close proximity, there is the possibility of sympathetic implosion of all if one should implode. In water depths less than about 1000 feet, buoyancy tanks (larger pressure-resistant structures filled with air) and less expensive foams are practical. In general, however, reduction of total submerged weight is not as practical as redesigning a foundation to handle the load. The addition of buoyancy increases the volume of the structure and, therefore, the lateral loads. It also increases the dry weight and mass of the structure. This generally increases the difficulty of handling and emplacement.

A third consideration concerns installations which are just slightly negatively buoyant. Installations should not be designed with an effective total density approaching that of seawater because sediment in suspension in the seawater can increase its effective unit weight. The maximum effective fluid densities possible in a sediment cloud are not known; however, during routine laboratory soils experiments, increases of 6 percent are regularly created and can endure for hours with no outside agitation. Such an increase in fluid density would have a tremendous influence on the submerged weight of an installation which was nearly neutrally buoyant in clear seawater. In some extreme cases it would be possible that an installation, which was just slightly negatively buoyant in clear seawater, would be simply floated off in such a sediment cloud.

Foundation Emplacement Considerations

This section is included for two reasons. First, a significant percentage of foundation performance problems are caused by improper emplacement, and second, foundation design is often seriously restricted by considerations of available handling and emplacement methods. These considerations, their effects upon the foundation design, and other considerations in foundation emplacement are discussed in the general order of their occurrence in an actual emplacement operation.

The installation, foundation and structure together, must be designed with a lifting point, or points for a sling, which will provide the proper orientation of the installation in water. An installation will often not have precisely the proper orientation when suspended by this point, or points, in air. This is satisfactory as long as it is stable in air. This lifting point should be well above the center of gravity of the installation, both in air and in water. A number of points should be provided for attaching securing lines during shipment and for tag lines during handling over-the-side. The foundation must rest on the deck of a ship before it rests on the seafloor. This often necessitates the use of special blocking or a second support system since installations are often much heavier in air.

In handling the installation over-the-side, it is preferable to use as short a suspension length as possible with sufficient vertical clearance to get the installation over the side rails. Articulated hydraulic booms are very effective. It is also necessary to have sufficient reach to prevent the installation's swinging into the side of the ship. A factor of safety in the handling system of at least 8 relative to the dry weight of the installation is necessary. The handling of the installation over-the-side should be as quick and smooth an operation as possible to prevent damage. The water will quickly damp out any swinging motion. Once in the water the installation should be quickly lowered well below the surface, to a depth of 50 feet or deeper if possible, to reduce dynamic line tensions and to prevent unanticipated pick-up of the installation with trapped water resulting from the ship's motion. Such a pick-up can cause severe loading on handling equipment. Installations should be designed to trap a minimum of water as it is sometimes necessary to recover them with the placement equipment. For installations with large vertically projected area, relative to their maximum laterally projected area, it is sometimes desirable to handle these through the sea-air interface on their side in order to reduce the large drag forces induced by ship's motion.

Installations are often handled over-the-side with one handling system and then transferred to a winch system for lowering to the bottom. This generally cuts down on the complexity of on-deck rigging and control, and induced line stress from ship's motion. This transfer of load can be made remotely or by divers working at depths to 70 feet (deeper is possible). For deep water deployments, synthetic lines offer the capability to handle shock loadings and have the advantage of lower line loads, partially due to very low in-water weight of payed out line. Typically they also lessen the possibility of line entanglement with the installation after release on the seafloor.

The maximum lowering rate available from many pieces of deck equipment is about 2 feet per second. This can be too fast for many installations since the lowering rate cannot exceed their free-fall velocity. If it were exceeded, this could result in the installations becoming entangled in the lowering line or other complications. A rough rule for maximum lowering rate is as follows:

$$V_{max} \text{ (feet per second)} = 1/4 \sqrt{W_{sub} \text{ (pounds)} \div A_v \text{ (square feet)}} \qquad (15)$$

On rougher days when ship's motion is significant, the rate should be reduced below the calculated V_{max}.

As an installation is lowered, its lateral position will be subject to some excursion relative to the ship's position. Assuming that the ship is maintaining station or attempting to do so, the maximum possible magnitude of this excursion can be approximated by the following two inequalities.

For shallow water or locations of very high currents -

$$r \leq 50 * A_1 * \frac{d}{W_{sub}} \tag{16}$$

For most other locations with water depths greater than 300 feet -

$$r \leq 10 * A_1 * \frac{d}{W_{sub}} \tag{17}$$

These equations will give an approximate maximum for all cases except where an oversized (from a loading standpoint) synthetic line is used. Normally the excursion will be much less than the magnitudes indicated above, however, these equations indicate what is possible where large currents are present.

As the installation approaches the seafloor it is necessary to reduce the lowering rate by at least a factor of four. The reason is to prevent impact with the bottom which can result in bearing failures. A high rate of approach also causes flat footings and similarly shaped installations to skate about laterally as they are lowered. This would result in a foundation being landed with some lateral velocity which typically causes improper emplacement, including initial inclinations as large as 15°.

Once bottom contact is made, payout of line should be ceased in order to prevent entanglement, and any checks for proper emplacement should be completed as quickly as possible. Then, the installation should be released and the lowering line moved clear to prevent entanglement with the installation. The means for releasing may be electro/mechanical, acoustic/mechanical, explosive, trip weight actuated, bottom contact actuated, or some other. If two lines are used, one load bearing and the other electrical, they should either be separated by a large distance at the surface, or secured to each other with allowances made for relative elasticities. Under normal circumstances two lowering lines should not be used.

If a foundation is to be recovered after deployment, it may be necessary to take into account the breakout force required to remove the foundation from a cohesive sediment (see Reference 21). This force can be several times larger than the installation's submerged weight even when it is removed slowly. In order to reduce the magnitude of this required force it is suggested that the recovery line be attached eccentrically to the foundation--possibly on the edge or corner of a footing. Rapid breakout by brute force should be avoided; rather, a load two or three times larger than the installation's submerged weight should be applied and maintained until breakout is achieved--usually a matter of minutes or tens of minutes for larger installations. Breakaway footings (such as that illustrated on Figure 2h) can be used to avoid the problem. These footings are designed with rapidly corroding connections such that after a period of several months the connection is sufficiently weakened that a tension between the structure and the individual footing will cause separation. This connection must be designed to handle the full design compression and shear loads until separation is effected.

EXAMPLE PROBLEM

The two example problems which follow are representative of actual situations. The complete design procedures are presented to illustrate the iterative process involved. During this process as various individual parameters are checked, found to be inadequate, and modified, it is necessary to revalue other parameters which are interrelated. In the example problems it may be noticed that revaluations for all interrelated parameters are not always illustrated after each modification. Where the resulting change would be small, the revaluing is omitted.

Design Example One

The first design example is an installation in deep water. The structure is part of an experiment to measure ambient noise in the deep ocean. It consists of a triangular platform supporting a radioisotope power source, a bale of line and assorted equipment for use in eventual recovery, several acoustic sensors, and five pressure-resistant housings of various shapes and sizes containing data conditioning, analysis, storage, and acoustic telemetry equipment. The characteristics of the structure are as follows:

x_{max} = 10 feet

y_{max} = 8.7 feet

z_{max} = 6.5 feet

A_1 = 34.5 square feet

A_v = 44 square feet

z_1 = 1.5 feet

W_{sd} = 6,200 pounds

W_{sw} = 3,600 pounds

The structure is to be placed in a deep ocean basin at a water depth of 12,000 feet. The maximum overall slope in the vicinity of the site, as determined from detailed bathymetric charts of the area, is 2 degrees. The microtopography and sediment type were not investigated directly; however, in this geologic province a weak cohesive sediment type and small microtopography would be expected.

The structure is designed for a deployment of five years. The emplacement capability consists of a 200-foot-long ocean-going Navy tug with a special lowering winch aboard. The load capacity of the boom system is limiting and has a maximum working load of 10,000 pounds. Vertical clearance is 22 feet and outboard reach is 14 feet. Divers will be available to assist with the transfer of the load from the boom system to the synthetic line from the winch. Release of the installation on the seafloor will be accomplished by an acoustically-actuated mechanical release. Recovery of the structure at the end of its deployment will be accomplished by running a heavier recovery line down a guideline which is either released from the structure and rises to the surface or is attached by a submersible.

Configuration Selection

This structure, because of its mission, must sit firmly on the seafloor--no rocking can be tolerated. This and the possibility of minor surface irregularity, or microtopography, suggest the use of a multiple, spread footing foundation with articulated individual footings. A life of five years is required; therefore, the use of aluminum and plastic or concrete would be recommended for this case. More exotic materials and coatings could be used, but the initial cost would be much higher.

Bearing Capacity

Assume initially that the submerged weight of the foundation will be 500 pounds.

W_{sub} = 4100 pounds

F_t = 8200 pounds for initial sizing of the footings

$F = 8200 \div 3 = 2733$ pounds

F_{all} = 3072 pounds for B = 8.0 feet (Figure 3b)

Check assumption on weight

$W_{sub} = 3600 + 3 * ((\pi * B^2 * 1/4 * 2) + (4 * B * 6) * 1.5) = 4770$ pounds

$F = 2 * 4770 \div 3 = 3180$ pounds, which is close enough for this preliminary trial

Lateral Force

$F_d = 12 * A_1 = 410$ pounds, A_1 of the foundation is negligible (1)[+]

$F_i = W_{sub} * \tan i = 4770 * \tan 2^o = 167$ pounds (3)

Checking lateral load

$F_i = F_d \leq 1/2 * W_{sub}$ (4)

$577 \leq 2385$, which is satisfactory

Checking for keys

$F_h = 0.1 * W_{sub} = 477$ pounds (5)

$F_i + F_d = 577 > 477$, therefore keys are required

$d_e = .17 + 0.05 * B = 0.57$ feet (6)

Overturning

Assuming the base of the structure is 2 feet above the plane of the footings,

$z_1 = 3.5$ feet

$z_{csw} = 3.1$ feet

$M_d = z_L * F_d + z_{csw} * F_i = 1550$ foot-pounds (7)

Placing the centers of the footings at the corners of the structure platform gives,

$r_{min} = 5.8$ feet

$F = 1/3 * W_{sub} + M_d \div r_{min} = 1930$ pounds (8)

[+]Numbers in parentheses reference the applicable equation from preceeding sections.

This is much smaller than assumed when the footings were first sized; therefore, B can be reduced.

For B = 7.0 feet,

F_{all} = 2105 pounds, which is satisfactory.

Use B = 7.0 feet.

Check required keying edge depth,

$d_e = 0.17 + 0.05 * B = 0.52$ feet (6)

Total submerged weight will also be reduced but likely not sufficiently to affect this design.

<u>Emplacement</u>

Maximum lowering rate for this installation,

$A_v = 44 + 2.5 * (3.5)^2 * 3.14 = 140$ square feet

$V_{max} = 1/4 * \sqrt{W_{sub} \div A_v} = 1.5$ feet per second (15)

This means that it will take at least 2 1/4 hours to lower the installation to the seafloor.

Maximum lateral excursion,

$r = 10 * A_l * d \div W_{sub} = 1450$ feet (17)

Handling characteristics,

r_h = 6.4 feet, ship has limitation of 14 feet, therefore, satisfactory.

z_h = 10.6 feet, have 22 foot limitation, therefore, satisfactory.

W_{sub} = 4770 pounds

W_{dry} = 6200 + 2550 = 8750 pounds, have limitation of 10,000 pounds, therefore, satisfactory, but since this is so close it would be advisable to check the handling system to assure its rated capacity

Recovery

Since this installation will presumably be located on a cohesive sediment, large breakout forces can be expected during recovery. The total force required at the seafloor during recovery could easily be of the order of 15,000 pounds. One means of reducing this would be to use breakaway footings.

Design Example Two

The second example design problem is a small structure to be deployed at a water depth of 70 feet in a bay near the entrance. The structure is an environmental monitoring and recording instrument which is to be deployed for 3 months to collect data on bottom current velocity and direction, tide, water temperature, and salinity. The information is to be used eventually in the design of a large permanent mooring. The instrument is quite small; however, it is necessary to "design" the foundation for two reasons. First, the economic cost of a foundation failure would be quite high in terms of expended effort and lost data. Second, as will be seen, the required foundation is often quite different from what one might expect. The use of a "typical sized foundation" for this case would likely lead to a foundation failure.

The maximum slope of the site as determined from bathymetric charts is 6 degrees. Diver observations of the bottom at the site indicate very small microtopography and a clean sand bottom. The latter observation was confirmed by samples. The available emplacement and recovery capability consists of a crane barge which can handle up to 10 tons directly to the bottom. Vertical clearance going over the side is 30 feet and clear outboard reach is 22 feet. Divers can work off this barge and will be available for both emplacement and recovery.

The structure consists simply of a vertical cylindrical case, the lower 18 inches of which is an open frame containing various sensors. The characteristics of the structure are as follows:

$x_{max} = y_{max} = 1$ foot

$z_{max} = 6.83$ feet

$A_1 = 6.83$ square feet

$W_{sd} = 350$ pounds

$W_{sw} = 135$ pounds

The three most important considerations in selecting a foundation configuration in this case are the large current drag forces at this location, the lack of significant micro-relief, and the relatively short duration of the deployment. The least expensive type of foundation would be a single spread footing, but it is not as efficient in resisting the expected large overturning moments as is a crossed strip footing. The latter is almost as inexpensive to fabricate and since there are insignificant limitations imposed by the available emplacement capability, the crossed footing configuration will be tried. Since the deployment is for a relatively short period of time and additional weight in the foundation will be useful in resisting the effects of the current drag force, standard structural steel will be selected for the material.

Bearing Capacity

Assume initially a foundation with a submerged weight of 500 pounds.

$W_{sub} = 135 + 500 = 635$ pounds

Doubling this load to initially size the footings,

$F = 1270$ pounds

Try $B = 1.5$ feet,

$F_{all} = 24$ pounds per foot, or for clean sand 48 pounds per foot (Figure 3a)

$2 * L - B = F \div F_{all} = 26.5$ feet

$L = 14$ feet

Checking the assumed foundation weight, the submerged weight of steel plate 1.5 feet wide by 5/16 inch thick with two 1-inch stiffners is 18.5 pounds per foot.
Total weight would equal,

$F_t = (2 * L - B) * 18.5 = 490$ pounds, which is satisfactory.

Lateral Forces

Drag force due to bottom current,

$F_d = 60 * A_1$ (2)

Since the lateral area of the foundation is negligible, A_1 for the installation equals 6.83 square feet.

$$F_d = 60 * 6.83 = 410 \text{ pounds}$$

Effective lateral force due to sloping bottom,

$$F_i = W_{sub} * \tan i = (135 + 500) * \tan 6^o = 67 \text{ pounds} \quad (3)$$

Checking that,

$$F_i + F_D \leq 1/2 * W_{sub} \quad (4)$$

$$W_{sub} \geq 2 * (67 + 410) = 954 \text{ pounds, therefore, must add weight}$$

Try simply thickening strip footings, use 9/16 inch,

$$W_{sub} = 135 + (2 * L - B) * 34 = 1040 \text{ pounds}$$

Checking

$$F_i = 1040 * \tan 6^o = 109 \quad (3)$$

$$F_i + F_d = 519 \leq 1/2 * 1040, \text{ which is satisfactory} \quad (4)$$

Check for keys, for sand,

$$F_h = 0.3 * 1040 = 312 \text{ pounds} \quad (5)$$

$$F_i + F_d = 519 > 312, \text{ therefore need keys}$$

$$d_e = 0.17 + 0.05 * B = 0.25 \text{ feet}$$

For sand this value is reduced by a factor of 3; therefore, the required depth of the keying edge equals 1 inch.

Overturning Moment

For this installation, the average height of the area projected laterally,

$$z_1 = 1/2 * z_{max} = 3.42 \text{ feet}$$

The vertical height to the center of submerged weight,

$$z_{csw} = ((3.42) * (135) + (0) * (905)) \div 1040 = 0.44 \text{ feet}$$

$$M_d = z_1 * F_d + z_{csw} * F_i = (3.42) * (410) + (0.44) * (109) = 1450 \text{ foot-pounds} \quad (7)$$

For a crossed strip footing

$$F = W_{sub} \div (2 * L - B) + (12 * M_d) \div L^2 = 128 \text{ pounds per foot} \quad (11)$$

Allowable for B = 1.5 feet is only 48 pounds per foot (from Figure 3a); therefore, increase B while holding W_{sub} constant or use tiedowns.

<u>Solution 1 - Using Tiedowns</u>

A tiedown would be required on each end of both footings and would need a capacity of,

$$F_a = M_d \div r_{min} = 206 \text{ pounds} \quad (14)$$

For a sand bottom, a screw anchor with a 4-inch diameter blade, embedded at least 2 feet would be adequate.

<u>Solution 2 - Increasing B and Using No Tiedowns</u>

The required F is 128 pounds per foot. From Figure 3a, F_{all} for B = 2.8 feet is 130 pounds per foot for clean sand.

Try B = 2.8 feet and plate thickness equals 5/16 inch with four 1-1/4 inch stiffners:

$$W_{sub} = 135 + (2 * L - B) * 36 = 1045 \text{ pounds}$$

Lateral resistance is satisfactory since W_{sub} has not changed significantly.

Check overturning

$$F = 1045 \div (2 * 14 - 2.8) + (12 * 1450) \div (14)^2 = 130 \text{ pounds per foot} \quad (11)$$

Since F_{all} = 130 pounds per foot for clean sand, this is satisfactory. Recalculate keying edge,

$$d_e = 0.17 + 2.8 * 0.05 = 0.31 \text{ feet}$$

For clean sand reduce this by a factor of three. Therefore, d_e = 0.124 feet or 1-1/4 inches.

Emplacement

Maximum lowering rate, for the installation,

$$A_v = (2 * L - B) * B = 70 \text{ square feet}$$

$$V_{max} = 1/4 * \sqrt{W_{sub} \div A_v} = 0.95 \text{ feet per second} \quad (15)$$

Maximum lateral excursion during lowering

$$r = 50 * A_1 * d \div W_{sub} = 23 \text{ feet} \quad (16)$$

Handling characteristics

$$r_h = 5.6 \text{ feet}$$

$$z_h = 7 \text{ feet}$$

$$W_{sub} = 1045 \text{ pounds}$$

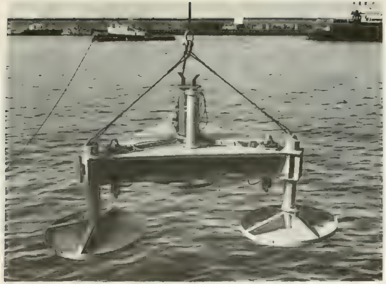
$$W_{dry} = (910) * (1.14) + 350 = 1390 \text{ pounds}$$

Recovery

Since the site is clean sand, there will be no breakout problem once the tiedowns, if used, are disconnected.

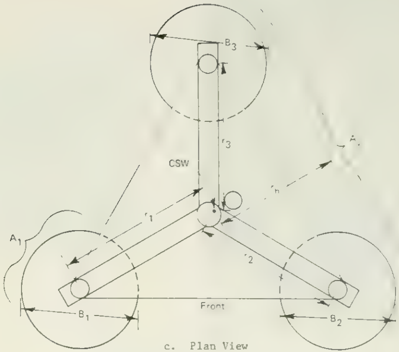
SUMMARY

The design procedures presented herein, and exemplified by the two preceding design examples, may be used for all non-manrated, non-strategic, small seafloor structures which are to be located on a slope of less than 10 degrees and which are not located in an area of very rapid deposition such as near the mouth of a large river. In many cases the guidelines presented here will provide an overdesigned foundation (conservative design). However, this determination can only be made where more detailed data on the site are available. When this is the case, and parameters such as the sediment strength profile or maximum bottom current need not be conservatively assumed, more precise rules of design can be applied. These procedures are summarized elsewhere.[1] The design procedures outlined here should give a reliability of 0.95 or slightly higher.

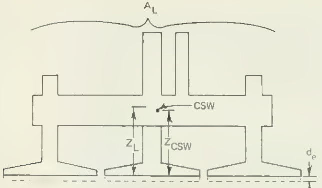


a. Structure on Multiple Spread Footing Foundation

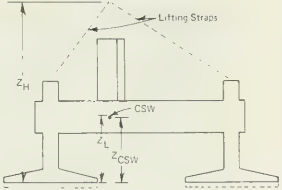
b. Footing Underside Showing Keying Edge



c. Plan View



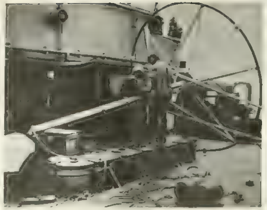
d. Front View



e. Side View

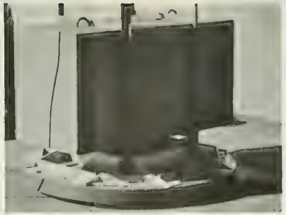
Figure 1. Example Spread Footing Foundation

a. Crossed Strip Footing



b. Ring Strip Footing

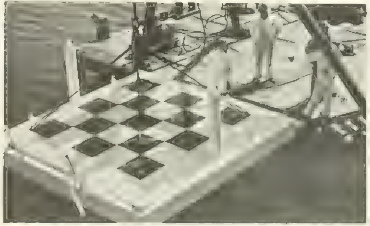
c. Multiple Spread Footing



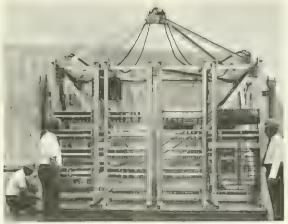
d. Circular Spread Footing with Keying Edge

e. Circular Spread Footing Properly Emplaced

f. Circular Spread Footing with Reduced Effective Thickness



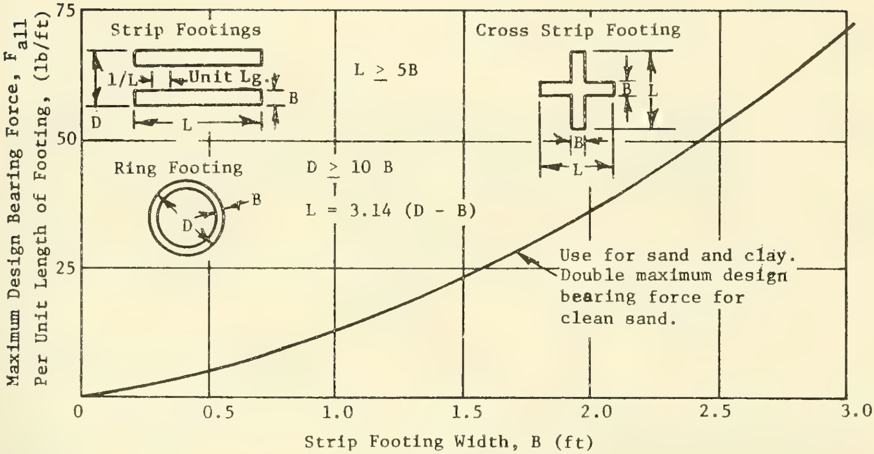
g. Large Spread Footing



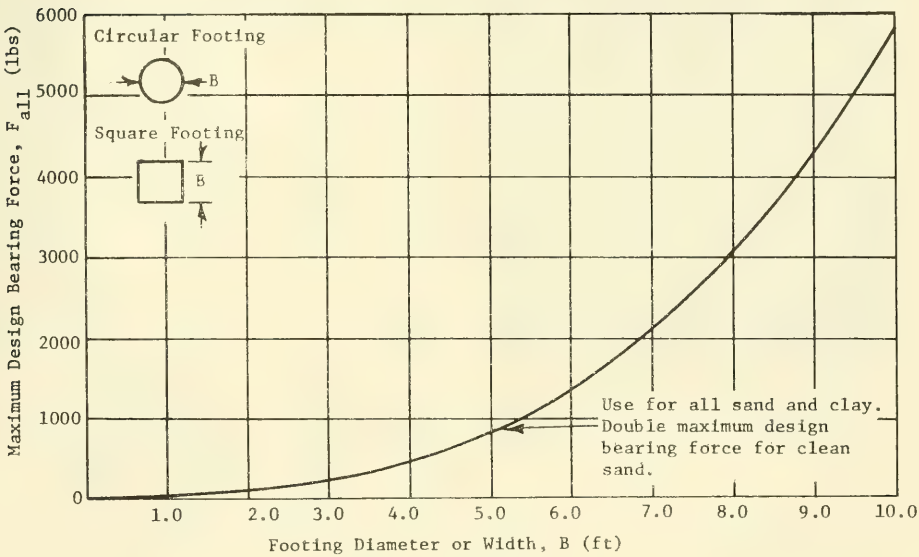
h. Strip Footings

i. Stubby Pile Foundation

Figure 2. Typical Foundation Configurations



a. Strip Footings



b. Spread Footing

Figure 3. Design Loads for Footings

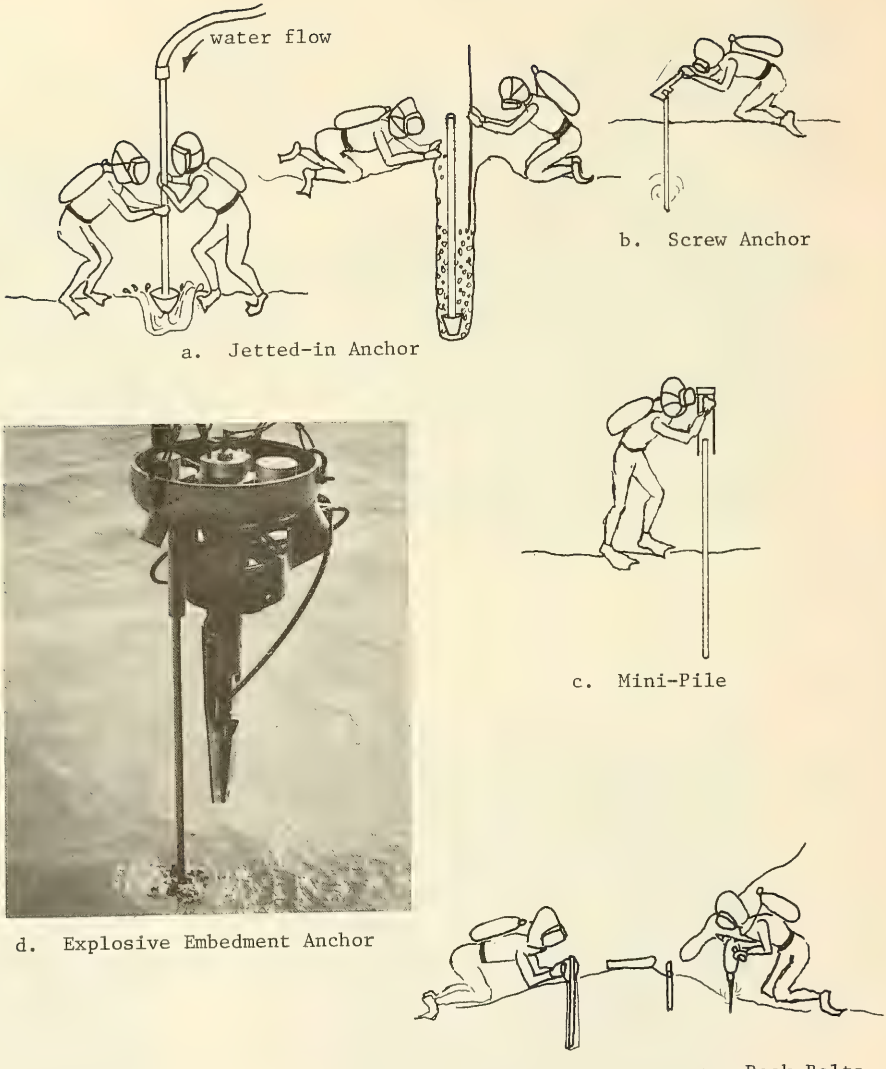


Figure 4. Tiedown Anchor Configurations

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LIST OF SYMBOLS

a	Angle of possible articulation for an individual footing (degrees)
A, A_1, A_2, etc.	Bearing area of an individual footing (square feet)
A_l	Maximum area projected laterally (square feet)
A_v	Area projected vertically (square feet)
B, B_1, B_2, etc.	Minimum lateral dimension of an indivdual footing (feet)
CSW	Center of submerged weight
d	Water depth (feet)
d_e	Vertical length of embedding key (feet)
D	Out-to-out diameter or distance between footings
F	Maximum vertical force on an indivdual footing (pounds)
F_a	Design capacity of a tiedown anchor (pounds)
F_{all}	Allowable design load for a footing (pounds or pounds per foot)
F_d	Maximum drag force (pounds)
F_h	Resistance capacity of a foundation to lateral loads (pounds)
F_i	Effective lateral force due to inclination of installation (pounds)
F_t	Maximum total vertical force on foundation (pounds)
i	Maximum inclination of slope at a site (degrees)
L	Length of a footing (feet)

M_d	Maximum foundation overturning moment (foot-pounds)
r	Maximum lateral excursion of an installation relative to the ship (feet)
r_1, r_2, etc.	Radius from CSW to center of individual footing (feet)
r_h	Minimum lateral clearance radius (feet)
r_{min}	Effective minimum distance to center of a footing element (feet)
t_e	Effective footing edge thickness (feet)
V_{max}	Maximum lowering rate (feet per second)
W_{dry}	Dry weight of installation (pounds)
W_{sub}	Submerged weight of installation (pounds)
W_{sd}	Dry weight of the structure to be supported (pounds)
W_{sw}	Submerged weight of the structure to be supported (pounds)
x_{max} y_{max}	Maximum lateral dimensions of the structure to be supported (feet)
z_{max}	Maximum vertical dimension of structure to be supported (feet)
z_{csw}	Vertical distance between the center of submerged weight and plane of footings (feet)
z_h	Vertical height of lifting point (feet)
z_1	Average height of A_1 above the plane of the footing elements (feet)

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13. ABSTRACT

This report presents a procedure for the design of foundations for small non-manrated and non-strategic seafloor installations. These procedures are applicable only to installations with dimensions less than 15 feet and submerged weight less than 4000 pounds. They do not require a detailed analysis of the prospective site and are applicable to all seafloor sites, except those located on slopes greater than 10° and those in areas of rapid sediment accumulation, such as off mouths of large rivers. The report includes analyses of vertical and lateral loading and load resistance, tiedowns, use of materials, and foundation emplacement. Several typical foundation types and special features are described. Two example design problems are included to illustrate the design procedures.

DD FORM 1 NOV 65 1473 (PAGE 1)

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