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UNIFIED FACILITIES CRITERIA (UFC)

GENERAL CRITERIA FOR WATERFRONT CONSTRUCTION

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UNIFIED FACILITIES CRITERIA (UFC)

DESIGN: GENERAL CRITERIA FOR WATERFRONT CONSTRUCTION

U.S. ARMY CORPS OF ENGINEERS

NAVAL FACILITIES ENGINEERING COMMAND (Preparing Activity)

AIR FORCE CIVIL ENGINEER SUPPORT AGENCY

Record of Changes (changes are indicated by **1**\ ... /**1**/)

Change No.	**Date**	**Location**
1	1 Sept 2012	Removed 5-5.2 “Emergency Power”; modified 4—1 and 5-1; editorial changes throughout, including updating references.

This UFC supersedes MIL-HDBK-1025/6, DATED May 1988.

FOREWORD

The Unified Facilities Criteria (UFC) system is prescribed by MIL-STD 3007 and provides planning, design, construction, sustainment, restoration, and modernization criteria, and applies to the Military Departments, the Defense Agencies, and the DoD Field Activities in accordance with USD (AT&L) Memorandum dated 29 May 2002. UFC will be used for all DoD projects and work for other customers where appropriate. All construction outside of the United States is also governed by Status of Forces Agreements (SOFA), Host Nation Funded Construction Agreements (HNFA), and in some instances, Bilateral Infrastructure Agreements (BIA.) Therefore, the acquisition team must ensure compliance with the most stringent of the UFC, the SOFA, the HNFA, and the BIA, as applicable.

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CHAPTER 1

INTRODUCTION

1-1 **SCOPE**. This UFC contains general criteria for the design of piling, deck, and substructure framing and bracing, and hardware and fittings for waterfront construction. Unless indicated otherwise, these criteria also apply to the design of offshore structures. This document, and all references contained herein, provides guidance primarily to DOD and U.S. Coast Guard activities. They may also be useful, however, to commercial firms that are engaged in the design and construction of waterfront facilities.

1-2 **CANCELLATION**. This UFC cancels and supersedes MIL-HDBK-1025/6, *General Criteria for Waterfront Construction*, (May 1988).

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CHAPTER 2

PILING

2-1 GENERAL REQUIREMENTS

2-1.1 Capacity

2-1.1.1 **Capacity as a Structural Member**. For pile sections embedded in the ground refer to \1\ UFC 3-220-01N, *Geotechnical Engineering Procedures for Foundation Design of Buildings and Structures* and UFC 3-220-10N, *Soil Mechanics*. /1/ For sections freestanding in water, treat piles as structural columns with accommodation made for their un-braced length. Where, due to long-term creep effects, the use of the coefficient of sub-grade reaction would be inappropriate or if one is unavailable, the following assumptions may be made:

2-1.1.1.1 In soft, cohesive soils, the point of fixity may be assumed to occur at a depth of 3.05 m (10 ft.) below the mudline for piles having modulus of elasticity - moment of inertia products (EI) of 2.925 x 10^9 kg-cm^2 (10 x 10^9 lb-in^2) or less. The point of fixity may be assumed to occur at a depth of 3.66 m (12 ft) below the mudline for piles having an EI greater than 2.925 x 10^9 kg-cm^2 (10 x 10^9 lb-in^2.) (E equals Modulus of Elasticity of Pile in pounds per in^2 and I equals moment of inertia of pile in in^4.)

2-1.1.1.2 In loose, granular soils and in medium cohesive soils, the point of fixity may be assumed to occur at a depth of 2.44 m (8 ft) below the mudline for piles having an EI of 2.925 x 10^9 kg-cm^2 (10 x 10^9 lb-in^2) or less, and at a depth of 3.05 m (10 ft) below the mudline for piles having an EI greater than 2.925 x 10^9 kg-cm^2 (10 x 10^9lb-in^2.)

2-1.1.1.3 For other cases, assume a point of fixity at a depth of 1.5 m (5 ft) below the mudline. The effective length factor K shall be taken as:

- 0.80 - when the deck structure is light, the piles have minimum embedment into the pile cap and there is no special provision for moment transfer into the deck structure.

- 0.73 - when the deck structure is light and provision is made for moment transfer by embedment or other device into the deck structure.

- 0.65 - when the deck structure is heavy and a positive means for moment transfer is provided.

NOTE: These provisions do not apply if pile embedment is less than 3.05 m (10 ft) into firm material or 6.10 m (20 ft) into soft or loose material. If lesser penetration is achieved, assume that the pile tips are hinged. Also, the indicated effective length factors (K) apply only if batter piles (minimum batter of one horizontal to three vertical) or some other means are provided to resist lateral loads, i.e., the plumb piles are not intended to resist lateral loads. If no such means are provided, side-sway can occur, and correspondingly increased K factors shall apply (usually taken as 2.0).

Design piles for a minimum eccentricity of 0.10 times the equivalent diameter of the pile. The moment resulting from this minimum eccentricity is not additive to the moments indicated by analyses of the actual applied loads.

2-1.1.2 **Capacity of the Ground to Support the Pile**. Refer to provisions of \1\ UFC 3-220-01N, *Geotechnical Engineering Procedures for Foundation Design of Buildings and Structures* and UFC 3-220-10N *Soil Mechanics* /1/, pertaining to friction, end bearing resistance, and settlements of single piles and pile groups.

2-1.1.3 **Lateral Load Capacity**. For pile groups within which individual piles are spaced between three and six diameters center-to-center, assume that the soil reacts laterally on an equivalent pile having a diameter equal to three times the actual diameter. For closer spacing, reduce the assumed equivalent diameter proportionally to the spacing.

NOTE: A number of design curves have already been published to illustrate this arching effect within a "retained" soil mass.

2-1.1.4 **Capacity of Existing Piles**. Refer to Chapter 6 for information pertaining to the capacity of existing piles.

2-1.2 Details Applicable to All Pile Types

2-1.2.1 **Minimum Penetration**. Conform minimum penetration of piles to the following:

- Penetrate sufficiently into an acceptable bearing stratum to distribute the axial pile load within the supporting capacity of the soil (refer to \1\ UFC 3-220-01N, *Geotechnical Engineering Procedures for Foundation Design of Buildings and Structures* and UFC 3-220-10N, *Soil Mechanics*. /1/).

- Penetrate sufficiently below any probable future dredge depth to distribute the pile load within the supporting capacity of the soil, discounting the resistance of soil that may be removed by future dredging. Depth should account for possible scour problems. All piles should be designed for a minimum of 1.5 m (5 ft) of potential scour or future dredging.

- Minimum values. If pile penetration is less than 3.05 m (10 ft) into firm material or less than 6.10 m (20 ft) into soft or loose material, assume the pile tip to be "pinned" and incorporate special provisions such as driving hardened tips into the refusing stratum, drilled sockets, or drilled dowels to secure the tips of the piles against lateral displacements due to eccentricities and intentional lateral forces. Develop a minimum lateral resistance of at least 5 percent of the pile's design axial load. Increase the effective length factors as described above.

2-1.2.2 **Tolerances on Installation**. For piles fully, or near fully, embedded in the ground, the provisions of Unified Facilities Guide Specifications apply. The following provisions relate to piling installed for elevated platforms where the piles project several feet above the mudline:

- A slope within 4 percent of plumb or the specified batter, as a reasonable compromise between the design requirements and the practicality of installation.

- In locating the pile head, there is no limit, provided the structure can tolerate the revised pile spacing. However, consider residual stresses in the piles due to forcing the pile into the pile cap in evaluating the capacity of the pile. Do not apply an increase (or additional increase) in allowable stress to stress-combinations that include these residual stresses.

NOTE: For effective un-braced length, divided by radius of gyration (Kl/r) between about 40 and 100, these effects can be substantial. But for fully embedded piles, Kl/r commonly is less than 40 and locked-in stresses can be neglected. Exercise caution when using driving frames because they prevent lateral movements of the head of the pile and mask the existence of locked-in stresses. Provide ample edge distances so that the piles will fit into the cap without excessive force or restraint. Allow for tolerance in the location of the pile head of at least 1.5 percent of the exposed height up to a maximum of 152 mm (6 inches).

2-1.2.3 **Minimum Spacing**. The minimum spacing requirements of piles are as follows:

- Provide for adequate distribution of the load on a pile group to the supporting soil. Recommend 1 m (36 inch) spacing.

- No minimum values are specified other than practical limitations to avoid piles interfering with or intersecting each other. One technique is to use a center-to-center spacing equal to 5 percent of the pile length for parallel piles.

2-1.2.4 **Pile Caps in Contact With the Ground**. Design piles to carry the entire superimposed load with no allowance made for the supporting contribution of the material between the piles.

2-1.2.5 **Connection of Piles to Caps**. The following requirements pertain to the connection of piles to pile caps:

2-1.2.5.1 **Timber Piles with Timber Caps**

- No Tension in piles - Secure pile tops to caps with spiral-drive drift pins or steel straps.

- Tension in piles - Secure pile top to clamp-type cap with thru-bolts with spike grids.

2-1.2.5.2 **Timber Piles with Concrete Caps**

- No Tension in Piles - Embed pile top 102 mm (4 inches) into cap and secure with single #5 dowel driven into pile butt.

- Tension in Piles - Secure pile tops with through-bolted dowel anchorage

2-1.2.5.3 **Concrete Piles with Concrete Caps**

- No Tension in Piles - Embed pile top 102 mm (4 inches) into pile cap and secure with deformed dowels or exposed pre-stressing strands as required to develop a tensile load equivalent to one-half the pile's rated compressive bearing capacity.

- Tension in Piles - Embed pile top 102 mm (4 inches) into pile cap and secure with deformed dowels or exposed pre-stressing strand as required to develop the tensile structural capacity of the pile cross-section.

2-1.2.5.4 **Steel Piles with Concrete Caps**

- No Tension in Piles - Embed pile top 102 mm (4 inches) (min.) into pile cap and secure with deformed dowels welded to the pile (within the jacketed length) capable of developing a tensile load equivalent to one-half the pile's rated compressive bearing capacity.

- Tension in Piles - Embed pile top 102 mm (4 inches) (min.) into pile cap and secure with deformed dowels welded to the pile (within the jacketed length) capable of developing a tensile load equivalent to the pile's rated compressive bearing capacity.

2-1.2.6 **Batter Piles**. Make connections to adjacent piles or to the pile cap capable of developing a tensile load equivalent to the smaller of the pile's compressive bearing capacity or the tensile structural capacity of the pile cross-section.

2-1.2.7 **Splicing**. The necessity for pile splicing should be avoided through selection of pile type and cross-section. However, where unavoidable, construct splices to provide and maintain true alignment and position of the component parts of the pile during installation. Construct splices capable of developing all germane structural capacities of the spliced pile cross-sections regardless of the expected design loads of the pile.

2-1.2.8 **Mixed Types or Capacities of Piling and Multiple Types of Installation Equipment or Methods**. Mixed types or capacities of piling and different types of installation equipment or methods are permitted, provided that the effects on the superstructure of differential elastic shortening and settlement are considered.

2-1.2.9 **Slope of Batter Piles**. Unless special provisions are made for the difficulties of installation and the effects of diminished driving energy on the capacity, keep the maximum slope of the batter piles to 5 horizontal to 12 vertical maximum batter or 4 horizontal to 12 vertical preferred or steeper.

2-2 REQUIREMENTS FOR SPECIFIC TYPE OF PILES

2-2.1 **Untreated Timber Piles.** Refer to \1\ UFGS 31 62 19.13, *Wood Marine Piles* /1/.

2-2.1.1 **Marine / Brackish Water Environment**. In general, do not use untreated timber piles without total encasement in a marine or brackish water environment. The one notable exception is their use as fender piles in volatile berthing areas where pile longevity is dictated by impact damage rather than by biological attack. Otherwise, encapsulate untreated piles exposed to air and water in one of the many flexible barrier products now available. It is becoming increasingly common to install untreated piles with these “wraps” as new construction, adding rub strips as appropriate for fender piles. Untreated piles are also required in some locations, i.e. San Diego, due to local environmental concerns over using salt treated timber in the marine environment.

2-2.1.2 **Fresh Water Environment**. Untreated, un-encased timber piles may be used in fresh water applications as long as they are not exposed to air. They must be cut off below the permanent ground water level or below MLW. As in the marine environment, it is becoming more common to install untreated piles with flexible “wraps” to protect the air-exposed portions as new construction.

2-2.1.3 **Geometry**. Where they are applicable, use untreated timber piles that conform to ASTM D25, *Specification for Round Timber Piles*. The greatest economy can be achieved by selecting the geometric property of butt diameter with respect to length in accordance with \1\ ANSI O5.1 /1/, Class 3 poles, those most commonly used by utility companies.

2-2.1.4 **Seasoning**. Seasoning for untreated timber piles is not required.

2-2.1.5 **Protection for Tops of Piling**. Protection for the tops of untreated, un-encapsulated timber piling is not required.

2-2.1.6 **Species**. Any species of wood may be used that will provide the necessary structural capacity and that will withstand the driving stresses.

2-2.1.7 **Peeling**. Peeling of untreated timber piles is not required.

2-2.2 **Treated Timber Piles**. Refer to \1\ UFGS 31 62 19.13, *Wood Marine Piles* /1/.

2-2.2.1 **Piles**. Use piles that conform to ASTM D25, *Specification for Round Timber Piles*. The greatest economy can be achieved by selecting the geometric property of butt diameter with respect to length in accordance with \1\ ANSI O5.1/1/ , Class 3 poles, those most commonly used by utility companies and most likely to be stocked at mills.

2-2.2.2 **Preservative Treatment**. Use treated marine piling that bear the appropriate American Wood Preservers Association (AWPA) Quality Mark as follows: MP-1 (dual treatment) for use in areas of extreme borer hazard and in marine waters where Limnoria and Pholadidea attack may be expected, or where oil slicks may contribute to borer attack, and MP-2 for other conditions where pholad attack is not expected. MP-4 treatment (water-borne preservatives) may be considered. For specific requirements at particular locations, \1\ consult NAVFAC Engineering Command, Pacific (NAVFAC PAC), Atlantic (NAVFAC LANT), Northern Division (NAVFAC ML), Southern Division (NAVFAC SE), NAVFAC WEST, /1/ and PWC Yokuska Applied Biology Offices. Refer to the paragraph entitled “Timber” for properties of treated wood.

2-2.2.3 **Seasoning**. Seasoning of treated timber piles is required prior to treatment.

2-2.2.4 **Species**. The preferred species are Southern pine and Douglas fir. Use of other species is subject to NAVFAC approval. \1\ AWPA Standard U1 /1/ requires that the species be either Southern pine or Douglas fir. It is not normally necessary to differentiate between these two species, as selection will be a function of geographic availability. In areas where treatable soft woods are scarce or unavailable in the length required, concrete piling often represents a more economical alternative than transported timber piles.

2-2.2.5 **Protection for Pile Tops**. Treat cut ends by puddling creosote. Puddling is accomplished by using a sheet metal band to form a reservoir on top of the pile. The reservoir is filled with creosote oil and left to stand for 8 to 12 hours. Alternative protection methods include coating pile tops with pitch (with or without sheet metal or plastic covers).

NOTE: Use of sheet metal covers as end protection for fender piles is discouraged because they are easily torn by impact and become a personnel hazard. However, sheet metal covers for bearing piles under pile caps provide good protection. In general, fit structural piling with waterproof caps.

2-2.3 **Precast (Including Prestressed) Concrete Piles.** These piles are covered in both \1\ UFGS 35 59 13.13, *Prestressed Concrete Fender Piling* and UFGS 31 62 13.20, *Precast/Prestressed Concrete Piles* /1/. Reference is also made to UFGS \1\ 03 31 29 /1/, *Marine Concrete*.

2-2.3.1 **Minimum Dimensions**. The minimum dimensions are 304.8 mm (12 in) for piles of uniform section and 203.2 mm (8 in) for tapered piles.

2-2.3.2 **Cover**. The minimum clear cover for reinforcement for permanent installations in salt water is 76.2 mm (3 in). For temporary installations and in fresh water, cover requirements may be relaxed to conform to the requirements of \1\ UFC 1-200-01, *General Building Requirements* /1/, for normal exposure conditions.

2-2.3.3 **Minimum Reinforcement**. Excluding pre-stressed piles, the minimum longitudinal reinforcement shall be 1.5 percent of the total cross section.

2-2.3.4 **Ties**. Provide spirals or ties for longitudinal reinforcement. Proportion spirals and ties in accordance with \1\ ACI 318/318R, *Building Code Requirements for Structural Concrete and Commentary* for structural columns except provide additional ties or spirals at ends as indicated in PCI STD-112, *Standard Prestressed Concrete Piles Square, Octagonal and Cylinder.* /1/

2-2.3.5 **Impact**. Forces induced by handling and driving shall be imposed magnified by a load factor of 1.25 (allowable overstress of 33 percent).

2-2.3.6 **Jetting**. Where jetting is contemplated, the jet pipe may be cast into the pile.

2-2.3.7 **Concrete Strength**. The minimum compressive concrete strength required is 34.47 MPa (5,000 psi) for pre-stressed concrete, and 27.58 MPa (4,000 psi) for non-pre-stressed concrete.

2-2.3.8 **Standard Details**. For information on standard details, refer to PCI STD-112.

2-2.3.9 **Minimum Residual Prestress**. The minimum residual pre-stress is 4.83 MPa (700 psi).

2-2.3.10 **Minimum Wall Thickness (Cylindrical Piles or square Piles With Voids)**. Provide a minimum of 3.81 mm (1-1/2 in) clear cover on inside (void) face. In no case may wall thickness be less than 102 mm (4 in).

2-2.3.11 **Venting**. If a void is provided which extends through to the lower end of the pile, vent the pile head to prevent the buildup of internal pressure during driving.

2-2.3.12 **Tolerances**. Locate voids, when used, within 9.5 mm (3/8 in) of the position shown on the plans. The maximum departure of the pile axis from a straight line, measured while the pile is not subject to bending forces, should not exceed 3.17 mm (1/8 in) in any 3.05 m (10 ft) length or 9.5 mm (3/8 in) in any 12.2 m (40 ft) length. Overall sweep should not exceed 0.1 percent of the pile length.

2-2.4 **Cast-in-Place Concrete Piles.** These piles are covered in \1\ UFGS 31 62 13.13, *Cast-In-Place Concrete Piles*. Reference is also made to UFGS 03 31 29, *Marine Concrete*. /1/

2-2.4.1 **General**. Cast-in-place concrete piles are not recommended for applications characterized by severe marine exposures and / or long un-braced lengths.

2-2.4.2 **Casings**. Use casings that meet the following criteria:

- Adequate strength to withstand driving stresses and to resist distortions imposed by the driving of adjacent piles.

- Except for portions of piles embedded more than 1.52 m (5 ft) below the mud line, casings are to remain in place to reinforce and / or protect the concrete core. Regardless of any coatings or other means of corrosion protection provided, do not consider casing metal thicknesses of 3.2 mm (1/8 in) or less as contributing to the structural capacity of the pile. Extend casings intended to provide flexural reinforcement to the point of fixity (defined in the paragraph entitled "Capacity as a Structural Member") and apportion the casings so that the metal thickness remaining at the end of the design service life equals or exceeds that required by analysis. Provisions for corrosion protection are discussed further in the paragraph entitled "On Hard Bottom (Rock or Hardpan)".

2-2.4.3 **Minimum Tip Diameter**. Use minimum tip diameter of cast-in-place concrete piles of 203 mm (8 in).

2-2.4.4 **Reinforcement**. Detail sections of piling requiring internal reinforcing in the same manner as pre-cast concrete piles. Cover requirements for reinforcing are the same as for pre-cast piles with spacers provided for longitudinal reinforcement to ensure that cover requirements are maintained.

2-2.4.5 **Concrete Strength**. The minimum concrete compressive strength to be used in the marine environment is 27.58 MPa (4,000 psi) and 24.13 MPa (3,500 psi) in freshwater applications.

2-2.5 **Steel H-Piles.** These piles are covered in UFGS \1\ 31 62 16.16 /1/, *Steel H-Piles.*

2-2.5.1 **Minimum Thickness of Metal**. Determine the thickness of metal from consideration of loss of section as established in \1\ UFC 1-200-01, *General Building Requirements* /1/ unless corrosion protection is provided as described in the paragraph entitled "Corrosion Protection". In any case, the minimum thickness should not be less than 10.55 mm (0.40 in). Splice plates should not be less than 9.5 mm (3/8 in) thick.

2-2.5.2 **Corrosion Protection**. When the required initial minimum thickness of unprotected metal would be excessive, provide alternative corrosion protection in the form of concrete encapsulation, bituminous or plastic (epoxy) coatings, flexible membrane / tape wrappings, or cathodic protection. For cathodic protection refer to \1\ UFC 3-570-02N, *Electrical Engineering Cathodic Protection* , UFGS 26 42 13.00 20, *Cathodic Protection by Galvanic Anodes*, and UFGS 26 42 19.00 20, *Cathodic Protection by Impressed Current*. /1/ Bituminous or plastic coatings are not considered effective below the mudline and require special care to avoid damage during driving. In tropical environments, and other locations where corrosion is particularly severe, encase steel pilings in concrete jackets to at least 0.61 m (3 ft) below MLLW and provide cathodic protection for the submerged and buried sections of steel.

2-2.5.3 **Cap Plates**. Bearing cap plates are not normally required for steel compression piles embedded in concrete pile caps. Where pile flexure or tension is intended, tie the tops of steel piles into the cap with reinforcing bars or structural sections welded to the pile and lap-spliced to the cap reinforcing.

2-2.5.4 **Hardware and Fittings**. Refer to Chapter 4 for guidance on specifications for hardware and fittings for H-piling.

2-2.5.5 **Limitation on Use**. The tips of all steel H-piles having a web thickness of less than 12.69 mm (1/2 in), and driven to end-bearing on sound rock by an impact hammer, must be reinforced with a driving tip (shoe). Both penetration resistance and equipment operation must be closely monitored so as to terminate pile driving immediately upon reaching refusal on the rock surface.

2-2.6 **Drilled Caissons and Auger-Placed Grout Piles**. These piles are covered in both \1\ UFGS 31 63 26, *Drilled Caissons* and UFGS 31 63 16, *Auger Cast Grout Piles.* /1/

2-2.6.1 **Applicability**. Usage of drilled caissons and / or auger-placed grout piles is normally reserved for conditions where the driven piles would bear on rock or other hard bottom, at shallow depth, without sufficient penetration for lateral support or uplift resistance.

2-2.6.2 **On Hard Bottom (Rock or Hardpan)**. Use a constant diameter shaft without the bell. Level sloping surfaces to receive the shaft. Anchorage in the form of grouted dowels or excavated keys should be considered.

2-2.6.3 **On Other Bottom Surfaces**. Consider shaft and bell construction if loads are heavy enough to warrant use of the bell. Unless supported on piling, embed the bell 0.61 to 1.22 m (2 to 4 ft) into firm material.

2-2.6.4 **Minimum Reinforcement**. Determine reinforcement by design requirements, not by a minimum reinforcement ratio.

2-2.6.5 **Embedment of Piling into the Bell**. Embed piling into the bell as required for transfer of load. Where tremie placement is employed, use 69 kPa (10 psi) bond resistance between pile and concrete, plus compression resistance of top of pile bearing in bell.

2-2.6.6 **Protection for Reinforcement**. The requirement for protection for reinforcement is the same as for concrete piling.

2-2.6.7 **Thickness of Metal Shell and Corrosion Protection**. The requirements for thickness of metal shell, and corrosion protection, are the same as for steel cased cast-in-place concrete piles.

2-2.6.8 **Installation**. Tremie placement of concrete fill is permitted. Provide for final cleanout of the bell or base of the cylinder immediately before concreting.

2-2.7 **Steel Pipe Piles**. Use pipe piles that conform to the applicable requirements for both steel H-piles and cast-in-place concrete piles (refer to the paragraphs entitled “Steel H-Piles and Drilled Caissons” and “Auger-Placed Grout Piles” \1\ /1/.) 2-2.7.1 **Material**. Use material conforming to ASTM A252, \1\ *Welded and Seamless Steel Pipe Piles* /1/, unless otherwise approved.

2-2.7.2 **Open-End Piles**. Reseat pipes installed with an open tip to full bearing after being cleaned out. If the pipe exhibits 50.8 (2 in) or more of additional penetration during re-seating, cyclically re-clean and re-drive until penetration on subsequent re-driving is less than 50.8 mm (2 in). If leakage of water into the pile is minor, pump the pile out and place one cubic yard of grout as an initial seal before the balance of concrete or sand fill is installed. If water leakage renders dry grout placement impractical, fill the pipe pile to its top with clean water and tremie the grout plug. Deposit the grout seal by means of a grout pipe to an elevation of at least 0.91 m (3 ft) above the bottom of the pile. After a sufficient time has elapsed to allow the grout to set, pump the pile dry and fill remaining space with concrete or clean dry sand.

2-2.7.3 **End Closure**. For friction piles, tip closures should not project more than 12.7 mm (1/2 in) beyond the pipe wall.

2-2.8 **Composite Piles**. These piles are seeing increase application throughout the Navy. \1\ NAVFAC LANT /1/ has prepared a regional guide specification, \1\ UFGS 35 59 13.14 20, *Polymeric Fender Piles* /1/ which covers their use.

2-2.9 **Sheet Piling--Steel**. This is covered in \1\ UFGS 31 41 16, *Metal Sheet Piling* /1/. The requirements for steel sheet piling are the same as previously established for steel H-piling (refer to the paragraph entitled “Steel H-Piles”), except as modified in the paragraphs below.

2-2.9.1 **Splices**. Use full-penetration butt welds for splices.

2-2.9.2 **Connection to Caps**. Embed steel sheet piles 52.3 mm (6 in) minimum into concrete caps. The use of structural sections for mechanical anchorage of sheet piling into the cap is neither required nor advised. However, provide stirrups (open to the side) through the handling hole near the top of each sheet as a positive connection between the steel sheet piles and the cap's longitudinal reinforcing. For steel channel caps, tack-weld each sheet to the cap member.

2-2.9.3 **Sleeves and Openings**. Detail all sleeves and openings for utilities and drains passing through the sheets to prevent loss of fill.

2-2.9.4 **Minimum Thickness of Metal**. For exposed faces of cofferdams, use a minimum sheet pile thickness of 12.7 mm (1/2 in). Elsewhere, use a minimum thickness consistent with the applied loading environment over the design service life of the structure. In no case should the minimum thickness be less than 3/8 in. (38 mm).

2-2.10 **Sheet Piling - Concrete**. Use piling that conforms to the requirements stated above for pre-cast, pre-stressed concrete piles (refer to the paragraph entitled "Precast (Including Prestressed) Concrete Piles**"**) except as modified in the paragraphs below.

2-2.10.1 **Joints**. Flush and grout joints tight to the mudline. Use of plastic sleeves is recommended.

2-2.10.2 **End of Sheets**. Use sheets cast with a drift-sharpened point. Embed tops of sheets 152.4 mm (6 in) into a continuous concrete cap.

2-2.10.3 **Sleeves and Openings**. The treatment of sleeves and openings in concrete sheet piling is similar to the procedure followed with steel sheet piling (refer to the paragraph entitled "Sheet Piling--Steel ").

2-2.11 **Sheet Piling - Timber**. Use timber sheet piling conforming to the requirements in the paragraphs entitled "Untreated Timber Piles**"** and "Treated Timber Piles**"** for treated and untreated timber piles except as modified in the paragraphs below.

2-2.11.1 **Applicability**. Timber, for the most part, no longer represents an economically competitive alternative for the construction of bulkheads. Within the range of structural applicability, aluminum and the burgeoning industry of plastic products represent lower life cycle costs than timber in all but the most benign marine environments. Therefore, the selection of timber will most probably be made based on specific aesthetic parameters.

2-2.11.2 **Treatment**. \1\ Use timber sheet piling in accordance with AWPA Standard U1 /1/. The types of treatment are as described for treated timber piles.

2-2.11.3 **Joints**. Use tongue and groove or splined joints (or Wakefield Type sheeting may be used). Install sheet piling tight to the mudline.

2-2.11.4 **Drift Sharpening**. Use drift sharpened timber sheet piling.

2-2.11.5 **Tops of Sheets**. Bolt tops of sheets to a continuous timber cap, with a width equal to or greater than the thickness of the sheet piling. Where a concrete cap is used, embed the sheets 152.4 mm (6 in).

2-2.11.6 **Sleeves and Openings**. Detail sleeves and openings through timber bulkheads to preclude the loss of fill material.

CHAPTER 3

DECK AND SUBSTRUCTURE FRAMING AND BRACING

3-1 **SUBSTRUCTURE**. The substructure includes pile caps, under-deck bracing, and other structural members (other than stringers) at and below the level of the pile caps.

3-1.1 **Pile Caps - All Types**. The effects of differential axial deformation among piles must be investigated where:

- heavy concentrated loads occur,
- piles are long,
- there is an appreciable variation in pile lengths
- pile types or installation methods vary.

Differential deformations will not appreciably affect ultimate strength if the cap can deform elastically without buckling or fracture.

3-1.2 **Timber**

3-1.2.1 **Hardware and Fittings**. For requirements pertaining to hardware and fittings, refer to Chapter 4.

3-1.2.2 **Species and Preservative Treatment**. Except for temporary structures, give substructure timbers a preservative treatment. Use a species that will accept deep treatment such as Southern pine or Douglas fir. Do not use untreated timber for permanent structures. Resistance to borers and decay of any untreated lumber - even of species presumed to be of superior resistance - is still inadequate for long-term use. Encase species that do not accept preservative treatment.

3-1.2.3 **Seasoning**. Use only seasoned timber for framing.

3-1.2.4 **Minimum Dimension**. The minimum nominal timber size should be 76 mm (3 in). National Fire Protection Association requires the following minimum dimensions: cross bracing – 102 mm (4 in); pile caps – 203 mm (8 in); stringers – 152 mm (6 in); and decking – 102 mm (4 in) with stringers, 76 mm (3 in) without stringers.

3-1.2.5 **Retention and Penetration of Preservative**. For guidance on the use of preservatives, conform to requirements of \1\ AWPA Standard U1 /1/.

3-1.3 **Concrete**. Refer to UFGS \1\ 03 31 29 /1/, *Marine Concrete.*

3-1.3.1 **Cover**. Use a cover that conforms to the requirements of the paragraph entitled “Cover”.

3-1.3.2 **Chamfer**. The minimum chamfer for all exposed outside corners should be 25.4 mm (1 in).

3-1.4 **Steel**. Use components comprising steel substructures that conform to the requirements for steel H piles (refer to the paragraph entitled “Steel H-Piles”).

3-2 **DECK**. The "deck" includes treads, planks, slabs, stringers, and other elements supported by the pile caps.

3-2.1 **Timber**. Use timber in the deck structure conforming to the requirements for substructure framing and bracing except as modified below.

3-2.1.1 **Treatment**. Give deck framing and bracing a preservative treatment. For the deck itself, do not use creosote on walking surfaces or surfaces normally touched by people (handrails, for example).

3-2.1.2 **Hardware and Fittings**. Refer to Chapter 4 for information on hardware and fittings.

3-2.1.3 **Decking**. Conform decking to the following materials and dimensions:

- Oak, maple, birch, black gum, or other species resistant to wear, may be used untreated for treads or traction cleats.
- Use decking not more than 300 mm (12 in) nor less than 100 mm (4 in) wide (nominal sizes).
- Use decking not less than 76 mm (3 in) thick (nominal size); 102 mm (4 in) when laid on timber stringers. Topping is 51 mm (2 in) sheathing, concrete, or asphalt.
- Provide a 10 mm (3/8 in) gap (minimum) between adjacent deck timbers.

3-2.2 **Concrete**

3-2.2.1 **General**. Conform to requirements for substructure framing and bracing.

3-2.2.2 **Deck Finish**. Broom-finish the deck to provide a skid-resistant surface.

3-2.3 **Steel**. When steel is used for stringers, use steel conforming to the requirements for steel H piles (refer to the paragraph entitled “Steel H-Piles”).

CHAPTER 4

HARDWARE AND FITTINGS (PERMANENT INSTALLATION)

4-1 **SALTWATER—IN OR BELOW SPLASH ZONE.** \1\ For information on mooring fittings, refer to UFC 4-159-03, *Design: Moorings*. /1/

4-1.1 **Minimum Diameter of Bolts**. Use bolt with minimum diameter of 25 mm (1 in).

4-1.2 **Minimum Thickness of Metal in Straps and Fittings**. Use metal with minimum thickness of 12.7 mm (1/2 in).

4-1.3 **Coatings**. Typically, mooring fittings are cast and painted.

4-1.4 **Washers**. Provide ogee washers for all bolts used in timber construction. Provide bolts used in concrete or steel structures with plate or standard circular washers with no more than two at any location. Fit inclined bolts or bolted surfaces with beveled washers.

4-1.5 **Size of Bolt Holes**. Drill all bolt holes in timber (other than holes for drift bolts) with a bit having a diameter 1.6 mm (1/16 in) larger than the diameter of the bolt shank. Align bolt-holes to allow insertion by tapping with a mallet. Do not drive or force-fit bolts. Drill holes for drift bolts 3.17 mm (1/8 in) less in diameter than the bolt diameter. Keep all drill bits sharp and control feed rate to produce shavings, rather than chips.

4-1.6 **Locking of Bolts**. Tack-weld nuts or damage the bolt threads outboard of the nut to lock all bolted connections.

4-2 **SALTWATER--ABOVE SPLASH ZONE**. Use same requirements as for installations in or below splash zone except as modified below:

- Minimum bolt diameter may be reduced to 19 mm (3/4 in).
- 10 mm (3/8 in) minimum thickness metal in straps and fittings.
- 6 mm (1/4 in) minimum thickness plate washers.

4-3 **FRESHWATER**. Requirements are the same for installation in saltwater, in or below splash zone, except as modified below:

- 15.87 mm (5/8 in) minimum diameter bolts.
- 6 mm (1/4 in) minimum thickness metal in straps, fittings, and plate washers.

4-4 **SPECIAL APPLICATIONS**

4-4.1 **Stainless Steel Fittings**. Stainless steel fittings may be used for special applications, if warranted, but should be used judiciously because many stainless steels do not perform well in saltwater. They often experience severe pitting and crevice corrosion more than regular carbon steel.

4-4.2 **Through Bolts**. Use through bolts to the fullest extent possible.

CHAPTER 5

SPECIAL CONSIDERATIONS

5-1 **SERVICE LIFE**. Unless specifically intended for a limited service life, or unless otherwise stipulated in job-specific criteria, design structures for a service life of 25 years. Where a service life of 1 year or less is intended, design may be predicated on an overall load factor of 1.15 for dead plus live load combined with any other single load; or 1.10 when combined with two or more other loads. Load factors for designs are intended for limited service life.

- Estimating Service Life. Assume structures detailed in accordance with this handbook meet the service life requirement in the paragraph entitled "Service Life".

- Accessibility. Since waterfront structures frequently serve longer than 25 years, detail so that all components with an anticipated service life of less than 50 years can be inspected and repaired or replaced.

5-2 **CORROSION OF STEEL**. The principal factors affecting rate of corrosion loss are:

- Geographical location,
- Localized zones relative to tidal planes,
- Exposure to salt spray,
- Sand, earth, or other cover,
- Protective coating(s),
- Abrasive conditions (surf zone versus deep water),
- Stray electric currents, and
- Soil type.

5-2.1 **Service Life of Coating Systems**. Refer to Table 5-1 for approximate expected periods of protection afforded by various coating systems.

5-2.2 **Tropical Climates**. Encase steel H-piling in concrete in and above the tidal range and to a minimum depth of 1.5 m (5 ft) below MLW. Cap steel sheet piling with concrete rather than with a steel channel or timber.

Table 5-1 Period of Protection for Steel to be Expected from Various Coating Systems of Common Use[a]

COAT DESCRIPTION[b]	PERIOD OF PROTECTION[c]
Coal tar epoxy (15 to 20 mils thickness)	10 - 20 years
Galvanizing (7 to 9 mils thickness)	10 - 15 years
Metallized Aluminum	15 - 20 years
Concrete Encasement	25 years

Notes: (a) Marine exposure
(b) Coatings applied properly
(c) Periods of good to excellent protection, i.e. negligible loss of metal

5-2.3 **Use of Weathering Steels**. Use "weathering" steels conforming to the following requirements:

- The following steels require coating in the splash zone and other areas not boldly exposed to sun, wind, and rain: \1\ Alloy steels conforming to ASTM A242/A242M, *High-Strength, Low-Alloy Structural Steel*; A588/A588M, *High-Strength, Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point with Atmospheric Corrosion Resistance*; or A690/A690M, *High-Strength Low-Alloy Nickel, Copper, Phosphorus Steel H-Piles and Sheet Piling with Atmospheric Corrosion Resistance for Use in Marine Environments*; and copper bearing steels conforming to ASTM A709/A709M, *Structural Steel for Bridges*. /1/

There are no consistent data on the rate of corrosion loss for steel surfaces in contact with various soils, so that consideration should be given to coating these surfaces in the same manner and degree as for \1\ ASTM A36/A36M, *Carbon Structural Steel*. /1/

- If an alloy conforming to \1\ ASTM A690/A690M is used, use hardware conforming to ASTM A588/A588M. /1/

5-3 **CATHODIC PROTECTION**. Consider cathodic protection for all buried or submerged steel structures or utilities in accordance with \1\ UFC 3-570-02N, *Electrical Engineering Cathodic Protection* /1/.

5-3.1 **Cathodic Protection Systems.** Consider a Cathodic Protection System (CPS) in conjunction with other protective measures such as material thickness incrementation, protective coatings and encasement for the following waterfront metallic structural systems:

- Steel sheet piling bulkheads
- Steel Bearing piles for piers
- Steel fender piles for piers
- Mooring components

In marine environments, CPS are most effective and can greatly extend the life of the submerged zones of steel waterfront structures. The splash and atmospheric zones will require reapplication of coatings and encasements for maximum system service life. Partial concrete encasement of steel piles creates a zone of high potential at the concrete encasement-to-bare steel pile interface where submerged. Provide CPS in these circumstances in addition to the partial encasement.

5-3.2 **Economic Feasibility.** Evaluate providing CPS for the following buried or submerges systems:

- Existing steel waterfront structures
- Reinforcing steel in concrete

Base implementation of CPS on life-cycle economics. Requirements for CPS will be determined by the corrosion engineer.

5-3.3 **Rehabilitation.** When rehabilitating existing steel pile bulkheads by driving new sheets outboard of the existing, include the following requirements for the CPS:

- Electrically isolate new piling from old piling
- Electrically isolate tie rods from existing sheet piling by cutting a hole in the old piling and providing a dielectiric sleeve through the pile.
- Coat tie rods and new piling on all sides.
- Consider CPS as part of the total corrosion protection system. Use conventional soil side anodes to protect the seaside and landside of the pile and to protect the tie rods if field tests indicate this to be feasible. Otherwise, consider using a deep anode bed system. Waterside anodes are appropriate only in areas not subject to maintenance dredging, water turbulence from ship/boat traffic, normal or storm generated heavy wave action, or constant movement of the sea bottom. Conduct a site survey to

determine the appropriate anode configuration and cathodic protection system requirements.

5-3.4 **Efficacy**. In general, the rate of corrosion loss below MLW is two-thirds to one-half the rate just below, at, and above MLW. Since cathodic protection is effective only below MLLW, it follows that cathodic protection should be accompanied by use of a concrete fascia or encasement to and below MLLW. Therefore, the obvious cost comparison is between such a composite system and complete concrete encapsulation.

5-3.5 **Maintenance Cost**. Consider the cost of electricity, replacement of anodes, and general repair of damage to wires and hangers in the economic analysis.

5-3.6 **Reliability of Maintenance Effort**. A cathodic protection system rendered fully or partially inoperative due to a lack of maintenance and repair is all too common. Implement regularly scheduled maintenance inspections to minimize risk of failure of the cathodic protection system.

5-4 EXPANSION, CONTRACTION, AND CONTROL JOINTS

5-4.1 **Open-Pile Platforms**. For open-pile pier and wharf platforms, refer to \1\ UFC 4-152-01, *Design: Piers and Wharves* /1/.

5-4.2 **Bulkheads**. In normal practice, no expansion or contraction joints are provided in the sheeting regardless of type. Assuming that a concrete cap will effectively grip the sheeting, there is no rational reason to joint the cap either as it is not free to strain axially anywhere along its length. Similarly, no special joints are required for timber or steel caps or for any anchor wall.

5-4.3 **Concrete Quaywalls**. Provide joints every 90 to 120 m (300 to 400 ft). Such formed joints need not be carried more than 1.52 m (5 ft) below MLLW.

5-5 MISCELLANEOUS REQUIREMENTS

5-5.1 **Protective Lighting**. \1\ Refer to UFC 4-025-01, *Waterfront Security Design,* currently in final draft and near final publication. /1/

\1\ /1/

5-5.2 **NAVAIDS**. Provide navigational aids at ends of piers, wharves, or quays. The cost of prominent, well-lighted markers is negligible in comparison to that of a collision. Refer to \1\ UFC 4-150-06, *Military Harbors and Coastal Facilities* /1/, for specific requirements.

5-6 **TIMBER**. The use of timber in the marine environment should be based on life-cycle economics. If timber is placed in the marine environment, it should be pressure treated according to American Wood Preservers Association Standards unless state and local regulations restrict its installation, cutting, use, or disposal. Conversely, the timber may be wrapped by plastic according to \1\ UFGS 31 62 21, *Piling: Composite, Wood and Cast In-Place Concrete* /1/. Field divisions and Activities should conduct site-specific risk assessments for each area containing a significant quantity of timber to determine the impact on the local marine environment. The risk assessment method may employ the software developed by the Western Wood Preservers Institute or other similar systems. The assessment may also include a leachability analysis if required by the locale. Most Field Divisions and Field Activities have applied biologists on staff to assist in the planning and design process.

For treated Douglas fir and Southern pine, see Table 5-2 for examples of structural characteristics as functions of preservative type.

5-7 FIRE PROTECTION REQUIREMENTS

5-7.1 **General**. Apply the provisions of the National Fire Protection Association (NFPA) Standard NFPA 307, *Standard for the Construction and Fire Protection of Marine Terminals, Piers, and Wharves*, and of \1\ UFC 3-600-01, *Fire Protection Engineering for Facilities* /1/.

5-8 **ANTI-TERRORISM / FORCE PROTECTION.** Refer to UFC 4-025-01, *Waterfront Security Design*, currently in final draft and near final publication /1/.

\1\ /1/

Table 5-2 Properties of Treated Woods

Type of Treatment	Average Properties							
	Modulus of Rupture		Modulus of Elasticity in flexure		Energy Absorption in Flexure		Compressive Strength	
	PSI	%	PSI	%	PSI	%	PSI	%
Fir								
Untreated	8394	100	1.922	100	6.388	100	3346	100
Creosote	6862	82	1.584	82	4.202	66	N/A	N/A
ACA Dual	6111	73	1.637	80	3.069	48	2714	81
CCA Dual	3844	46	1.171	61	3.364	63	2333	70
ACA	5620	67	1.416	74	2.078	33	2462	74
Pine								
Untreated	8007	100	1.942	100	5.24	100	N/A	N/A
Creosote	6960	74	N/A	N/A	N/A	N/A	N/A	N/A
ACA Dual	4726	59	1.568	81	2.829	64	N/A	N/A
CCA Dual	4167	52	1.441	74	2.413	46	N/A	N/A
ACA	5534	69	1.538	79	N/A	N/A	N/A	N/A
CCA	5410	68	N/A	N/A	N/A	N/A	N/A	N/A

NOTES:
1) “N/A” indicates a large spread in measured values for a small number of samples.
2) % = percent of the value for untreated wood.
3) Source: Civil Engineering Laboratory, Technical Note (TN) No. N1636 Mechanical Properties of Preservative Treated Marina Piles - Results of Limited Full - Scale Testing.

CHAPTER 6

STRENGTH EVALUATION OF EXISTING WATERFRONT STRUCTURES

6-1 **EVALUATING STRENGTH OF EXISTING MATERIALS.** Recent work by Naval Facilities Engineering Service Center (NFESC) has accomplished much in evaluating the strength of existing materials. NFESC has conducted impact load tests, followed by finite element analysis to evaluate the strength of a number of waterfront locations. This was followed by development of innovative repair techniques such as installation of a laminate composite overlay on the underside of existing Navy piers. \1\ A series of reports have been published on this topic that can be obtained from NFESC: https://portal.navfac.navy.mil/portal/page/portal/navfac/navfac_ww_pp/navfac_NFESC_pp . /1/ For strength evaluation of mooring hardware, reference UFC 4-150-08 *Inspection of Mooring Hardware.*

6-1.1 **General**. Apply the provisions of \1\ UFC 3-301-01, *Structural Engineering* /1/ relating to the use of used and / or unidentified materials.

6-1.2 **Number of Tests Required to Establish Strengths of Undocumented Materials**. Where no documentation exists pertaining to the strength of an existing material, the strength must be established by tests of the material. Use the value that sampling and testing indicates to have a 95 percent probability of being exceeded, as the strength of material to be assumed for strength evaluation of the structure. Use no fewer than four samples of a given material for testing.

6-1.3 **Tests and Test Specimens**

6-1.3.1 **Steel Members**. For steel members, take test specimens from locations and as described in \1\ ASTM A6/A6M, *Standard Specifications for General Requirements for Rolled Structural Steel Bars, Plates, Shapes and Sheet Piling.* /1/

6-1.3.2 **Concrete Members**. For concrete members, use drilled cores and sawed beams as described in \1\ ASTM C42/C42M, *Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.*/1/

6-1.3.3 **Wood Members**. For wood members, stress-grade visually as described in \1\ ASTM D245, *Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber*. /1/

6-2 **COMPUTING STRENGTH OF THE STRUCTURE**. Base analyses on measured in-place dimensions and existing conditions. A badly deteriorated or obviously overloaded structure often continues to support the applied loads with no discernible indications of distress. It is important to consider the factors contributing to this phenomenon when evaluating the strength of an existing structure. The more important of these factors are presented in the following paragraphs.

6-2.1 **Simplifying Design Assumptions**. Structural design commonly employs simplifying assumptions intended to make the design effort more manageable. These assumptions are invariably conservative and often result in substantial excess strength, for example, in presenting the distribution of concentrated loads to a slab, and in the structural design of variable sections.

6-2.1.1 **Distribution of Concentrated Loads to a Slab**. Conventional procedures, such as those described in the references of \1\ UFC 1-200-01, *General Building Requirements* /1/ and the American Association of State Highways and Transportation Officials (AASHTO), underestimate distribution of concentrated loads applied to a slab. Previous discussion in the paragraph entitled "Evaluating Strength of Existing Materials**"** relates here.

6-2.2 **Locations of Weakened Sections**. Members are proportioned for maximum stress conditions. The section required at points of maximum stress frequently is carried for the full length of the member to minimize the costs of fabrication or of form work, or for aesthetic reasons. If the deterioration of a member is localized and does not occur at a point of maximum stress, the "strength" of the overall member may not be impaired by exhibited deterioration.

6-2.3 **Changed Design Methodology**. If a structure was originally designed based on an elastic analysis, re-analysis based on ultimate strength, plastic redistribution of moments, or moment distribution based on the concept of yield line theory will frequently yield a greater analytical capacity.

6-2.4 **Design Live Loads**. Design live loads are seldom realized in practice. Therefore, original design loadings should be assessed and compared to actual load requirements for continuing validity.

6-2.5 **Excess Section**. Designs often contain excess strength by way of provision of sacrificial metal, rounding member sizes to the next heavier section, or to a lighter but stronger section, or for satisfaction of requirements for minimum thickness of metal or limiting deflection. Piling, in particular, is often sized beyond its structural requirement to resist driving stresses or for load transfer to the soil.

6-2.6 **Change of Structural Action**. The structural response to an applied load may differ from that assumed during design. Ordinary beams and slabs are a common case in point. These are proportioned on the basis of pure flexural behavior. However, except for large ratios of span to depth, pure flexural action is not realized, and the member resists the load, at least partly, by catenary action or arching. Composite actions between separate different structural elements may develop over time, which had not yet developed during original construction. Yield points may develop, changing the moment diagram and reactions, thus increasing some and decreasing others, with the changes often being non-critical.

6-2.7 **Change in Loading**. In some cases, the design load represents a temporary or construction condition, and the service loads are of lesser severity. For example, if a retaining wall is well drained, maximum lateral pressure will occur during and shortly after back-filling, dissipating somewhat with time. Another example is that of a hydraulic fill. The lateral pressure decreases as the fill drains. Borings will help in evaluating actual, in-place soil properties at the time that evaluation is made.

6-3 EVALUATING CONCRETE STRENGTH USING LOAD TESTS

6-3.1 **Method**. Apply the provisions of \1\ ACI 318/318R /1/, supplemented as described in the following paragraphs.

6-3.2 **Test Load**. Use test load magnitudes of 1.4D + 1.7L where live load is the reduced load (for tributary area). This increased loading intensity will require careful observation and control to preclude precipitating collapse. For this purpose, load in six increments, rather than four, and where feasible, use water-loading for safe provision of emergency drainage and load removal.

6-3.3 **Lateral Loads**. Simulate lateral loads expected to occur simultaneously with vertical loads in the test.

6-3.4 **Loaded Area**. Make the loaded area large enough to ensure that additional strength due to continuity and three-dimensional action within the structure is properly reflected in the test.

6-4 SPECIAL PROVISIONS REGARDING CAPACITY OF EXISTING PILES

6-4.1 **Structural Capacity**. Check existing piles checked for effects of deterioration. A reconnaissance survey should be made to identify areas of "worst conditions." Measurement of overall residual strength in 1 percent to 2 percent (but not less than 4) of the piles will be considered an adequate statistical sample on which to base judgment of capacity. Use the "worst" piles of the group as identified in the reconnaissance survey.

Give consideration to probable, future progression of loss of strength. Usually, the mudline under a platform has accreted well above the normal, stable slope line drawn from the existing dredge level alongside the platform. Discount this material in estimating the un-braced pile length (L). Should future dredging to greater depth be contemplated, consider the increased un-braced pile length that would result.

6-4.2 **Capacity of the Soil to Support the Pile**. Unless the embedded length of a pile has been reduced by scour or dredging, assume no reduction in bearing capacity from that initially achieved during driving. Where installed capacity is not known, consider the use of load tests to establish capacity.

6-4.3 **Sheet Piling**. Take the capacity of sheet piling to support vertical loads one-half the value indicated by conventional formulae relating capacity to driving resistance.

6-4.4 **Interpretation of Load Tests**. For interpretation of load tests refer to \\1\ UFC 3-220-01N, *Geotechnical Engineering Procedures for Foundation Design of Buildings and Structures* and UFC 3-220-10N, *Soil Mechanics*. /1/

CHAPTER 7

DETERIORATION OF WATERFRONT STRUCTURES

7-1 **GENERAL.** Considerable treatment of this topic may be found in UFC 4-150-07 *Maintenance of Waterfront Facilities* and UFC 4-150-08 *Inspection of Mooring Hardware.*

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APPENDIX A

REFERENCES

\1\ GOVERNMENT PUBLICATIONS

1. Unified Facilities Criteria (UFC): http://dod.wbdg.org	\1\ UFC 1-200-01, *General Building Requirements* UFC 3-220-01N, *Geotechnical Engineering Procedures for Foundation Design of Buildings and Structures* UFC 3-220-10N *Soil Mechanics* UFC 3-301-01, *Structural Engineering* UFC 3-570-02N, *Electrical Engineering Cathodic Protection* UFC 3-600-01, *Fire Protection Engineering for Facilities* UFC 4-025-01, *Waterfront Security Design; currently in final draft and near final publication* UFC 4-150-06, *Military Harbors and Coastal Facilities* UFC 4-150-07, *Maintenance of Waterfront Facilities* UFC 4-150-08, *Inspection of Mooring Hardware*

	UFC 4-152-01, *Design: Piers and Wharves* UFC 4-159-03, *Design: Moorings* /1/
Unified Facilities Guide Specification http://www.wbdg.org/references/pa_dod.php	\1\ UFGS 03 31 29, *Marine Concrete* UFGS 26 42 13.00 20, *Cathodic Protection by Galvanic Anodes* UFGS 26 42 19.00 20, Cathodic Protection by Impressed Current UFGS 31 41 16, *Metal Sheet Piling* UFGS 31 62 13.13, *Cast-In-Place Concrete Piles* UFGS 31 62 13.20, *Precast/Prestressed Concrete Piles* UFGS 31 62 16.16, *Steel H-Piles* UFGS 31 62 19.13, *Wood Marine Piles* UFGS 31 62 21, *Piling: Composite, Wood and Cast In-Place Concrete* UFGS 31 63 16, *Auger Cast Grout Piles* UFGS 31 63 26, *Drilled Caissons*

	UFGS 35 59 13.13, *Prestressed Concrete Fender Piling* UFGS 35 59 13.14 20, *Polymeric Fender Piles* /1/

NON-GOVERNMENT PUBLICATIONS

American Society for Testing and Materials (ASTM) 100 Bar Harbor Road West Conshohocken, PA 19428-2959 http://www.astm.org	\1\ ASTM A6/A6M, Standard Specifications for General Requirements for Rolled Structural Steel Bars, Plates, Shapes and Sheet Piling ASTM A36/A36M, Carbon Structural Steel ASTM A242/A242M, High-Strength, Low-Alloy Structural Steel ASTM A252, Welded and Seamless Steel Pipe Poles ASTM A588/A588M, High-Strength, Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point with Atmospheric Corrosion Resistance ASTM A690/A690M, High-Strength Low-Alloy Nickel, Copper, Phosphorus Steel H-Piles and Sheet Piling with Atmospheric Corrosion Resistance for Use in Marine Environments ASTM A709/A709M, Structural Steel for Bridges

	ASTM C42/C42M, Obtaining and Testing Drilled Cores and Sawed Beams of Concrete ASTM D25, Specification for Round Timber Piles ASTM D245, Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber /1/
American National Standards Institute (ANSI) 1899 L Street, NW Washington, DC 20036 http://www.ansi.org	\1\ ANSI O5.1, Class 3 Poles /1/
\1\ American Wood Protection Association (AWPA) P.O. Box 36784 Birmingham, AL 35236-1784 /1/ http://www.awpa.com	\1\ AWPA Standard U1 /1/
American Concrete Institute (ACI) International P.O. Box 9094 Farmington Hills, MI 48333 http://www.aci-int.org	ACI 318/318R, Building Code Requirements for Structural Concrete and Commentary
Portland Cement Association 5420 Old Orchard Road Skokie, IL 60077 http://www.cement.org/	\1\ PCI STD 112, Standard Prestressed Concrete Piles Square, Octagonal and Cylinder /1/
National Fire Protection Association (NFPA) 1 Batterymarch Park Quincy MA 02269-9101 http://www.nfpa.org	NFPA 307, Standard for Construction and Fire Protection of Marine Terminals, Piers, and Wharves

/1/