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Great Lakes Small-Craft Harbor and Structure Design for Ice Conditions: An Engineering Manual



by C. Allen Wortley ◻ University of Wisconsin Sea Grant Institute

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C. Allen Wortley

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"I may add, that the ignorance, or want of proper appreciation, of the properties of ice, evinced in the construction of numerous wharves, piers, and bridges on the inland lakes and rivers of Canada and the northern States, has proved a source of infinite annoyance and of immense expense."

-- J. H. Dumble
"Ice Phenomena"
from Observations on Rice Lake

Preface

I first became hooked on ice 15 years ago. At the time, I was a design engineer in private consulting practice with a commission to design a marina for a Lake Superior harbor. I did--and my docks sunk, crushed by the first winter's ice! Since that experience, ice has fascinated me.

Ten years ago, I elected to change careers, and rather than design engineering works I chose to research and teach about them. The University of Wisconsin Sea Grant Insitute was interested in Great Lakes harbor construction, and thus began a continuing research effort that now has spanned a decade.

This UW Sea Grant effort has involved extensive field observations of winter conditions in harbors throughout the Great Lakes. The effort has also included some field and laboratory experimentation, literature searches and information-gathering, and the planning of conferences and seminars about ice and small-craft harbors.

Very few simple solutions to the problems of design have been found. However, as set forth in this manual, I hope you will agree that progress is being made.

My approach in writing the manual is to clearly and simply say what we know, and what we don't know, about small-craft harbor design for ice conditions so that you, the designer and builder, can do the best job possible.

I anticipate that you will read this more as a treatise narrative than a step-by-step manual procedure: the latter is simply not possible at this time.

Dumble was a railway engineer in 1858 who was annoyed with the effects ice had on his constructions. Let's hope yet another hundred years doesn't go by and we're still annoyed!

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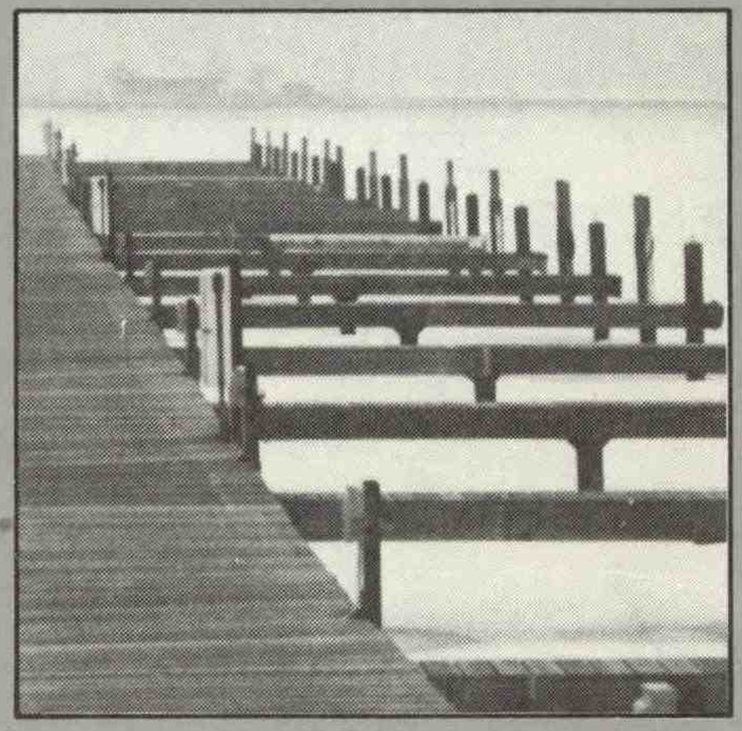
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Part One:
REVIEW AND INTRODUCTION



1. Introduction

This University of Wisconsin Sea Grant advisory report is a "how-to" manual based on many people's experiences--a generalized synthesis of my Sea Grant Advisory Services field work, ice engineering research and observations both in the Great Lakes region and abroad.

This manual presents technical information both useful and necessary in designing and building small-craft harbors and related structures in areas where ice is a factor. I have tried to present the "why" for each "what," but this was not always possible: in some situations, the "what" is unknown--that is, we don't yet know what should or can be done.

A small-craft harbor, or marina, is a man-made structure in a water-soil environment. The resulting structure-water-soil interaction is quite complex. In northern climates, winter freezing creates an even more complex ice-structure-water-soil interaction--which is what we are trying to understand and accommodate here. Experience has demonstrated at best the annoyance and at worst the immense expense of our lack of knowledge about this interaction. However, as engineers and contractors, you can design for this interaction. This manual endeavors to help you do just that.

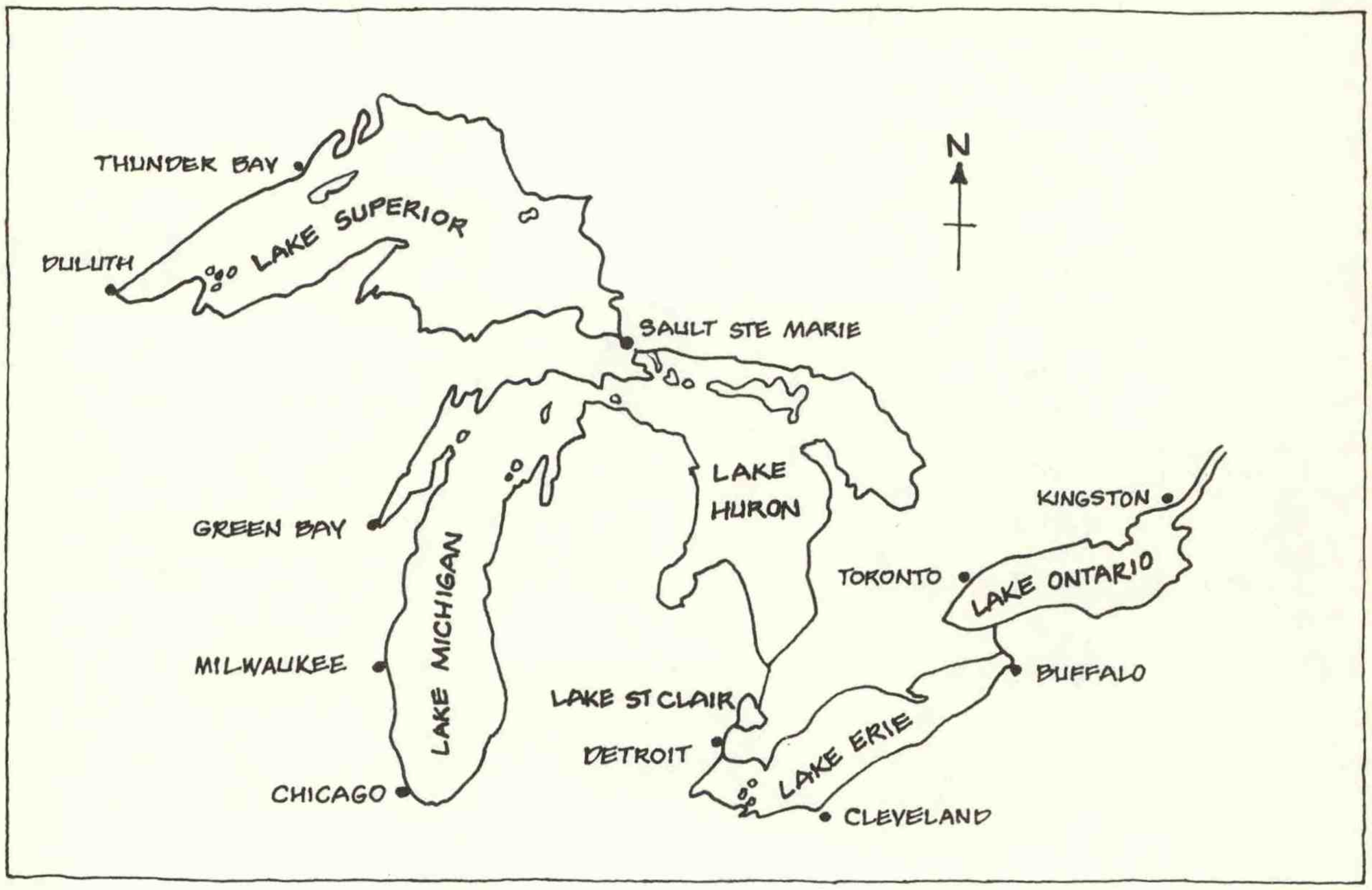


FIGURE 1.1: The Great Lakes

The bibliography is extensive; some of it is annotated. It can direct you to more detailed information regarding ice, ice engineering and geotechnical engineering. I have listed many sources with useful information on this subject. If you want more details or need to clarify something, consult the original sources. The literature search, as of mid-1983, is done.

This manuscript is organized into four parts. To develop the necessary basic understanding of the problem, I recommend reading all four. Part One is a review and introduction. Geotechnical engineers should pay particular attention to the two chapters on ice engineering, and ice engineers should pay particular attention to the chapters on soil mechanics and foundation engineering. After the introduction and review, information about preliminary design, detailed design and some aspects of construction is presented. These parts and their chapters comprise the body of this manual.

I do not endorse any of the companies or products, either named or implied, that are cited in this manual; they are mentioned only in context. If you want assurances, you'll have to check them yourself.

The units of measurement used in this manual are from the U.S. Customary set. If you usually work with SI Metric, see Table 1.1 for a few basic conversions.

TABLE 1.1: Conversion Factors for U.S. Customary to SI Metric Units

1	inch	=	25.4	millimeters
1	foot	=	0.305	meters
1	pound	=	4.45	newtons
1	kip	=	4.45	kilonewtons
1	kip per foot	=	14.6	kilonewtons per meter
1	pound per square foot	=	47.9	pascals
1	pound per square inch	=	6.89	kilopascals
1	kip per square inch	=	6.89	megapascals
1	mile per hour	=	0.447	meters per second
1	cubic foot per minute	=	0.000472	cubic meters per second
1	BTU per hour per square foot	=	3.15	watts per square meter
1	BTU per hour per square foot per degree Fahrenheit	=	5.68	watts per square meter per degree Celsius

2. Ice and Ice Covers — An Introduction

This chapter presents an introduction to ice and ice covers on bodies of water. It includes information on the structure of ice and its genesis, the freeze-up and break-up of ice covers, and some observed behaviors of lake ice. The next chapter presents introductory information on ice engineering. For detailed information on the physics and mechanics of ice, I recommend Glen's (1974, 1975) monographs and the textbooks by Pounder (1965), Hobbs (1974) and Michel (1978). The last reference is a state-of-the-art book.

THE STRUCTURE OF ICE

Three peculiarities distinguish ice from other materials. First, ice usually exists in nature at a temperature near its melting point. Ice on a lake has a bottom surface temperature equal to its melting point, and an upper surface temperature that varies with ambient conditions. Therefore, ice usually contains some liquid water between its crystals, or grains, as they are sometimes called. This liquid water acts as a lubricant and reduces slide resistance between crystals.

The second peculiarity is that ice does not react chemically with impurities and forms no solid solutions with them. The impurities accumulate gradually along crystal boundaries and form many pores, cavities, etc., in the ice.

The third peculiarity is the relatively large size of ice crystals. Their transverse dimensions vary from less than a tenth of an inch to several inches and sometimes several feet. By comparison, in metals these dimensions usually measure in fractions of hundredths of an inch. These peculiarities cause ice to be a material that in its totality appears to consist of plastic hinges, so it reacts sharply to static loads (Lavrov 1969).

Many forms of ice exist, but here we are concerned with only two ordinary, natural forms of ice: freshwater ice and sea ice. This manual will sometimes mention sea ice, or saline or brackish ice, because harbors with these types of ice have similar problems to those on the freshwater Great Lakes.

Mellor (1983) notes that it is useful to regard sea ice not as some unique and exotic material, but simply as a variation of freshwater ice. Sea ice is formed by direct freezing of sea water and is an anisotropic (i.e., having different properties in different directions) crystalline solid containing bubbles of air, pockets and films of brine, and sometimes solid salt. The complications introduced by salinity and structure are important, but not to the extent that sea ice need be regarded as something wholly different from freshwater ice.

Each crystal of ice is made up of water molecules arranged in a regular geometric pattern forming hexagonal prisms. Each mulecule has an oxygen atom with chemical bonds with two hydrogen atoms. A good deal of space exists between molecules; hence, ice has a rather open structure.

Because of this fact, water--unlike almost all other substances--is less dense in its solid state than when liquid. In short, ice floats.

The unit weight of both freshwater and sea ice is about 57 pounds per cubic foot (9 kN/m^3). Both types of ice are composed of grains of relatively pure ice. The shape of each individual grain--essentially a single crystal of ice--can vary from granular to tabular to columnar, and the size of each grain can vary widely as well. However, each grain has hexagonal crystallographic symmetry. One axis of symmetry--called the c-axis, or the optic axis--is perpendicular to an important plane of deformation called the basal plane. The resistance of the basal plane to viscous shear is small. The resistance of nonbasal planes to viscous deformation is 10 to 100 times larger. As a result, each grain tends to deform like a pack of cards when subjected to a shear stress.

Natural ice is polycrystalline and composed of many grains. This means that the bulk properties of ice will be some kind of average of the single crystals of which it is composed. For some properties, like elasticity, this averaging is fairly straightforward; but for others, like plastic deformation, there are strong anisotropic effects. This anisotropy in the deformation behavior of individual grains has important implications for the way an ice cover (i.e., the sheet ice on a lake, river or sea) responds to stress.

Moreover, the grain size--as well as the crystal orientation of polycrystalline ice--has an effect on the strength and behavior of an ice sheet. Ice covers are subject to large variations in their properties, both spatially and with time, because their growth depends on weather and water conditions.

GENESIS OF ICE

In the absence of very great supercoolings or supersaturations, ice will form only if some nucleus is present on which it can grow. This can be ice itself: if an ice seed is present, ice will normally develop and grow. The nucleus can also consist of some other material. But once a nucleus is present, ice can grow from the surrounding water as long as the temperature remains favorable.

How the ice grows depends on whether the water surface is supercooled and the amount of impurities it contains. Michel and Ramseier (1969) classified river and lake ice based on its genesis, structure and texture. Their classification system is a standard and includes: primary ice, secondary ice and snow ice.

Primary Ice

Primary ice forms first on a body of water. On calm water, primary ice is in the form of an ice skim that grows horizontally in the supercooled layer and is about a hundredth of an inch thick. Usually when water freezes, little plate-like discs form. Depending on the amount of supercooling and the rate of heat loss, they may rapidly develop dendritic extensions, or branch out.

Since their planar structures will naturally float with planes parallel to the surface, the first ice that forms often has its surface crystals oriented with their c-axis vertical--that is, perpendicular to the free surface. These

initial crystals spread over the surface until they interact and form a layer over the whole surface of the water. The subsequent growth of this layer downwards into the water is not usually dendritic.

In rough and turbulent water, primary ice consists of congealed frazil slush up to an inch thick. Frazil slush is a floating agglomerate of loosely packed individual ice crystals having the form of small discoids or spicules, which are formed in supercooled turbulent water, such as a fast-flowing stretch of river. Congealed snow slush (i.e., a loosely packed agglomerate of floating snow particles) also may be part of the primary ice.

Secondary Ice

Michel and Ramseier (1969) include several subtypes of secondary ice. While we will ultimately be concerned more with the macro-properties of ice, for background purposes subtypes S1, S2 and S4 will be described here.

Secondary ice forms parallel to the heat flow, which is perpendicular to the primary ice. As the ice grows down into the water under conditions of calm formation and growth, the grain boundaries are almost perpendicular to the surface. The result is S1 ice: a columnar-grained structure with the c-axes vertical in the long direction of the grains. S1 ice can be found in lakes, reservoirs and in slow-flowing rivers.

S2 ice has horizontal c-axes and can be found in lakes, reservoirs, rivers and along shores. It forms under conditions similar to those for S1 ice, but the primary ice has a random orientation, or a preferred vertical orientation superimposed on a random orientation. As the ice grows down into the water, the c-axes vertical preferred orientation gives way to one in which the c-axes are horizontal. Some crystals are wedged out at the expense of others. When ice grows, the growth occurs more readily in the direction of the basal planes, which means a tendency for the c-axes to become horizontal.

Sea ice has a strong preferred-growth fabric, with the crystals elongated vertically parallel to the heat flow and the c-axes of the crystals oriented almost perfectly in the horizontal plane. Nearshore lake ice also has a tendency to be S2.

S4 is a granular rather than a columnar secondary ice, and it is composed of congealed frazil slush. Its crystal boundaries are irregular, and the grains highly angular and randomly oriented. Frazil is found in rivers where it is taken downstream and deposited upwards under the secondary ice overlying slower water. S4 ice is found in rivers and reservoirs, and in lakes fed by turbulent waters. An entire ice cover can consist only of S4 ice, or S4 ice can be found layered between columnar forms of ice.

Snow Ice

Snow ice, referred to as T1 by Michel and Ramseier (1969), is the third basic form of ice. In general, snow ice is formed from any imaginable kind of water source. It may form due to variations in water discharge, melting or rainfall, or by depression of the ice cover due to a heavy load of snow. This type of ice always forms on top of the primary ice and lies over the ice cover. Its grains are round to angular and randomly oriented. As it is granular, it is similar in appearance to S4 ice.

Impurities in Ice

The most common impurity in freshwater ice is air. It is an integral part of granular ice formed from snow and gives snow ice its whitish appearance. Since air molecules are not easily incorporated into the ice-crystal lattice, they are rejected at the ice-water interface as the ice grows downward. This rejected air is supplemented by gases arising from the biochemical processes occuring in natural bodies of water. Air can also be incorporated into the ice from the atmosphere through cracks, drained snow ice or an agitated water surface.

If ice growth is slow, most of this air will be rejected, and the ice will as transparent as glass. Transparent ice is called "black ice," or sometimes, mistakenly, "blue ice"--a term often used in connection with glacial ice. However, I have seen 12-inch-thick slabs of Lake Superior ice that were an azure blue--most striking in appearance.

Whether black or blue, we have clear transparent ice, and we have opaque, white milky ice. The ice you make in the refrigerator freezer compartment is usually white ice; whereas, the machine-made ice you buy is clear ice, mainly to be more appealing to you. It is made more slowly and under controlled temperature conditions.

Ice covers are not simple structures. Figures 2.1 and 2.2 are sections cut from Lake Superior harbor ice. Note the snow ice and transparent ice, and the ice sheet variability.

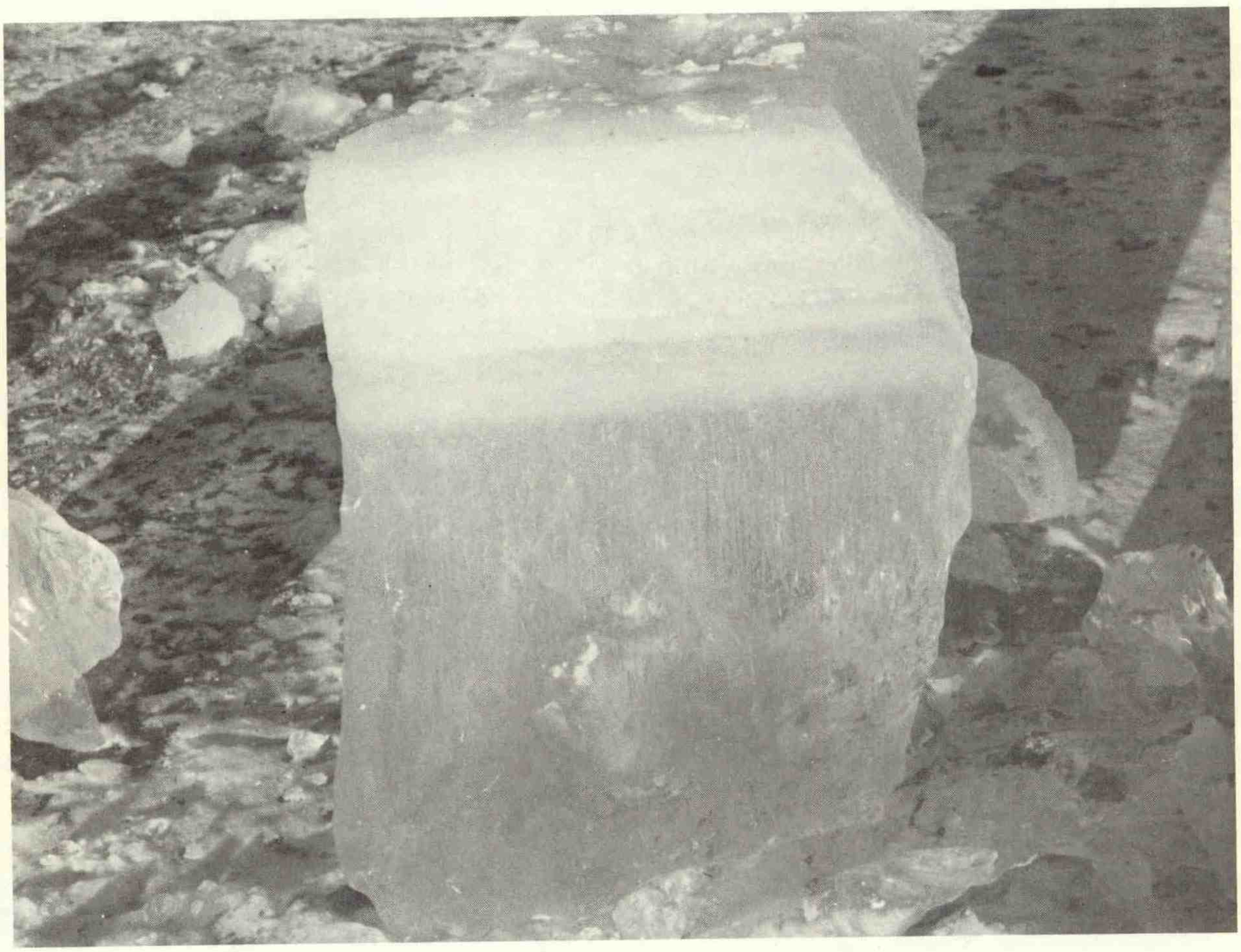


FIGURE 2.1: Ice Cross Section Showing Snow Ice Over Clear Ice

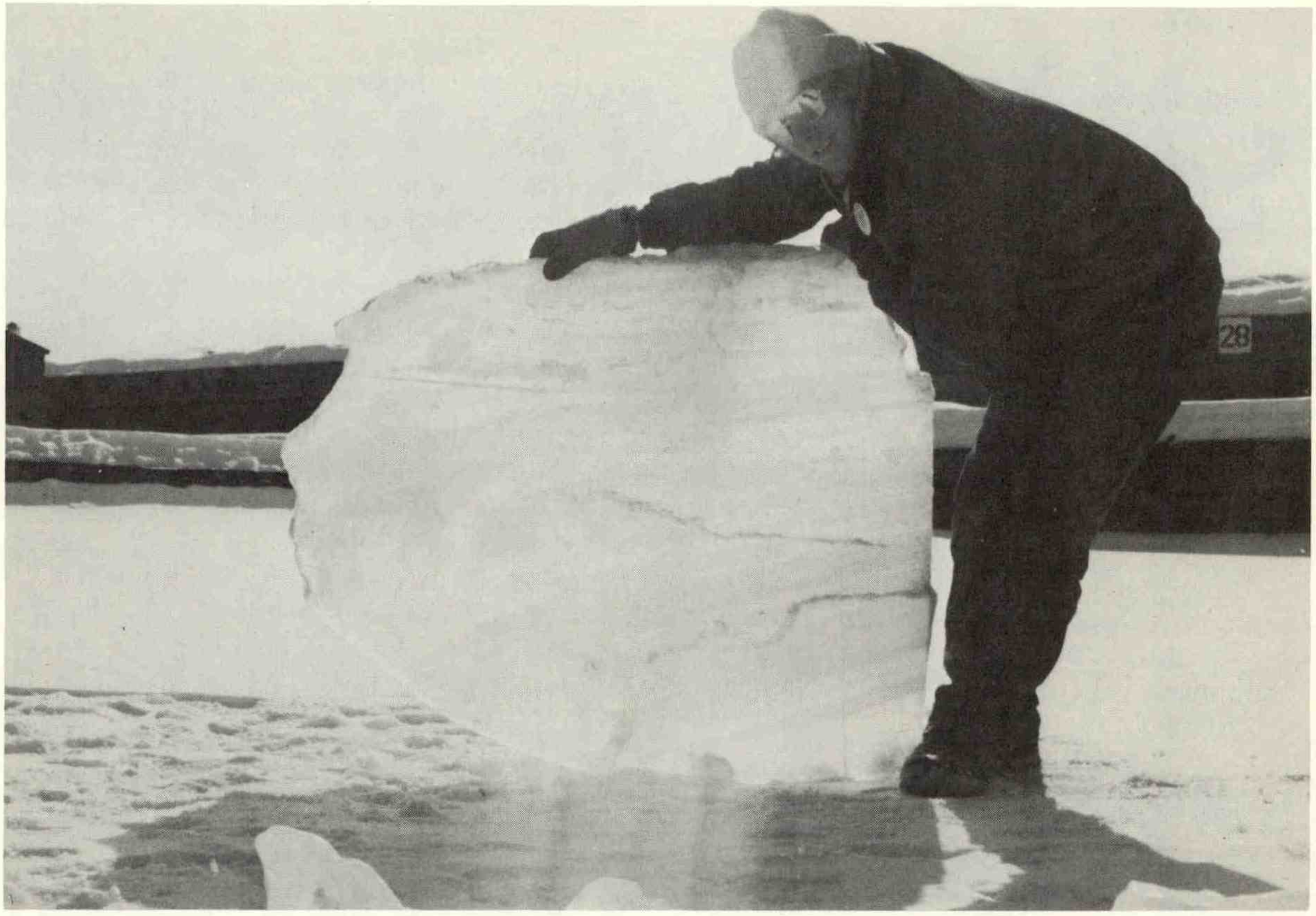


FIGURE 2.2: Ice Cross Section Showing Variability of Ice Sheet

In sea ice, the principal impurities are salts. Like air, these salts are rejected during freezing, and they become more concentrated in the water immediately in front of the ice-water interface. When the concentration is sufficiently high, the interface becomes unstable, and the salt is incorporated into the ice as brine pockets. These brine pockets affect the strength and deformation behavior of sea ice, which is weaker than freshwater ice and tends to be columnar rather than granular in structure.

FREEZE-UP AND BREAK-UP OF ICE COVERS

One of nature's anomalies is water's negative coefficient of expansion between 39°F and 32°F, where water volume increases as the temperature decreases. Since the volume of a given mass of water is smallest at 39°F (4°C), this is the temperature of its maximum density. Water warmer or colder than 39°F is lighter.

Typical sea water is about 3.5 percent salt, its freezing point is 29°F, and its most dense state also occurs at 29°F. The salinity is so high (i.e., greater than 2.5 percent) that the density of the water increases continuously with decreasing temperature down to the freezing point.

A number of authors have described how lakes freeze and thaw. Among them are Bengtsson (1981), Gerard (1983b), Hinkel (1983), Michel (1971) and Williams (1966). The following observations and descriptions are from these and other sources.

Fall Freeze-Up of Ice

Freeze-up occurs in the fall, the ice grows thicker during the winter, and the ice breaks up in the spring. This is the general ice cover scenario. The ice can melt out in midwinter, as happened in February 1984, or it can be broken up and dispersed by wind and waves during winter storms. But for now let us assume it forms in fall, thickens over the winter, and melts in spring.

The formation of an ice cover on a lake is a function of the water's heat exchange with the atmosphere, the initial amount of heat stored in the water body, and the amount of inflow of warm water and earth heat to the site. The amount of heat lost to the atmosphere is a function of air temperature, wind velocity and solar radiation. The amount of heat that can be stored in a water body is a function of depth. Usually, the deeper the lake, the deeper the convective mixing and the slower the rate of water cooling for a given surface heat-loss.

A freshwater water lake cools in two stages: a gradual cooling until all the water is about 39°F, and cooling of the surface water from the time the water is isothermal (that is, the same temperature, top to bottom) at 39°F until the sheet forms.

First, the warm surface water cools down--often releasing its heat in a steam-like mist if the air is much colder (which may form hoarfrost on nearby trees if the air is below freezing). The water thereby contracts and becomes more dense. This "heavier" water sinks into the less-dense water of the lake, forcing the warmer waters below up to the surface to be cooled.

This process is repeated until the lake is isothermal at 39°F, at which point the lake is said to have "turned over." From this point onward, the lighter, cooler surface water continues to cool down, giving off its heat by convection, evaporation and radiation to the atmosphere until it freezes as sheet ice. A thin layer of ice first forms along the edges of the lake, because a convective air current, pulled in by the rising warm air over the lake, cools the lake edges more rapidly than the center. Since the density of ice is even less than that of freezing water, the ice floats on the water below it. Further freezing can only result from heat flow upward by conduction from higher temperatures to lower temperatures.

Winter Thickening of Ice

The permanent ice cover gradually thickens during the winter. When the ice cover is thin and the air very cold, a relatively large amount of heat from the water will flow through the ice to the atmosphere. This will cause a rapid increase in ice thickness at the interface between the ice and water. I have measured the growth of 28 inches of ice in just three weeks in a Lake Huron harbor at 46°N latitude when the average minimum daily temperature was 5°F. This rate is more than an inch a day.

The insulating effect of the thickening ice cover slows the rate of ice growth, and a snow cover on top of the initial ice cover can effectively prevent any further ice growth.

We know that ice grows downward into the water and also thickens on top from the formation of snow ice. The ice of a snow-covered frozen lake can vary

greatly in stratigraphy. Snow cover can conceal weak, slushy layers of ice as well as strong, sound ice, depending how on the ice formed.

Hinkel (1983) states that the rate of growth of an ice cover is strongly influenced by site-specific characteristics other than air temperature. Since the rate of ice growth (and decay) are functions of many interacting processes, it is necessary to recognize that sophisticated forecasting algorithms require more detailed input data than are presently available. A simple parastatistical model indicated that rates of ice growth for nearshore sites--which are complex in terms of operating processes and difficult to model over time and space--are strongly influenced by the snow cover and dynamic site-specific factors. These factors included water depth, incoming streams, underground springs, normal lake conditions and other items not pinpointed in the study.

Bengtsson (1981) notes that the thermal effect of the atmosphere results mainly in the build up or thawing of an ice cover, but it causes practically no change in the water temperature. Heat exchange with the bottom and through-flow are decisive external factors in the formation of the thermal conditions of ice-covered bodies of water. Mixing in ice-covered lakes is due to through-flow currents and convective currents generated by the heat transfer from the bottom sediments to the water.

When this heat is transferred to the water, it becomes warmer and denser. The water tends to slide along the bottom towards greater depths, so there must be a compensating upward current in the central part of the lake. Measurements in Swedish lakes found such current velocities to be on the order of a few yards per day.

A very pronounced thermocline develops in ice-covered lakes. In a lake with through-flow, the depth of the thermocline is determined by the inlet and outlet conditions. In a lake with no through-flow, the thermocline is very close to the underside of the ice. Over the winter, the bottom sediments heat the lake water, but this heat does not pass through the stratification, and so the thermocline becomes more and more pronounced. The heat transfer rate from sediments has been measured at rates as great as a BTU per hour per square foot in early winter, dropping to one-third to one-half BTU in midwinter.

An ice-covered river has a thermal profile that is the same throughout its depth, and the temperature of the water is only a few hundredths above freezing (an ice-covered river could be a reasonably good place to check the calibration of temperature-measuring devices).

Ice covers on sea water do not protect a reserve of heat except under special circumstances. The temperature of the ice and the water are nearly the same--29°F. Depending on salinity, a temperature gradient may exist in ice-covered brackish water.

Spring Break-Up of Ice

The clearing of ice from bodies of water at spring break-up is affected by heat gain from the atmosphere, snow and ice conditions, wind and currents, and inflow or runoff of warm water to the site. The heat gained from the atmosphere weakens and melts the ice. The amount of solar radiation

absorbed is determined by the reflectivity of the snow or ice surface. Various properties of the ice cover determine the depth to which solar radiation will penetrate and cause internal melting.

Impurities lower the freezing points of solutions, and impurities in ice accumulate at grain boundaries, so ice covers begin to melt on grain boundaries. Because of its fine grain size and opaque character, snow ice, after some grain boundary melt, loses its strength more slowly than clear, columnar ice. Moreover, snow ice protects any underlying columnar ice from penetrating solar radiation.

The ice around the shoreline tends to melt first, partly because of the sunlight-absorbing darker surface layers produced by slush from runoff, but also because the ice is usually thinner close to shore, where it can be no thicker than the water is deep. Eventually, the ice cover completely melts around the shoreline, leaving the main body of ice floating free. These free-floating bodies of ice can still have considerable strength, however.

During melting, drainage holes can develop in the ice sheet. In the early stages of the ice melt, these holes develop near the shore, where runoff flows into old thermal cracks. Such drainage holes may enlarge rapidly, sometimes developing into holes 1-2 feet in diameter.

In the later stages of melting, the free-floating main body of ice melts at the surface. As the surface-melt water drains away, the ice surface becomes a porous, crumbly white, which is highly reflective and retards melting by solar radiation. As the melt season progresses, penetrating solar radiation causes internal melting in the ice sheet. The ice sheet may then consist of a shallow, porous surface layer several inches thick, below which is a layer of water-logged ice also several inches thick, and then solid, unmelted ice whose lower surface is melting in the lake water.

In the final stages of break-up, the underlying entrapped layers of water darken the surface ice, so most of the incoming solar radiation is absorbed. The ice is now ripe for break-up by wind and currents and is unsafe for over-ice transportation. The current created by strong winds will break up the ice cover and induce circulation that brings the warmer subsurface water to the top. This can cause rapid melting of ice from a lake. Indeed, the final disappearance of ice covers has occurred so quickly that early observers thought that the ice actually sank. (The idea of sinking ice seems farfetched, but I sometimes wonder: I have walked on water-logged ice with more than three inches of standing water, and it indeed seemed to be sinking!)

The phrase "rotten ice" is sometimes used to refer to disintegrating ice at break-up time. If the ice has a columnar structure, it becomes candled and is referred to as candled ice (see Figure 2.3). Melting is concentrated at the boundaries of the columnar prisms leaving a weak candle-like structure.

OBSERVED BEHAVIOR OF ICE SHEETS

Anecdotal accounts of ice behavior are contained in the literature. During the 19th century, Dumble (1858), an engineer with a railway company,

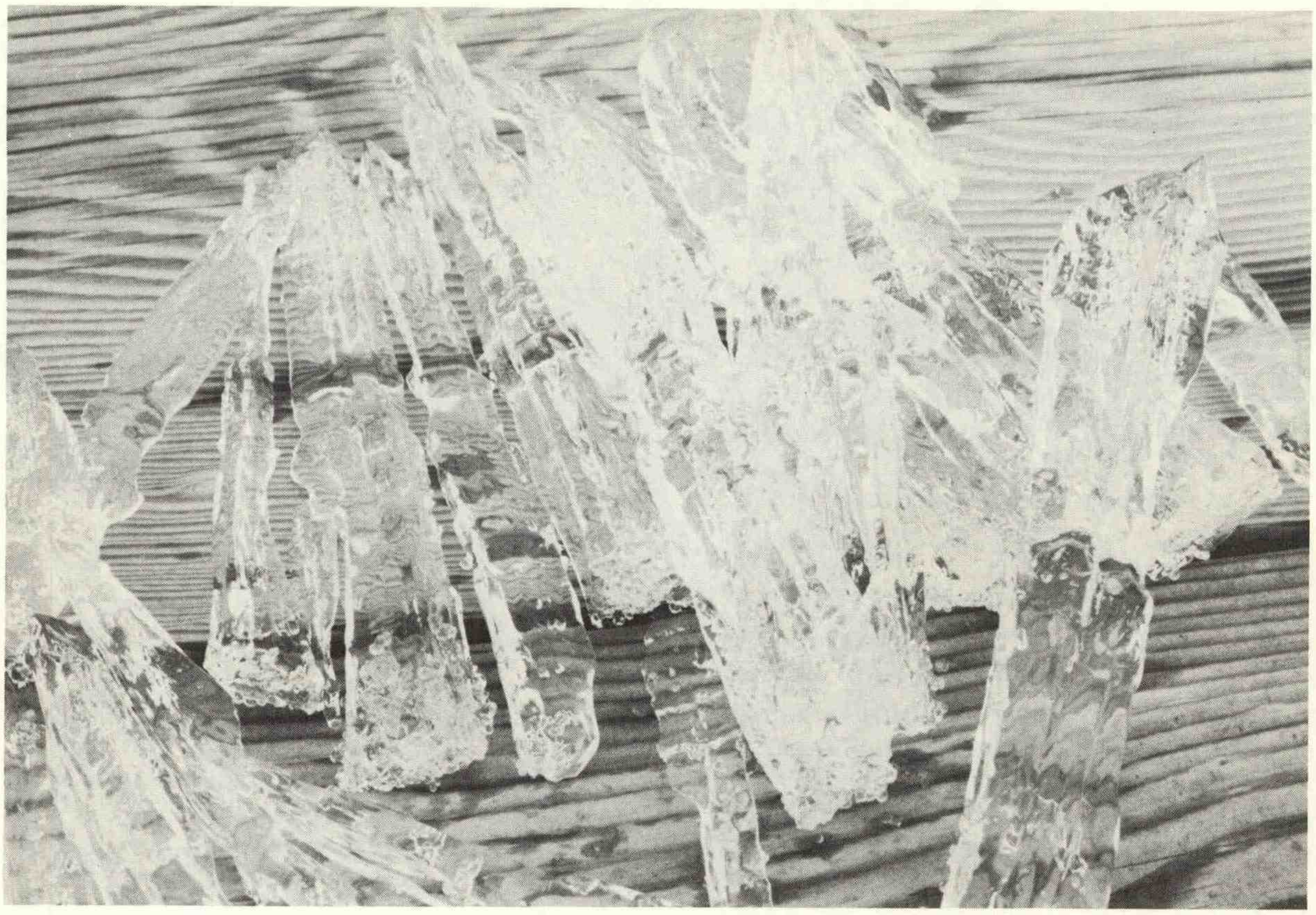


FIGURE 2.3: Candled Ice

published his observations on inland lakes and rivers in the northern states and Canada. A few of his observations are repeated here with some of my comments:

* The most violent shoves of ice occur prior to rainstorms. This is associated with warm winter weather and thermal expansion.

* Contraction generally occurs at night and is accompanied by sharp reports. This is associated with cooling nighttime weather conditions and thermal cracking.

* A coating of snow more than six inches thick effectively prevents any motion of the ice below. Snow is a very effective insulator and prevents thermal motion of ice.

* It is reasonable to suppose that any solid that is equally dense throughout its dimensions and susceptible to expansion--when equally acted upon by the active agent or moving cause--would expand from its center towards its circumference. This is the effect produced on any large field of ice of equal thickness and density, when acted upon uniformly by either the midday sun or warm winds. It is a fact, however, that it moves from directions other than the center of the lake.

* Ice, owing to the peculiar circumstances under which it sometimes forms, is not found to be equally pure or dense, neither is it of uniform thickness. This ice irregularity, acted upon by warm winds or by the slanting rays of the sun at different altitudes, shoves or expands from various directions other than from the center of the lake. Shoving is from the stronger and most susceptible ice, toward the weaker, less expansive.

More recently Striegl (1952) described the formation of ice in the Great Lakes, particularly Lake Michigan. Engineering structures built in the lakes for harbor protection or other purposes must be designed to support enormous ice loads. These loads are due to the ice build-up on the structure by continued freezing of wave wash and spray, or to loose ice piled on top of the structure by ice jams, which is then consolidated by continual freezing. These ice loads may pile up 20 to 25 feet above the top of structures.

Because of the extra support and bridging effect of the ice itself, ice loads usually do little damage by way of overloading individual structural members, but they may cause considerable damage in other ways. Through freezing and thawing action, stones or concrete may be broken to such sizes that they later will be displaced by wave action. Breakwater stones weighing several tons may be encased in large ice blocks and carried bodily from the structure. Ice floes carry such stones miles from the original structures and drop them in shipping channels, obstructing navigation.

A huge ice load was created in January 1948, when a strong easterly windstorm piled ice in a solid mass along the entire Illinois Lake Michigan shoreline. The ice was as much as 15 feet above the water surface and extended 50 to 100 feet in front of the beaches and natural shorelines at other points.

3. Introduction to Ice Engineering

This chapter introduces many of the basic concepts relevant to small-craft harbor concerns. These concepts include deformation and strength of ice, ice adhesion and friction, ice forces, bearing capacity of ice, ice pressures and other fundamentals.

DEFORMATION AND STRENGTH OF ICE

Ice covers exist at a temperature no more than 100 F° from that at which they melt. In this thermal state, they are viscoelastic or viscoplastic materials that are time-dependent (viscous) as well as dependent on temperature, stress and other factors. (Gold 1973, 1978).

When ice is strained slowly, it behaves in a ductile manner. At high strain rates, it is brittle. At strain rates in between, it can be either ductile or brittle, and the largest strength seems to occur somewhere in this range--that is, neither in a brittle nor a ductile zone. These terms are not well defined.

If the period of loading for polycrystalline ice is one second or less, it can be assumed to respond elastically to failure. For lower loads, the period of elastic behavior is longer and increases with decreasing temperature. At ice temperatures below 23°F and stresses less than 75 psi, response is essentially elastic up to 100 seconds. This means that elastic theory can be applied to practically all bearing-capacity problems involving moving loads and ice force problems for which the imposed strain rate is greater than 0.001 per second. For example, if a 10-inch sample deformed 0.1 inch in a laboratory compression test in a 10-second period, the strain would be 0.1/10 = 0.01, and the strain rate would be 0.01/10 = 0.001 per second, usually written $10^{-3}s^{-1}$.

For many ice engineering problems, the conditions experienced in the field or assumed for design are no longer elastic but involve viscoplastic effects. The longer the load is left on, the more the ice will deform. Therefore, strain is a function of time. This is termed "creep behavior," which is subdivided into primary creep, secondary creep and tertiary creep. Strain can also be a function of other parameters, such as stress.

Creep Behavior

When the load is first placed on the ice, the ice has an instantaneous elastic response, rather like elastic compression in a spring. If the load is too great, the ice may break before creep can take place. However, assuming that the load is moderate, the ice will next exhibit a delayed elastic response. This response is comparable to the travel in a dashpot--like a plunger moving in a viscous fluid. Together, these two responses make up the creep period called primary creep.

During secondary creep, the strain increases with time; if the ice behaves as a linear viscoelastic material, this strain is linear.

At some point, the strain will accelerate. This increasing strain is the tertiary creep. Under a compression load, the ice compresses at a faster and faster rate until it fails. At lower stresses, ice may not exhibit a tertiary creep but may fail during secondary creep.

Ice covers can be sampled and tested in the laboratory, or ice specimens can be prepared and tested. Creep behavior for a granular ice tested at a constant stress of 145 psi and temperature of 14°F for six days is reported by Gold (1973) (see Figure 3.1a). Primary creep lasted a half a day and represented about one percent strain; secondary creep, 3.5 days and three percent strain; and tertiary creep, six days and the sample strained about 10 percent.

Michel (1981) presents a simplified rheological model for creep of ice for engineers. In many engineering problems, the condition of loading is that of constant speed at high rates for the design of structures. Under these conditions, the creep curve is very steep at the origin and the primary creep not very significant. It can then be simulated by a perfect elastic-plastic material.

For very high loading rates with strain rates greater than 0.1 per second, the ice behaves like a perfectly elastic material and fails brittlely by crack propagnation, either in tension or compression. The elastic modulus is the dynamic modulus that gives the instantaneous elastic deformation.

For strain rates between 0.1 and 0.00001 per second, the ice will fail in a transitional zone, sometimes brittle, sometimes as a perfect elastoplastic body. The maximum yield strength of ice in compression will then be about 1.5 times the brittle compressive strength, but in tension it will keep the same value.

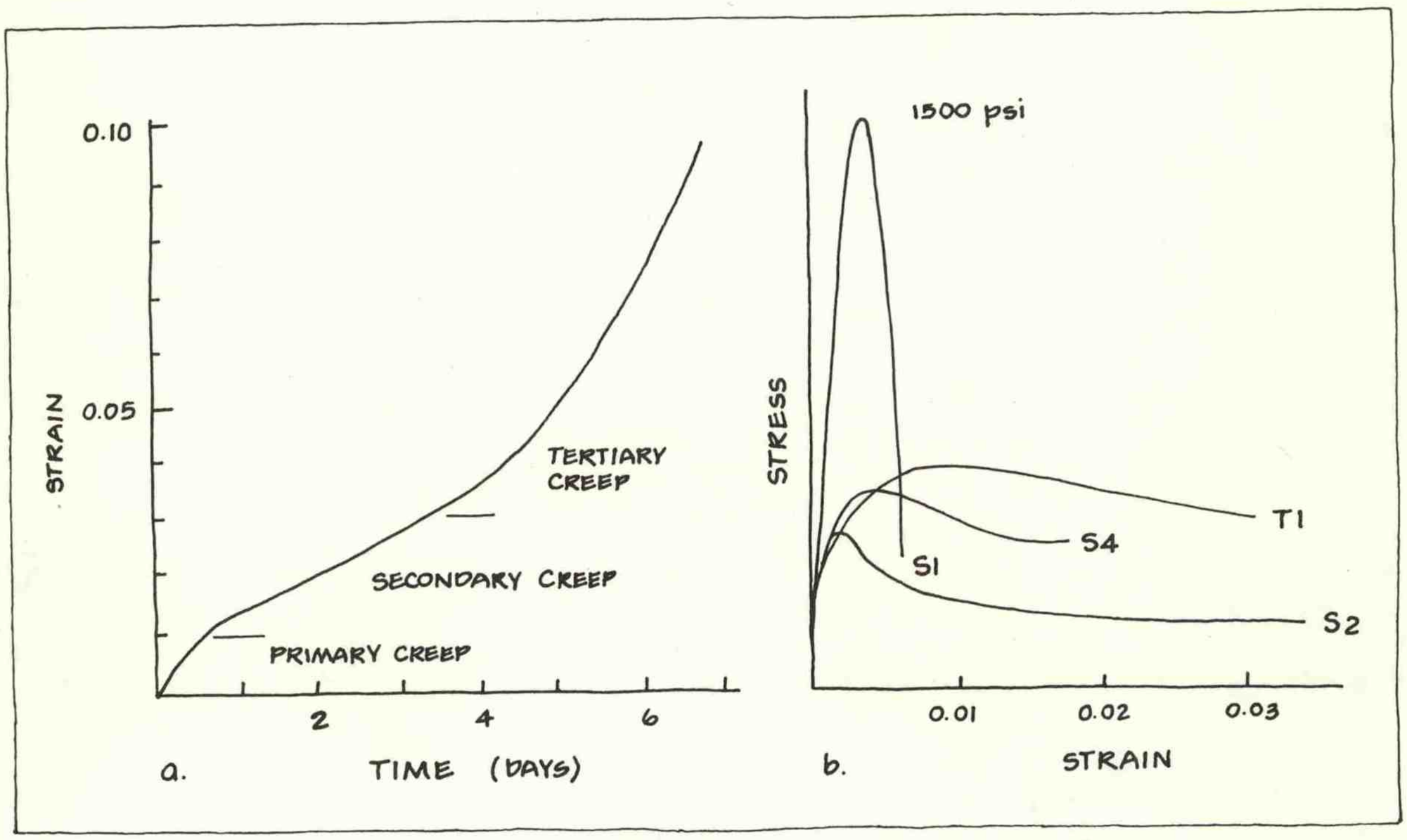


FIGURE 3.1a: Creep Curve for Granular Ice 3.1b: Strain Dependence on Stress (after Gold 1973)

Constant Rate of Strain

Gold (1973) also reports constant rate of strain, instead of constant stress, tests on samples of S1, S2, S4 and T1 ice (see Figure 3.1b). Some engineering problems involve conditions approaching constant strain. The deformation of the ice is very much dependent on the ice type. The columnar S1 and S2 samples were loaded perpendicularly to the long direction of the columnar grains of the ice. T1 is granular ice, and S4 is frazil ice. The temperature of the ice was 15°F, and the strain rate 1.67 x $10^{-5}s^{-1}$. The time to reach one percent strain was 10 minutes, and the time to reach three percent was a half hour. For this strain rate and ice temperature for these samples, the S1 ice strained less than one percent and behaved brittle-ly. It had a much higher maximum stress than the S2 columnar and the S4 and T1 granular ice samples. This is due to the fact that when the applied stress is perpendicular to the long direction of the grains, there is no resolved shear stress on the basal planes, the planes of easy slip.

Describing Ice Behavior

To adequately describe the deformation and strength behavior of ice, many things must be considered--the type of ice, the temperature, the type of load (compression, tension, etc.), load direction with respect to ice structure, crystal size, strain rate and stress load. To analyze ice in an engineering context, it is also necessary to establish failure criteria in either ductile or brittle modes of behavior under a variety of boundary conditions.

Ice behavior is not yet completely understood. Conflicting reports about its behavior add to our lack of understanding. For example, because snow ice has more grain boundaries than columnar ice, one source says that snow ice is weaker than columnar ice--while another says that fine-grained ice is stronger than columnar ice. Other such conflicting statements exist in the literature, so be wary.

Also, be careful when using published laboratory strength-test data from ice specimens (or from the simulated specimens required for physical modeling similitude), as the values determined by such testing may not be represent-ative of conditions in the field. For example, uniaxial laboratory strength tests may not represent biaxial and triaxial conditions encountered in the field because of the influence of shear stress in the deformation and failure of ice. Also, small samples generally do not adequately represent conditions in large, irregular, cracked sheets of ice containing various impurities.

The Michel (1978) reference book on ice mechanics deals with the basics and some engineering problem applications. Also, the new monograph by Mellor (1983) presents good basic technical information on the mechanics of deform-able solids and the mechanical properties of ice. Subsequent sections of this manual will introduce strength parameter values applicable to ice engineering problems in small-craft harbors.

FRICTION AND ADHESION OF ICE

Ice adheres tenaciously to most construction materials. There is no compre-hensive treatise on ice adhesion, but there are a number of research reports and papers. This section presents some of those research findings.

Friction Coefficients and Adhesive Strengths

Oksanen (1980, 1983a, 1983b) has conducted laboratory studies on the coefficients of friction between ice and construction materials, plastics and coatings, and on the adhesive strength of ice. Quoting Sackinger and Sackinger (1978), Oksanen notes that when the movement of ice is intermittent, the interaction between ice and a structure is determined by the coefficient of static friction (when ice is stationary for a short time), or by the adhesion strength (when ice is stationary for a long time).

Adhesion is also the dominant mechanism in the transfer of forces when, due to changes in water level, a floating ice cover acts upon a structure to which it is frozen.

The main difference between freshwater ice and sea ice is that sea ice consists of two phases: solid ice and liquid brine. Freshwater ice is mainly solid. For adhesion strength, the existence of the liquid phase is very important. If the brine can form a continuous liquid layer at the contact surface, the adhesion strength will decrease drastically. Even in cases where this does not happen, the adhesion strength will be reduced because of the liquid brine, part of which is accumulated at the ice-solid interface, thus reducing the effective contact area and concentrating stresses.

Oksanen's laboratory observations indicate that fractures can be either adhesive (along the ice-material interface) or cohesive (in the ice) failures. Mean values of adhesive strengths for freshwater ice for polyethylene (PE) and polyvinylchloride (PVC) materials and wood were about 40 psi in the temperature range of -4°F to 14°F, with PVC being somewhat less at the colder temperature. Oksanen measured a mean adhesive strength of 120 psi on 14°F steel. Mean values for saline ice for PE, PVC and steel in the same temperature range were considerably less, about 15 psi. Rather small salinities cause a breakdown in strength.

In measuring static friction of wet 32°F ice and materials, Oksanen (1980) found that friction tended to be greater on wet steel, concrete and wood (0.07 to 1.00) and less on wet plastics and coating systems (0.01 to 0.57). He found no systematic correlation between static friction and normal load, and weak dependencies on temperature. But the adhesive strengths of materials like PE, PVC, polytetraflourethylene (PTFE), nylon and commercial coatings (Inerta 160, Intertuf Epoxy, Intertuf HB Vinyl and Interchlor HB Primer) were strongly dependent on temperature between 32°F and -4°F for freshwater ice. The adhesive strengths increased with decreasing temperatures from 5-10 psi to 30-35 psi for the plastic materials, and from 5-20 psi to 60-80 psi for the coatings. Wood, steel and concrete samples showed no or only slight temperature dependency.

Sackinger and Sackinger (1977) report sea ice coefficients of static friction in the range of 0.3 to 0.7 and kinetic friction of 0.025 to 0.25 for steel. The shear strength of the adfreeze bond of sea ice to steel structures decreases as ice salinity increases, and the adfreeze bond increases as the temperature decreases. The shear strengths of adfreeze were measured in the laboratory with a torqued cylindrical shaft in ice. Average values for low-salinity cold ice was 100 psi, for warm ice about 50 psi and for moderate-salinity cold ice about 50 psi. The maximum shear stress measured was a whopping 230 psi on 0.4 percent salinity ice at -9°F.

Frederking (1979) performed laboratory tests on 2- to 6-inch model wood pilings and measured adhesive strengths of about 100 psi at a loading rate of 1.5 inches an hour and somewhat less strengths at slower rates.

Muschell and Lawrence (1980) report on ongoing field studies on ice uplift on pilings. They found that--for air temperatures ranging from 2°F to 70°F, ice temperatures of 18°F to 32°F and ice unit weights of 41 to 62 pounds per cubic foot--temperatures and unit weights had no direct effect on ice-pile adhesion or direct-shear strength in uplift measurements on 8- and 10-inch steel H- and pipe-piles. Several uplift forces had adhesive strengths of 50 to 75 psi and reached nearly 50 kips. More recent work by Muschell (unreported) gives adhesive strengths of about 100 psi for pipe-piles.

Michel (1978) suggests that the adhesive strength for ice on construction materials is comparable with the shear strength of the ice itself, usually in the range of 60 to 150 psi.

Reducing Ice Adhesion

Jellinek (1970) presented a survey on ice adhesion and abhesion; "ad-" in the sense of "to hold a spread-out substance on the surface of a solid substance in a thin layer of molecules," and "ab-" in the sense of "to take in or suck up, soak or blot, not reflect." (Jellinek: "It seems to me that one solution to ice problems on structures will be to develop materials and coatings to prevent ice from bonding, to promote slippage or to freeze tightly in areas where we wish to force the ice to crack and fail, and thereby relieve forces elsewhere.")

Jellinek notes it is quite feasible to choose satisfactory substrates of sufficiently hydrophobic (i.e., not absorbing water) nature to diminish ice adhesion to an acceptable extent, but the main problem is that such substrates become contaminated after a few adhesions and become useless. The same is true for special interfacial films (e.g., monolayers)--these not only deteriorate but are removed on repeated adhesion. So the problem is not so much to find a suitable hydrophobic surface (which can be achieved fairly easily) but to find a surface that renews itself during use and that remains efficient.

Ice adhesion can also be reduced by two other, somewhat similar methods. A decrease in the shear strength of the layer near the ice interface in the substrate can be achieved by a liquid-like or pasty layer, like oil or grease, with a low shear strength. These substances should preferably be hydrophobic and may contain surface active agents to decrease interfacial free energies. A weakening of the mechanical (shear) strength of ice near the interface can be achieved by inorganic and organic substances added in small amounts to the substrate. These compounds are soluble in water and pass preferentially into the crystal grain boundaries, enlarging the latter and weakening the ice structure.

Low-Adhesion Coatings

There has been and is ongoing work on low ice-adhesion coatings (e.g., with icing problems on helicopter blades). Minsk (1983) notes that much effort over many years has gone into the search for an effective, durable, easily

applied and inexpensive material to eliminate the force of adhesion between ice and a substrate. The objective of zero ice-adhesion on an unheated surface--which would either prevent the formation of ice or ensure self-shedding of very thin accretions--has not yet been achieved. Many commercially available coatings do succeed in reducing the force of adhesion below 15 psi and survive a few freeze-release cycles. The most satisfactory surfaces subject to rain erosion (i.e., rain with a velocity of several hundred miles per hour impinging a whirling blade) are silicone-based materials. Materials with adhesions less than 15 psi include silicon rubber, silicon grease, silicone compounds and sheet teflon.

Sayward (1979) also discusses low adhesion in the context of icing-impaired helicopter blades. Adhesion results from secondary (van der Waals) forces, yet it exceeds normal cohesive strengths. It depends on free surface energy, low contact angle, good contact and wetting, cleanliness, and texture. Poor adhesion occurs with low-energy surfaces or contaminants (e.g., hydrocarbons, fluorocarbons, waxes, oils, etc.), particularly when textured or porous.

Compared with inorganics (metals, oxides, etc.), the adhesion of ice is lower on organic polymers due to the inertness of the hydrocarbon or fluorcarbon composition. Fluorocarbons generally have low ice-adhesion. For teflon PTFE, however, repeated freezing cycles, or high droplet-impact velocities, produce stronger ice adhesion than expected.

Other polymers of appropriate structure--more truly meltable and less viscous, and/or nonporous--may have more impermeable surfaces and hence lower ice adhesions. It may be that fluorocarbons are superior to hydrocarbons for low ice-adhesion. Teflon is an exception, due perhaps to porosity. Other things suggest that hydrocarbons (as found in polyethylene, silicone, acrylic and some other polymers) may be equally or more attractive for low adhesion.

Emphasis should be placed on polymers with hydrocarbon $-CH_2-$ or $-CH_3-$ and fluorocarbon $-CF_2-$ or $-CF_3-$ chains or tails screening other structural components to provide a low-energy, inert, hydrophobic, water-repelling, nonwetting, high-contact angle surface.

Besides normal secondary (van der Waals) forces, attraction and adhesion of water and ice to solids is influenced by the hydrogen-bonding capability of the substrate. Components capable of hydrogen-bonding (mainly oxygen) should therefore be avoided or well screened in the substrate structure.

Work on ice adhesion and low friction also has been furthered by current interest in extended winter navigation. Calabrese et al. (1976) report on low-friction icebreaker hull coatings as determined from laboratory tests and full-scale field tests. Two coatings that showed promise were unfilled, non-solvented polyurethane ("Zebron," Xenex Corp.) and nonsolvented, two-part epoxy ("Inerta 160," Teknos Maalit Oy, Helsinki).

Hanamoto (1983) and Frankenstein et al. (1976) report on CRREL's work with Jellinek at Clarkson College on developing a coating with reduced adhesive-strength properties between the ice and coated lock wall surfaces to enable ice removal. After many compounds with the desired properties had been tested, a long-chain block copolymer was selected. The compound--a poly-(dimethylsiloxane)-bisphenol-A-polycarbonate--reduced the force needed to break the ice coating-bond to metal or concrete by 97 percent (Jellinek et al.

1979). The coating does not prevent the formation of ice, but it does make it easier to remove. Also, an active heat source used in conjunction with this passive coating is an efficient way to remove ice build-up. The coating does not withstand abrasion and rubbing, and its life is unknown.

Interfacial Forces

Raraty and Tabor (1958) note that when water is frozen onto a surface that it completely wets, the interfacial forces are larger than the cohesive forces in the ice. Consequently, when the junction is stressed, the break occurs in the ice. For clean metal surfaces, therefore, little is gained in changing from one metal to another, as the break occurs in the ice itself near the surface.

The adhesion of ice to plastic surfaces is different. It appears that the interfacial forces are smaller than the cohesive forces, so the break occurs truly at the interface.

When ice is cooled below 32°F, it contracts. The coefficient of contraction is considerably larger than for metals and considerably smaller than for plastics. Consequently, when ice is frozen onto a metal surface at temperatures well below 32°F, the ice near the interface is subjected to tensile stresses parallel to the interface; when frozen onto a plastic, the interface is subjected to compressive stresses that are believed to be unimportant in adhesive strength.

Raraty and Tabor's main conclusion is that the adhesion to intrinsically water-repellent surfaces is limited by interaction forces between the ice and the substrate, and not by the strength of either the ice or the substrate itself.

This section began by noting that ice adheres tenaciously to most construction materials. Assuming the ice's grip has not been broken with some suitable construction material or coating, or the ice has not been removed with a deicing scheme, the ice will exert vertical and horizontal forces on harbor structures. In the next sections, we will introduce ice plate analyses for uplift forces and then for bearing capacity to support imposed loads.

VERTICAL FORCES FROM ICE SHEETS

One of the most significant and damaging ice forces results from changes in water levels. These changes cause the ice sheet to move up and down, tearing and pulling harbor structures. The reasons for and the magnitudes of these water level changes will be addressed later, but for now remember that an ice cover in a harbor, a river or on a sea coast is not stationary--far from it, in fact.

Neill (1981) stresses that, in considering vertical loads resulting from adhering ice, the first question to be addressed is the nature of the ice formation around the structure. In the case of diurnal (daily) or more frequent water level fluctuations (e.g., tides or power dams), the ice cover is continually broken free of the structure, and the ice loading is not well defined. Depending on the range of the water level fluctuation, an ice "bustle" or "active zone" may form. The vertical loads are thought to be low or limited in this case. The most severe situation is an ice cover on a

stable water body that is growing and adhering to a structure, followed by an abrupt water level fluctuation. This is the situation to be examined here.

The vertical force on an embedded pile or pier may be limited by the adhesive strength between the ice and the structure surface, by the shear strength of the ice, or by bending failure or deflection of the ice sheet at some distance from the structure (Neill 1981). Hodek (1979) observed three kinds of failure mechanisms, depending on the weather, the past failure history of the ice-pile system and the rate and magnitude of water level change. They are (1) a bending failure of the ice sheet, which leaves a collar of ice firmly attached to the pile and a set of radial cracks in the floating ice sheet (see Figure 3.2); (2) a shear failure in the ice along some preexisting failure ring outside the pile diameter, and (3) a shear failure or slippage at the ice-pile interface with no evidence of bending overstress in the ice sheet. My observations have been the same as Hodek's.

The analysis of vertical forces limited by bending failure or deflection of the ice sheet is theoretically quite difficult and leads to quite variable results, depending on the failure criteria assumed. The brief summary that follows is based on methods detailed in Nevel (1972) and Kerr (1975) and summarized in my earlier ice engineering guide (Wortley 1978).

Minimum Uplift (First Crack Analysis) on Pilings

Table 3.1 lists the minimum ice sheet piling uplift load for a single pile in an infinite ice field. It is based on an elastic solution to a biharmonic differential equation describing a homogeneous, thin, elastic plate on an elastic foundation when shear stresses are ignored. The minimum load is that

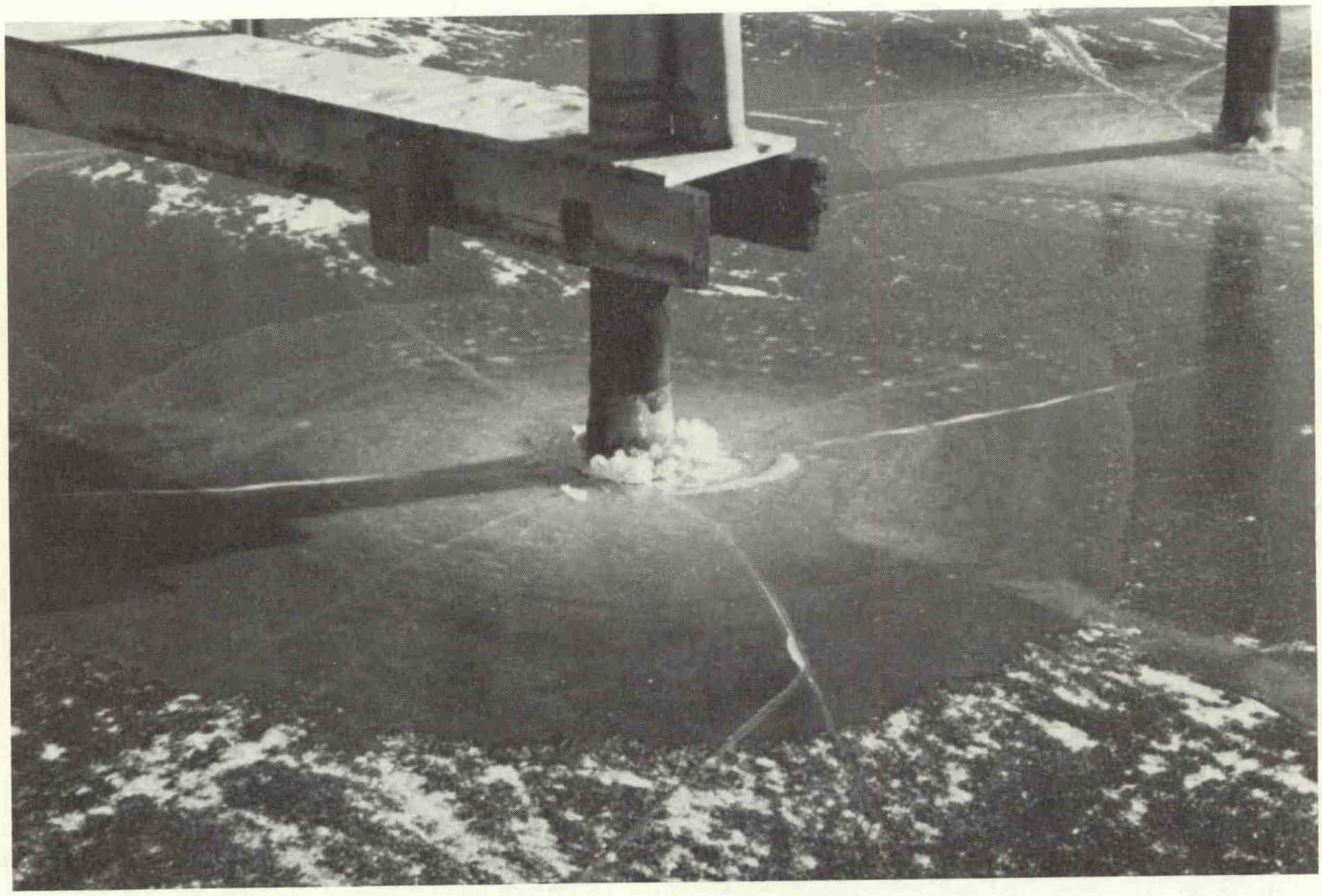


FIGURE 3.2: Thin Ice Cracking on a Steel Piling

TABLE 3.1: Minimum Ice Sheet Piling Uplift Load
(in kips)

Approximate Ice Thickness (inches)	Approximate Radius of Load Distribution (inches)				
	6	12	18	24	60
12	8	10	12	14	20
18	16	20	22	25	35
24	28	33	38	44	60
30	44	52	56	64	85

load on the piling when the sheet first cracks. This cracking load is based on a bottom flexural stress of 200 psi (1378 kPa), a modulus of elasticity of 750 ksi (5168 MPa) and a Poisson's ratio of 1/3 for the ice plate. The uplift load is a direct function of the flexural stress, whereas variations in the values of the modulus of elasticity and Poisson's ratio have a relatively minor effect on the uplift load.

The load is tabulated for several thicknesses of ice and radii of load distribution. The radius of load distribution is the distance from the center of the piling to the circumscribing crack (the so-called "first crack") in the ice. For marina piles, this crack occurs about six inches off the face of the piling. For example, a 12-inch pipe pile has a radius of 6 inches, plus a 6-inch ice collar, for a radius of load distribution of 12 inches.

Table 3.1 is used as follows. For a 12-inch piling with a 6-inch collar frozen in a 24-inch ice sheet, the minimum uplift load would be 33 kips. If the ice were weaker--say, 100 psi instead of 200 psi (assumed for strong, sound lake-ice flexural strength)--the uplift load would be about half as much, or 16.5 kips.

This analysis is based on the first elastic crack and the other assumptions given above. These assumptions are just that--assumptions--and in the field may not be representative. First-crack analysis is a minimum criterion for failure.

Maximum Uplift on Pilings

Nevel (1972) also has formulated a more severe failure criterion with radial cracking and additional circumferential cracking at the ends of the radial cracks, giving a near, if not maximum, upper bound to the solution of the uplift problem. This cracking pattern forms a series of truncated wedges whose tips are supporting the load and whose bases are failure planes when the circumferential crack develops. The uplift loads computed from the truncated wedges criterion are 3 to 5 times the first-crack criterion loads for the range of ice thicknesses and radii of load distributions given in Table 3.1.

Assume we have the elastic solution bounded. We're still faced with an estimate of uplift for a pile in a sheet of infinite extent with the answer varying in a range of 3 to 5 times the minimum force. Now, on top of this we'll add the complexity of a solution by Kerr (1978) for a semi-infinite row of piles wherein the piles on the extremities receive more load than those closer to shore.

The force in the first pile in a long pier is several times larger than the forces in the piles away from the front of the pier. In a typical marina, where pier piles are normally spaced 10 to 15 feet apart, the outer pile would receive 2 to 3 times as much force as the other piles. (It has to "pick up" load from the ice plate area beyond.)

These analyses assume a lot of things, but currently they are the only analytical tools available. They give a feel for the answers to the problem, but obviously a lot depends on personal judgement. The design methods I present later sidestep these analyses somewhat, so the lack of a firmer technical basis won't be a problem.

Before discussing bearing capacity theory in ice sheets, note that two other vertical force problems are typically encountered. They are uplift on a wall or linear structure, and download from a water level drop.

Other Vertical Forces on Pilings and Walls

When the water level beneath an ice sheet drops, the ice becomes a hanging dead weight spanning "supporting pilings." Pilings and structural framing should therefore be designed for this full-thickness dead weight. The maximum unit weight of ice is about 57 pounds per cubic foot (9 kN/m^3). This download situation is aggravated by a tendency for thicker ice to form near structures because of cracking, flooding and freezing. Ice in the more open aisles and fairway areas usually is not as thick as next to the structures.

The weight of hanging ice can also be a problem for floating docks. If a floating dock and its guide piles become joined by ice, then the floating dock will receive bending loads when water levels drop.

The lifting force (per unit length of the circumference of the icebound structure) of rising water levels beneath an ice sheet decreases as the size of the structure increases. The lifting force per unit length will approach values for long straight walls. Löfquist (1951) and Michel (1970) have estimated the vertical forces exerted by ice rigidly attached to walls. The estimates are based on an elastic analysis and rapid rises in water levels.

Table 3.2 lists lifting force per unit length of wall and associated water level rise. The computations are again based on a flexural strength of 200 psi (1378 kPa), a modulus of elasticity of 750 ksi (5168 MPa) and a Poisson's ratio of 1/3. The analyses are linear with flexural strength and relatively insensitive to the values chosen for the modulus and Poisson's ratio.

The table is used as follows. A 24-inch ice sheet that sustains a rapid water level rise of 3.2 inches would lift on the wall about 0.8 kips per lineal foot before the sheet would crack. If the ice were weaker, say only 100 psi, the force would be halved, as would be the required water rise.

TABLE 3.2: Lifting Force on Wall and Rapid Water Level Rise to Cause Force

Approximate Ice Thickness (inches)	Force (kips per foot)	Water Level Rise (inches)
12	0.3	2.3
18	0.6	2.8
24	0.8	3.2
30	1.1	3.6

Usually an ice sheet has cracks in it parallel to a wall or long crib structure, so the uplift is less than that assumed for rigid attachment. Consequently, walls and cribs seem to experience little damage due to ice uplift, though occasionally the top of a crib will be pulled off because it was inadequately secured to the bottom of the crib.

BEARING CAPACITY OF ICE SHEETS

Ice sheets are capable of supporting large loads. Workers and construction equipment can be placed on them safely, and working from the ice in winter can be more economical than working from barges and skiffs in summer.

This subsection introduces ice sheet bearing capacity and discusses some of the construction methods and precautions necessary for working from ice covers. For a fuller understanding of the bearing capacity of floating ice plates, Kerr (1976) offers a critical survey of the literature on this complex problem.

Bearing Capacity Concerns

When a load is placed on an ice sheet, several concerns occur immediately. Will the sheet break? How long can the load remain on the sheet? Will the sheet deflect so much that water seeping up through cracks will flood it? Answers to these questions depend on solutions to viscoelastic-plastic analyses.

What is the source of the bearing capacity of an ice sheet? It is not the strength of the ice per se, but rather water pressure that supports the ice and its load. The hydrostatic pressure on the bottom of a large floating ice sheet equals the weight per unit area of the ice sheet and its snow cover, if any. When a load is placed on the ice, the ice "sinks" until the hydrostatic pressure on the bottom of the sheet has increased enough to balance the load. The load is therefore supported by the water--not by the ice sheet directly. Sheet deflection merely governs the area over which the load is distributed; that is, the characteristics of the ice sheet simply govern the size of the "raft" (see Gerard 1983a).

To distribute its load, an ice sheet must deform. This deformation generates the stresses which eventually cause the failure of the ice sheet and consequent loss of its ability to distribute the load. If a uniform load is placed on the middle of an ice sheet and the load is increased, the ice will first crack perpendicular to the load site in one or more radial cracks. If more load is added, a circular crack--somewhat concentric to the load site and circumferential at the ends of the radial cracks--will form. The cracking pattern in an ice sheet thus usually defines four to eight wedges. At this point, "breakthrough" is about to occur. (You can observe this cracking pattern yourself in the fall just after freeze-over by reaching down from a pier and pushing on the thin ice until it cracks and breaks through.)

Frankenstein (1966) describes the results of a number of distributed and concentrated load-to-breakthrough tests performed on an ice-covered lake. Loads up to 80 kips were supported on 15- to 18-inch ice sheets for 20 to 30 minutes.

Gerard (1983a) notes three bearing-capacity situations of particular concern:

1. Short-term loads--such as a slow-moving vehicle or a crane lifting a load;
2. Moving loads heavy enough and moving fast enough to cause the ice sheet-water system to oscillate at or near the natural frequency, which is not particularly fast; and
3. Long-term loads--such as parked vehicles, stored materials and construction equipment--which is the most difficult situation to analyze.

Nevel (1976) has formulated a mathematical creep model for ice that includes primary, secondary and tertiary creep. His equations show that the deflection at the load increases with time, while the stresses decrease, or relax. This means that the maximum tensile stress--usually the critical stress in an ice plate problem--occurs at the moment the plate is loaded. In other words, the sheet should fail at once or not at all, as the stresses relax with increasing deflection. This, however, is contrary to observations on ice sheets under sustained loads. A possible explanation is that the tensile strength is somehow affected by the creep process. A usual failure criterion is to limit the maximum tensile stress.

Maximum Safe Loads on Ice

Table 3.3 lists the maximum safe load on an ice sheet and the associated deflection under the load. Again, the computations are based on a modulus of elasticity of 750 ksi (5168 MPa) and a Poisson's ratio of 1/3. The answers are insensitive to these chosen values. The analyses are linear with the flexural strength selected. Because we are predicting safe allowable loads, we necessarily should be conservative. Accordingly, the flexural strength selected is 100 psi (689 kPa), not the 200 psi used to determine lifting forces.

These analyses are for short-term loads placed on circular areas of an ice sheet of infinite extent. Also, they represent the loads required to produce the first crack, not breakthrough. The deflections are instantaneous, not long-term creep, and should be checked against ice sheet submergence (i.e., 8 percent of the thickness of the ice).

TABLE 3.3: Maximum Safe Loads on Ice Sheets and Deflection under Load

Thickness of Ice Sheet (inches)	Approximate Radius of Load Distribution (feet)							
	2.5		5.0		7.5		10	
	Load (kips)	Defl. (in.)	Load (kips)	Defl. (in.)	Load (kips)	Defl. (in.)	Load (kips)	Defl. (in.)
12	15	2.5	20	3.3	25	4.2	30	5.0
18	30	2.2	39	2.9	47	3.5	56	4.1
24	50	2.1	64	2.7	76	3.2	87	3.6
30	75	2.0	93	2.5	110	2.9	125	3.3

Table 3.3 is used as follows. An 8-foot by 10-foot, 20-kip load is to be moved on an ice sheet: how thick should the ice be? The load distribution is estimated by converting the area of the rectangle to its circular equivalent and finding the radius. In this case, the loaded area is 80 square feet, and the equivalent circular area has a radius of 5 feet ($\pi r^2 = 80$; $r = 5.0$). Incidentally, the average contact pressure can be checked for reasonableness: it is 250 psf (20,000 / 80), which is alright.

If this 20-kip load is placed on an 18-inch-thick ice sheet whose load capacity is 39 kips and corresponding deflection is 2.9 inches, the deflection can be calculated from the approximate linear load deflection relationship. The 18-inch sheet will deflect (20 kips / 39 kips) x 2.9 inches = 1.5 inches. The allowable deflection of an 18-inch sheet is 0.08 x 18 inches = 1.4 inches. Therefore, an 18-inch sheet of ice with a strength of at least 100 psi is needed to support a 20-kip load on an 8-foot x 10-foot area without cracking or submerging the ice sheet.

If the possibility of a couple of inches of flooding doesn't matter, this same load can be put on a 12-inch sheet--but the ice better be at least 12 inches thick everywhere, and the load better not stay in one place very long. Figure 3.3 shows a small pile driver operating on the ice: area-wise it is about the same size as the above example, but it probably doesn't weigh 20 kips.

HORIZONTAL DYNAMIC FORCES FROM ICE SHEETS

Besides the damaging vertical forces of water level changes, significant horizontal forces from ice sheets must be considered. The principal forces of concern are dynamic forces, and forces of static thermal origin.

Dynamic ice forces are usually considered to be those resulting from moving ice floes driven by streamflow, currents or wind. Forces also are exerted by accumulations of broken ice, such as ice jams, ice piles in front of large structures and forces on booms.

This manual will not deal directly with large dynamic ice loads, because small-craft marinas are placed in sheltered harbors out of the path of such

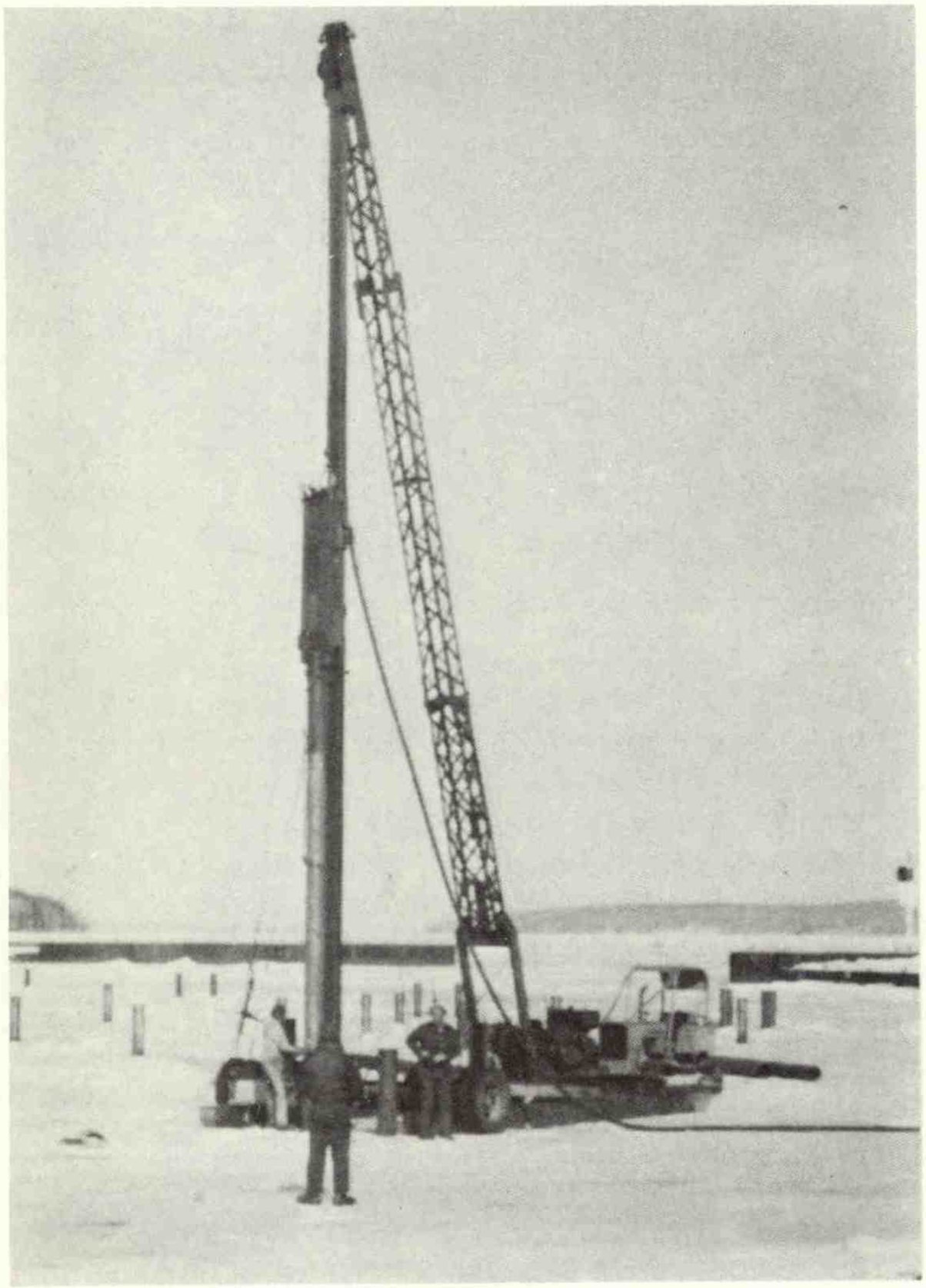


FIGURE 3.3:

Lightweight Pile-Driving Rig
Operating from an Ice Cover

moving loads. Breakwaters are certainly exposed to open expanses of lake ice and thus to large ice floes and shoves, but the design of breakwaters is outside the scope of this manual. I presume the hydraulic force criteria used to design and size breakwaters are adequate to handle the possible ice forces; also, I am unaware of any ice problems with Great Lakes' breakwaters other than the occasional "plucking" of armour stones, as was mentioned earlier.

Those wishing more information on dynamic ice forces should consult Caldwell and Crissman (1983), Carstens (1980), Määttänen (1978) and Neill (1976, 1981), and the proceedings of recent Port and Ocean Engineering under Arctic Conditions (POAC) conferences and International Association for Hydraulic Research (IAHR) Ice Symposia.

Forces and Vibrations of Slender Structures

Määttänen (1978, 1981) has developed and tested a theory on conditions that give rise to self-excited, ice-induced autonomous oscillations in slender

marine pile structures. A theoretical model is formulated by connecting the properties of ice to the dynamic equations of the motion of structures through an averaged ice crushing-strength curve.

This work is in part a result of several Finnish navigational-aid structures that failed from forces of moving ice or were rendered unserviceable because of vibrations. While vibration from ice loads is not usually a concern in small-craft harbors, the theory sheds some light on "dynamic" loads on slender marine pilings.

For self-excited vibrations on slender structures, an energy interchange is needed. A rigid structure cannot store and release energy during the ice load build-up. The ice crushes or buckles from energy stored in the ice in "in-plane" compression. A slender structure will accept the ice load and deflect until the ice suddenly fails and the structure springs back.

The key parameter in this ice/structure interaction is the dependence of the crushing strength on the stress rate, or strain rate. Strength decreases with increasing stress rate, and the ice force becomes about half of the maximum value. The most common integrated response is sawtooth-like (see Figures 3.4a and 3.4b).

At the beginning of the cycle, the displacement rate is the same or slightly lower than the initial velocity. At the last stage of the displacement grow-up its rate decreases, and the point of action in the crushing curve

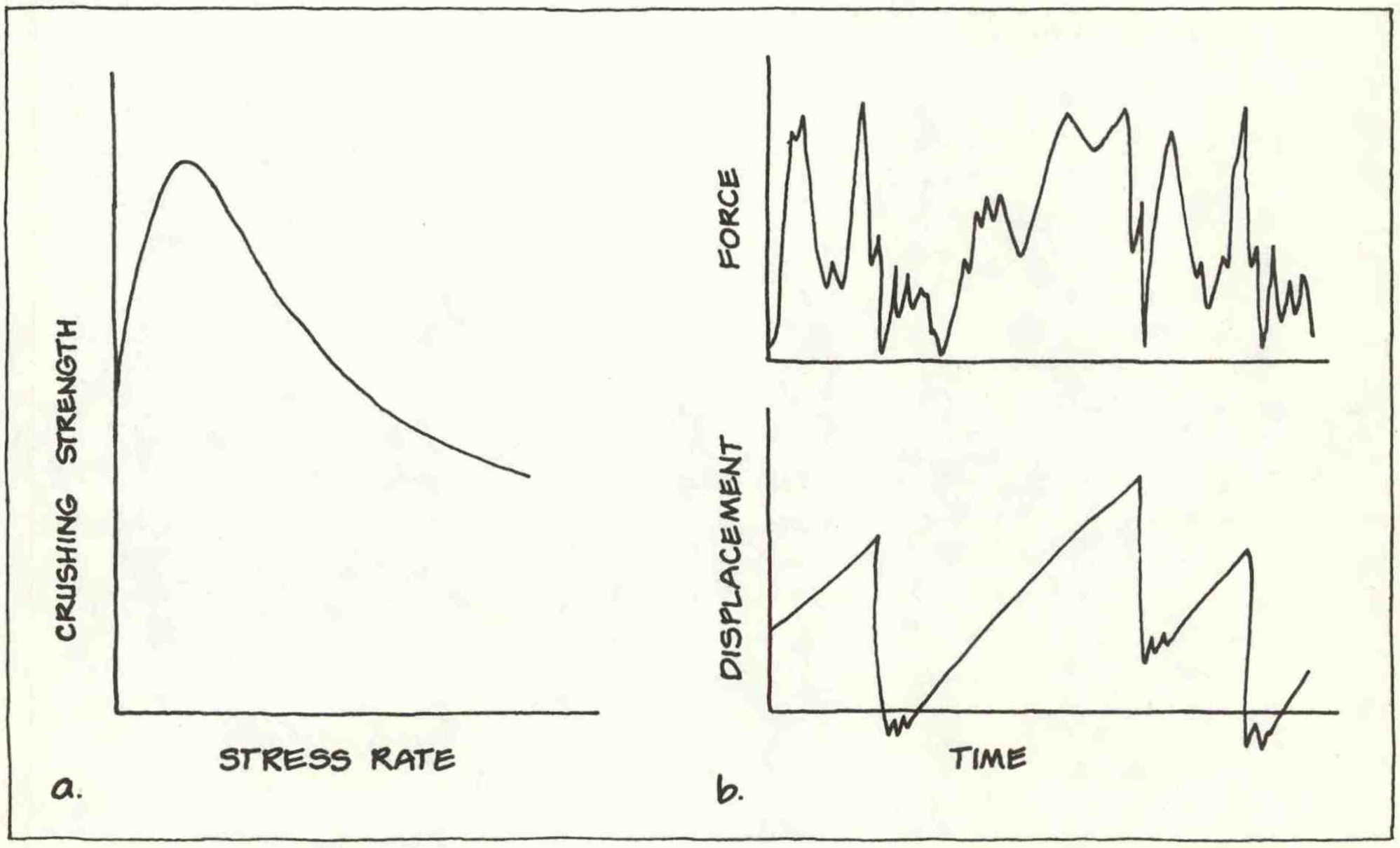


FIGURE 3.4a: Crushing Strength vs. Stress Rate

FIGURE 3.4b: Force and Displacement vs. Time Plots

moves toward the maximum point. Immediately after this point is passed, the crushing and displacement spring-back will start.

When the displacement spring-back has ended, the point of action returns to the region of low stress rate. If displacement spring-back has been great enough, contact betweeen the ice and structure is lost. Before the next cycle starts and steady contact between the ice and structure is established, there may be several small hits and spring-backs.

I have observed this phenomenon while standing on flexible pile-supported marina dock structures as ice was running through the harbor. The docks deflected up to a point and then spring back with such a lurching action that I maintained my balance only with good "sea-legs." Ice broken up by winds and waves is shown hitting marina dockage in Figure 3.5.

Impact Forces

A rough estimate of the maximum forces involved can be made, assuming the structure is rigid and struck by an ice floe or iceberg. The approximate calculation that follows is an extension of the analyses presented in a paper by Cammaert and Tsinker (1981).

They assume that the piece of ice colliding with the rigid pier will dissipate half of its kinetic energy in progressively crushing the ice, and then the



FIGURE 3.5: Broken Ice Cover Moving through Marina Dockage

piece will rotate and pass by. Michel (1978) states that the maximum velocity of moving ice under the influence of a steady wind is not more than 3 percent of the wind velocity (ice runs in rivers can have much larger velocities).

The broken-up ice hitting the dock piles shown in Figure 3.5, which were nominally 12 inches, was about 2 feet thick and 5 feet by 10 feet in dimension. The ice was moving about a half-foot per second when the wind was blowing more than 30 mph (or the ice was moving at about 1 percent of the wind velocity in this case). I assumed an ice strength of about 100 psi. The impact force is estimated by:

$$F = \left[v\ h\ \sigma \left(\frac{3\ d\ W^{1/2}}{2g} \right)^{2/3} \right] \qquad (3.1)$$

Where:

F = impact force (pounds)

v = floe velocity (feet per second)

h = ice block thickness (inches)

σ = ice strength (pounds per square inch)

d = pile diameter (inches)

w = full weight of ice floe piece (pounds)

g = gravity (32.2 feet per second squared)

$$F = \left[(0.5)\ (2)\ (100)\ (144) \left(\frac{3 \times 1 \times 2 \times 5 \times 10 \times 57}{2 \times 32.2} \right)^{1/2} \right]^{2/3}$$

= 3,820 lbs. (or about 4 kips impact force)

This is approximate at best, but it gives an idea of what magnitudes of forces could be involved.

ICE PRESSURES OF THERMAL ORIGIN

Ice pressures of thermal origin are pressures, or forces, likely to be seen in a marina. The analysis presented here is based primarily on laboratory research at Laval University under Prof. B. Michel, and on other literature as cited.

There is a paucity of facts from the field--and plenty of differing opinions in the literature--on thermal ice forces. I have found nothing for the forces on a free-standing pile being shoved over, or the pontoon being squeezed by ice. The reported forces of thermal origin are on more linear structures and don't directly deal with isolated structures and their edge effects in an infinite sheet.

Drouin and Michel (1974) state that the thermally induced pressures of ice on dams and other hydraulic structures are significant and may even be the controlling design load for some structures. In the past, these thermal thrusts have been calculated for the condition of ice failing by crushing stresses, such as 400 psi (2756 kPa). This resulted in thermal loads known to

be too high--for example, 50 kips per foot. Now values of one-fourth to half these are normally used.

During the past 50 years, engineers and scientists--for example, Royen, Brown and Clarke, Rose, Monfore, Löfquist, Lindgren and, more recently, Carstens (1980) and Bergdahl and Wernersson (1978)--have researched and estimated thermally induced ice pressures. The research preceding Drouin and Michel did not take into account the crystallographic characteristics of the types of ice and actual strain rates that exist in nature in the regime of thermal expansion of ice sheets. (These strain rates are quite low, on the order of $10^{-8}s^{-1}$.) Also, the initial temperature of the ice was not considered in some of the previous work, and some research was based on too few tests.

The composite picture of past studies is one of ambiguities and many deficiencies. The next subsection reviews Drouin and Michel's work, which considered these past studies, and it presents some recent comments about Drouin and Michel's methods and results.

Laboratory Studies of Drouin and Michel

Drouin and Michel's studies and solutions do not take into account any thermal boundary layers (air/ice and air/snow) or any solar radiation on the ice. But they believe the pressures they calculate on the basis of heat transfer solely by conduction are of the proper order of magnitude and perhaps greater than the pressures calculated with the air/ice boundary layer and the absorption of solar radiation taken into consideration.

Drouin and Michel found--contrary to many other published theories and results--that the pressure in an ice sheet was highest for small, not large, rates of temperature increase at the surface of the ice. They explain this finding by the deeper penetration into the ice sheet of the temperature variations at the time when the stress at the surface attains its maximum. The top zone of an ice sheet is where the stressed condition--related to the maximum pressure exerted by thick ice sheets--develops. Attenuation of temperature variation in the interior of an ice sheet is very rapid. Muschell and Lawrence (1980) state from their measurements of field ice temperatures that "very rapid" may be too strong a description.

Drouin and Michel used a sinusoidal air temperature variation to determine the stress in an ice sheet as a function of temperature. The sinusoidal variation is where the ice temperature rises at an increasing rate until it has achieved half its total rise. This occurs when half the time needed to achieve the total temperature rise has passed. The temperature continues to rise but at a decreasing rate until the maximum temperature is reached.

Other factors that affect the temperature variation in an ice sheet include the presence of snow on the ice sheet, the thickness of the ice sheet, increase of thickness in the ice sheet as a function of time, solar radiation absorbed, and variable thermal properties of ice. These thermal properties vary primarily with temperature and are listed in Table 3.4.

Bergdahl and Wernersson (1978) and Carstens (1980) question Drouin and Michel's (1974) total neglect of the thermal boundary layers. Bergdahl and Wernersson claim that the method used by Drouin and Michel gives incorrect

TABLE 3.4: Thermal Properties of Ice (after Drouin and Michel 1974)

Temperature (°F)	Thermal Expansion (1/°F)	Specific Heat (Btu/lb./°F)	Thermal Conductivity (Btu in./hr./ft.^{2}/°F)
32	0.000030	0.506	15.6
14	0.000029	0.487	16.1
- 4	0.000028	0.468	16.8
-22	0.000027	0.450	17.6
-40	0.000026	0.431	18.6

results and warn that the choice of boundary conditions is more important than the choice of ice parameters.

They also note that the magnitude of the ice pressure in the ice cover will be due to the rate of change of temperature in the ice, the coefficient of thermal expansion, the rheology of ice, the extent to which cracks have been filled with water, the thickness of the ice cover, and the degree of restriction from the shores.

Studies aiming at thermal ice pressures tend to oversimplify the energy balance of the surface by simply setting the surface temperature equal to the air temperature, or only calculating advective heat transfer. Short-wave and long-wave radiations increase the rate of change in the temperature in the mornings, especially during clear weather. The long-wave back radiation can cause a considerable depression of the ice surface temperature, which is very pronounced during clear, calm weather and at night. Omitting radiation, therefore, results in an underestimation of the daily temperature variations in an ice cover.

I have observed an intriguing and haunting radiation effect on windswept lake ice a mile or so offshore: on a cold, calm, sunny late afternoon, the ice was emitting heat! The visual effect is the same as seen over hot pavement while driving through the desert--except I was driving across an ice road to an island.

Table 3.5 presents the results of some of Drouin and Michel's (1974) work. The pressures are for ice samples restrained in one direction. The maximum thrusts of S1 columnar ice, loaded perpendicular to the c-axis, and T1 snowpack ice are reported. They were computed from the laboratory test data and a sinusoidal varied ambient temperature rise from an initial steady temperature state in the ice (i.e., the initial ice temperature profile varied linearly from the ambient temperature at the surface to the melting point at the ice/water interface). The thickness of the ice was assumed to remain constant during the period of ambient temperature increase. With time, the ice warms up and exerts thermal stresses. These stresses reach maximums--and

the stress in the ice at various depths can be summed up to obtain the maximum ice thrust per unit length for a given ice thickness, total temperature rise and duration of temperature rise.

Thrusts for ice thinner than that in Table 3.5 will be less than the tabulated values. Also, the analysis of stresses indicates that the stresses in an ice sheet are particularly high in the first 8 to 12 inches of thickness of a thermally stressed ice sheet. The bottom region of a thick ice sheet may consist of a different type of ice without appreciably altering the stressed conditions, the heat transfer being the same.

Table 3.5 is used as follows. Suppose we have a 20-inch ice sheet with a surface temperature of -4°F. Assume the surface warms up to 32°F and that this occurs during a period of 10 hours. Also assume that the ice--or at least the upper portion of it--is snow ice (S1) rather than mostly columnar ice (T1). The thermal thrust would be about 14 kips per foot.

How do we know what period of temperature rise to assume? Local climatological data would be required, and this would also get into probabilities of recurrence for different rates and durations of temperature rise. In the example above, the rate of temperature increase would be 3.6°F per hour (-4°

TABLE 3.5: Nominal Pressure of Thermal Origin by an Ice Sheet Restrained in One Direction (kips per foot) (after Drouin and Michel 1974)

Approximate Thickness of Ice (in.)	Ice Surface Temperature					
	14°F		-4°F		-22°F	
	S1 Ice	T1 Ice	S1 Ice	T1 Ice	S1 Ice	T1 Ice
	Duration of Temperature Increase: 5 Hours					
20	5	4	11	8	19	13
30	5	4	11	8	20	14
40	5	4	11	8	20	15
	Duration of Temperature Increase: 10 Hours					
20	6	5	14	11	23	16
30	7	5	15	11	24	18
40	7	5	15	12	25	19
	Duration of Temperature Increase: 20 Hours					
20	9	7	18	13	27	18
30	9	8	20	15	30	20
40	9	8	20	16	32	22

to 32° divided by 10 hours). That is rapid: climatological records for Québec City, for example, indicate that this rate of temperature rise for a 10-hour period occurs about once in 50 years.

Figure 3.6 shows a small-craft harbor main head pier bowed (permanently) from thermal forces. Figure 3.7 shows a thermal aisle crack in an ice sheet. The sheet was locally buckled, or pushed up, before damaging the piers.

Other Factors Affecting Ice Thrusts

Except for complete biaxial restraint, most factors in nature tend to decrease calculated thrusts. How important is biaxial restraint in estimating thermal ice forces? Drouin and Michel (1974) recommended using thrusts in reservoirs based on uniaxial tests of ice. Michel (1978) indicates that biaxial stresses are some 1.85 times the uniaxial stress for ice. Does this mean that forces might be nearly doubled from those listed in Table 3.5? For a marina, probably not (unless it was completely surrounded by vertical sheet piling walls); for a swimming pool, probably so. One of the problems of the buckled pier shown in Figure 3.6 is that the harbor basin is surrounded by vertical



FIGURE 3.6: Small-Craft Harbor Head Pier Bowed by Thermal Ice Pressure



FIGURE 3.7: Thermally Active Ice Sheet Aisle Crack Buckled Between Head Piers

sheet piling for 75 to 80 percent of its perimeter. Although not verified, biaxial stress states may exist at this site.

What about the factors that mitigate ice pressures? The two most important are snow cover and ice cracks. An inch or two of snow affords very good insulation, and only a small portion of any surface temperature fluctuation is transmitted to the ice. Furthermore, a snow depth less than the ice cover thickness is usually sufficient to submerge the ice cover and allow it to be flooded through cracks (Gerard 1983a).

Metge (1976) has reported a five-year study and field observation of thermal cracks at Kingston, Ontario. The frequency of cracking tends to decrease as an ice cover becomes thicker. Metge categorizes thermal cracks into three groups. The first group is dry cracks, which absorb a significant amount of thermal ice movement. They are common and consist of cracks that extend from the top of the sheet down one-half to two-thirds of the ice thickness. These dry cracks open and close according to ice temperature.

The second and third groups are wet cracks, which are relatively rare, and wide wet cracks, which are generally too wide to refreeze into solid ice. Wet cracks are narrow enough to refreeze rapidly and, in so doing, add materially to the ice sheet size (nonmitigating for thermal pressures). Wide wet cracks are wide enough and common enough to put a leg through, as I can attest from personal experience.

Dry thermal cracks usually are less than three-quarters of an inch wide and act as bellows taking up thermal activity. They are mitigating and perhaps reduce thermal pressures by 50 percent or more.

Summary Notes

In closing this section on horizontal ice forces, note that there are very few thermal ice pressure measurements for any given situation and, as stated earlier, they are nonexistent for isolated harbor structures like freestanding mooring piles.

Watch out for situations where there is open water on one side of a structure and ice on the other, or thin ice on one side and thick ice on the other. In such situations, the full lateral thrust must be expected.

Suspect that thrusts exerted on obstructions of small width are likely to be larger (per unit width) than on wider objects, such as dam faces.

Finally, pay particular attention to ice loads in design because--unlike most other loads on such structures--they are lateral loads.

Many of the ice engineering-related concerns not discussed in this chapter (e.g., what thermal thrust pressure should be used for design?) will be discussed later. The next two chapters will review soil mechanics and foundation engineering and will introduce foundation design. Though there is a big difference between frozen water and dirt, there are also many similarities, both inherently and with respect to the methodologies and research progress--Meyerhof's work on the bearing capacity of soils and of ice, for example.

4. Review of Geotechnical Engineering: Soil Mechanics and Foundations

Karl Terzaghi, generally regarded as the father of soil mechanics, opened the first international conference on soil mechanics and foundation engineering at Harvard University in 1936 by saying, "Since these men [the outstanding executives and experienced construction engineers attending the conference] owe their success and their professional standing to a keen discrimination between reality and fiction, I am sure they will appreciate our feelings against half-baked textbook wisdom and assist us in getting down to tangible facts."

In presenting the soil mechanics and foundation engineering part of this manual, I am going to have to start with some textbook wisdom that I hope is not too "half-baked." This will be by way of a review or introduction to the field now usually called "geotechnical engineering"--the mechanics of soil and the engineering of foundations. The review and introductory materials are taken largely from Holtz and Kovacs (1981) and to a lesser extent from Sowers (1979).

INDEX AND CLASSIFICATION PROPERTIES OF SOILS

This section will introduce the basic terms and definitions that geotechnical engineers use to index and classify soils. Classification of soils is important because it is the "language" engineers use to communicate certain general knowledge about the engineering behavior of the soils at a particular site. It also discusses the engineering properties of soils necessary to the design of foundations and earth structures.

Volumetric and Weight Relationships

In general, any mass of soil consists of a collection of solid particles with voids in between. The soil solids are grains of different minerals and organic matter, whereas the voids may be filled with water or air, or partly filled with both. If all the void space is filled with water--as will be the case for the soils under water in harbors--the soil is said to be "fully saturated," or just "saturated."

Another important volumetric relationship is the void ratio (e). The void ratio describes the fraction of void space relative to the space occupied by solid soil grains (e = the volume of voids divided by the volume of solids). The void ratio is used to describe the process of consolidation and settlement, which occurs as the volume of voids decreases because of some type of loading. The soil grains themselves don't compress much: they only get closer to one another as settlement occurs. The void ratio is also used to describe how dense a soil deposit is.

The other basic relationships for a mass of soil are weight relationships. First, soil engineering has three (or four) useful unit weights: (1) dry unit weight (no water present); (2) wet unit weight, or in-situ unit weight or

natural unit weight (some water in the voids, called pores); and (3) saturated unit weight (water and no air present).

If the soil is submerged, it is assumed to be saturated. Its unit weight, therefore, is its saturated weight ("sopping wet" weight) minus the unit weight of water--in other words, the soil's buoyant weight is the saturated weight minus 62.4 pounds per cubic foot. If saturated sand weighed 120 pounds per cubic foot, for example, its bouyant unit weight would be 57.6 pounds per cubic foot. Generally, bouyant unit weights will be used in harbor design calculations for stresses, etc.

The last basic weight relationship is water content (w). Water content is the weight ratio of the water to the solids, expressed as a percentage (weight of water divided by weight of solids). Peat soils may have more weight of water than weight of fibrous or amorphous peaty substances, so these soils have water contents--also called natural moistures--of 300 or 400 percent. For most soils, however, the water content is less than 100 percent.

Soil Texture

The "solids" part of the soil mass consists primarily of particles of mineral and organic matter in various sizes and amounts. The "texture" of a soil is its appearance or "feel," and it depends on the relative sizes and shapes of the particles as well as the range or distribution of those sizes. Coarse-grained soils (e.g., "sands" and "gravels") are obviously coarse-textured, while a fine-textured soil might be predominantly composed of very thin mineral grains invisible to the naked eye (e.g., "silt" and "clay" soils).

Such textural classification terms--gravels, sands, silts and clays--are useful in a general sense in geotechnical engineering practice. For fine-grained soils, the presence of water greatly affects their engineering response--much more so than grain size or texture alone. Water affects the interaction between the mineral grains, and this may affect their plasticity and their cohesiveness.

Texturally, soils may be divided into coarse-grained and fine-grained soils. A convenient dividing line is the smallest grain visible to the naked eye. Soils with particles larger than this size (about 0.002 of an inch) are called coarse-grained, while soils finer than that are termed fine-grained. Another convenient way to classify soils is according to their plasticity and cohesion (the sticking together of like materials). For example, sands are nonplastic and noncohesive (cohesionless), whereas clays are both plastic and cohesive. Silts fall somewhere between clays and sands; they are fine-grained yet non-plastic and cohesionless. (If you ever made a sand castle, you know you need damp sand, not dry sand. Why?--mainly because damp sand has a certain amount of cohesion, really "apparent cohesion," due to capillary water tensile stresses. For this reason, some people prefer to call sandy soils "granular" rather than "cohesionless," and call predominantly clay and silt soils "cohesive" or "fine-grained.")

A grain size curve for a given soil sample can be constructed by laboratory tests that identify the percentages, by weight, of the various particle sizes present in the sample. If most of the particles are of the same size, the soil is said to have a uniform gradation (like a bag of pea gravel or beach

dune sand); if most sizes are present, the soil is said to be well graded (like a good quality road base mix).

A crude shape parameter is the coefficient of uniformity, C_u. It is computed as the ratio of 60 percent passing-size to 10 percent passing-size ($C_u = D_{60}/D_{10}$, where D is grain diameter). A uniform soil has a C_u of 2 or 3; a well-graded soil, 15 or greater.

The shape of the individual particles is at least as important as the grain size distribution in affecting the engineering response of granular soils. A qualitative shape determination is made as a part of visual classification. Coarse-grained soils are commonly classified as rounded, subrounded, subangular or angular. Grain shape is very significant in determining the frictional characteristics (shear strength) of granular soils.

Atterberg Limits

Water contents classify fine-grained soils by their Atterberg limits. The methods used relate the natural moisture content to the Atterberg limits water contents. The limits are determined in the laboratory by test methods developed in the early 1900s by Swedish soil scientist A. Atterberg.

Atterberg realized that at least two water contents were required to define the plasticity of clays--corresponding to the upper and lower limits of plasticity. These consistency limits are the liquid limit (LL) and the plastic limit (PL), and the difference between them is the range of moisture contents over which the soil is plastic. This range is called the plasticity index (PI), which equals LL minus PL. (A soil wetter than LL could be described as a slurry, pea soup to soft butter, or a viscous liquid. A soil drier than the PL is a semi-solid, like cheese. A plastic soil will deform but not crack, like soft butter to stiff putty.) The limits set by Atterberg are relatively arbitrary; but they are standardized in engineering practice, and many soil properties are indexed to them.

The way is related the natural moisture content of a given soil, or scale it to the plasticity, is through the liquidity index (LI), which equals the natural moisture content minus the plastic limit, divided by the plasticity index.

This section is summarized by the following list of relationships:

Degree of saturation:
$$S = \frac{\text{volume of water}}{\text{volume of voids}} \times 100\% \qquad (4.1)$$

Void ratio:
$$e = \frac{\text{volume of voids}}{\text{volume of solids}} \qquad (4.2)$$

Water content:
$$w = \frac{\text{weight of water}}{\text{weight of solids}} \times 100\% \qquad (4.3)$$

Coefficient of uniformity:
$$C_u = \frac{D_{60}}{D_{10}} \qquad (4.4)$$

Plasticity index: $$PI = LL - PL \quad (4.5)$$

Liquidity index: $$LI = \frac{w - PL}{PI} \quad (4.6)$$

SOIL CLASSIFICATION

The preceding discussion gave a general idea about how soils are classified. Usually, however, general classifications like sand or clay include such a wide range of engineering characteristics that additional subdivisions or modifiers are required to make the terms more useful in engineering practice. These terms are collected into "soil classification systems," usually with some specific engineering purpose in mind.

A soil classification system is, in effect, a language of communication between engineers. It provides a systematic method of categorizing soils according to their probable engineering behavior, and it allows engineers access to the accumulated experience of other engineers. A soil classification system does not eliminate the need for detailed soils investigations or testing for engineering properties. But the engineering properties have been found to correlate quite well with the index and classification properties of a given deposit. By knowing the soil classification, then, the engineer already has a fairly good general idea of the way the soil will behave in the engineering situation, during construction, under structural loads, etc.

One classification system widely used by the engineering agencies of the U.S. government, geotechnical engineering consulting firms and soil testing laboratories is the Unified Soil Classification System. There are three major divisions: coarse-grained, fine-grained (including organic soils) and peat.

Coarse-grained soils are subdivided into gravels and gravelly soils (G) and sands and sandy soils (S). These groups are divided into four secondary groups according to the grain size distribution and nature of the fines in the soils. The uniformity coefficient and a coefficient of curvature set specific limits between secondary groups. Well-graded (W) soils have a good representation of all particle sizes, whereas the poorly graded (P) soils are either uniform, or skip- or gap-graded (some sizes missing).

Fine-grained soils are subdivided into silts (M, from the Swedish terms "mo," meaning very fine sand, and "mjäla," meaning silt), and clays (C), depending on their liquid limit and plasticity index. Organic soils (O) and peat (Pt) are also included in this fraction. The silt, clay and organic fractions are further subdivided on the basis of relative low (L) or high (H) liquid limits. The dividing line has been arbitrarily set at 50. Peat is a separate category of soil containing at least 25 percent organics by weight.

Table 4.1 is an abbreviated version of the Unified Soil Classification System (the full, unabbreviated version should be consulted when classifying soils).

Though the letter group symbols in the classification system are convenient, they do not completely describe a soil or soil deposit. For this reason, descriptive terms are also used along with the letter symbols for a complete

TABLE 4.1 Unified Soil Classification System (abbreviated)

Group Symbol	Coarse-Grained Soils
GW	Well-graded gravels, gravel sand mixtures, little or no fines.
GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.
GM	Silty gravels, gravel-sand-silt mixtures.
GC	Clayey gravels, gravel-sand-clay mixtures.
SW	Well-graded sands, gravelly sands, little or no fines.
SP	Poorly graded sands, gravelly sands, little or no fines.
SM	Silty sands, sand-silt mixtures.
SC	Clayey sands, sand-clay mixtures.
	Fine-Grained Soils
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
OL	Organic silts and organic silty clays of low plasticity.
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
CH	Inorganic clays of high plasticity, fat clays.
OH	Organic clays of medium to high plasticity, organic silts.
Pt	Peat and other highly organic soils.

soil classification. Such characteristics as color, odor and homogeneity of a deposit are observed and included in a description.

For coarse-grained soils, such items as grain shape, mineralogical content, degree of weathering, in-situ density and degree of compaction, and presence or absence of fines are noted. Adjectives like rounded, angular and sub-angular are commonly used to describe grain shape. Terms like very loose, loose, medium, dense and very dense are used to describe in-situ density.

For fine-grained soils, such items as natural water content, consistency and remolded (disturbed in some way) consistency are noted. Consistency in the natural state corresponds in some respects to the degree of compaction in coarse-grained soils. Terms like very soft, soft, medium, stiff (sometimes the word "firm" is used synonymously), very stiff and hard are used to describe consistency.

SOIL STRUCTURE AND FABRIC

The term "clay" can refer to specific minerals, such as montmorillonite or kaolinite. But in civil engineering, clay often means a "clay soil"--a soil that contains some clay minerals as well as other mineral constituents, has plasticity, and is "cohesive." Clay soils are fine grained, but not all fine-grained soils are cohesive or clays. Silts are both granular and fine grained. The individual silt grains, like clays, are invisible to the naked eye, but silts are noncohesive and nonplastic.

Certain characteristics of granular soils--such as grain size distribution and the grain shape--affect the engineering behavior of these soils. On the other hand, the presence of water, with a few important exceptions, is relatively unimportant in their behavior.

In contrast, the grain size distribution for clay soils has relatively little influence on the engineering behavior, but water markedly affects their behavior. Silts are an "in between" material. Water affects their behavior--they are "dilatant" (can expand in bulk with change of shape due to the increase of space between rigid particles as they change positions)--yet they have little or no plasticity, and their strengths, like sands, are essentially independent of water content.

Clay minerals are very small particles that are very active electrochemically. The presence of even a small amount of clay minerals in a soil mass can markedly affect the engineering properties of that mass. As the amount of clay increases, the behavior of the soil is increasingly governed by the properties of the clay. When the clay content is about 50 percent, the sand and silt grains are essentially floating in a clay matrix and have little effect on the engineering behavior.

In geotechnical engineering practice, the "structure" of a soil is taken to mean both the geometric arrangement of the particles or mineral grains as well as the interparticle forces that may act between them. Soil "fabric" refers only to the geometric arrangement of the particles.

In granular or cohesionless soils, the interparticle forces are very small, so both the fabric and structure of gravels, sands and, to some extent, silts are the same.

In fine-grained cohesive soils, interparticle forces are relatively large and thus both these forces and the fabric of such soils must be considered as the structure of the soil.

The structure strongly affects, some would say "governs," the engineering behavior of a particular soil. Geotechnical engineers must consider the soil structure and fabric, at least qualitatively, when cohesive soils are encountered in engineering practice.

The structure of most naturally occurring clay deposits is highly complex. The engineering behavior of these deposits is strongly influenced by both the macro- and the microstructure. At present, no quantitative connection exists between microstructure and the engineering properties, but it is important for the engineer to have an appreciation of the complexity of the structure of cohesive soils and their relation to engineering behavior.

Grains of soil, which can settle out of a soil-fluid suspension independently of other grains, will form what is called a "single grained" structure. This is the structure of a sand or gravel pile, for example. Deposition media include both air (loess deposits, sand dunes) and water (rivers, beaches). Single-grained structures may be "loose" (high void ratio, low density) or "dense" (low void ratio, high density). A wide range of void ratios is possible, depending on the grain size distribution and the packing arrangement of the grains.

The greatest possible void ratio, or loosest possible condition of a soil, is called the maximum void ratio. It is determined in the laboratory by very carefully pouring dry sand from a funnel, with no vibration or free-fall, into a calibrated mold of known volume. The maximum void ratio is then calculated from the weight of the sand in the mold. Similarly, the minimum void ratio is the densest possible condition that a given soil can obtain, which is calculated by vibrating a known weight of dry sand into a known volume.

Relative Density

Relative density (D_r) is useful in comparing the void ratio of a given soil with the maximum and minimum void ratios. Relative Density is usually expressed as a percentage, defined as:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\ \% \qquad (4.7)$$

The relative density of a natural deposit very strongly affects its engineering behavior. It is important to conduct laboratory tests on samples of sand at the same relative density as in the field. Sampling of loose granular materials--especially at depths greater than a few yards--is very difficult, if not nearly impossible. Since the materials are very sensitive to even the slightest vibration, you can never be sure the sample has the same density as the natural soil deposit. Consequently, different kinds of penetrometers are used in engineering practice, and the penetration resistance values are roughly correlated with relative density (I will use such correlations later).

WATER IN SOILS

As the preceding discussion of the Atterberg limits, classification of soils and soil structure indicates, water in the soil mass is very important. Water very strongly affects the engineering behavior of most soils, especially fine-grained soils, and water is an important factor in most geotechnical engineering problems. In general, water in soils can be thought of as either static or dynamic.

The groundwater table, even though it actually fluctuates throughout the year, is considered static for most engineering purposes. Similarly, capillary water is usually taken to be static, though it too can fluctuate, depending on climatic conditions and other factors. Other static water and phenomena in soils include absorbed water on clay minerals, shrinkage, swelling and frost action.

Water flowing through the voids and pores in a soil mass is dynamic. The ease with which water moves through a soil mass is characterized by its permeability (also referred to as "hydraulic conductivity") as given by Darcy's law. Darcy's law states that the quantity of water flowing in a unit time is equal to the permeability times the hydraulic gradient times the cross sectional area through which it flows. The hydraulic gradient is the driving head divided by the length over which the flow takes place.

Effective Stress and Neutral Stress

The concept of intergranular or effective stress is very important to an understanding of soil behavior:

$$\sigma = \sigma' + u \tag{4.8}$$

where: σ = total normal stress

σ' = intergranular or effective normal stress

u = pore water or neutral pressure

Both the total stress and pore water pressure may readily be estimated or calculated with knowledge of the unit weights and thicknesses of the soil layers and the location of the groundwater table. The effective stress cannot be measured--it can only be calculated!

The total vertical stress (or normal stress) is called the "body stress" because it is generated by the mass (acted upon by gravity) in the body. To calculate the total vertical stress σ_v at a point in a soil mass, simply sum up the unit weight of all the material (soil solids + water) above that point:

$$\sigma_v = \gamma h \tag{4.9}$$

where: σ_v = total vertical stress

γ = the soil unit weight

h = the height of soil above a point

The neutral stress or pore water pressure is similarly calculated for static water conditions. It is simply the depth below the ground water table to the point in question times the unit weight of water, or:

$$u = \gamma_w z_w \tag{4.10}$$

where: u = pore pressure or neutral stress

γ_w = the unit weight of water (62.4 lbs. per cubic ft.)

z_w = height of water above a point

The stress in the water is called the neutral stress because it has no shear component. Recall from fluid mechanics that by definition a liquid cannot

support static shear stress. A liquid only has normal stresses that act equally in all directions. On the other hand, soil stresses can have both normal and shear components. The effective stress is simply the difference between the total and neutral stresses (Equation 4.8).

Physical Meanings of Effective Stresses

What is the physical meaning of effective stress? In a granular material like a sand or gravel, it is sometimes called the "intergranular stress." However, it is not really the same as the grain-to-grain contact stress, since the contact area between granular particles can be very small and even approach a "point" with rounded grains. Rather, the intergranular stress is the sum of the contact forces divided by the total, or gross, area. The total vertical force, or load, can be considered to be the sum of the intergranular contact forces plus the hydrostatic force in the pore water.

In granular materials, the contact areas approach point areas. Equation 4.8, first proposed by Terzaghi in the 1920s, defines effective stress. This is an extremely useful and important equation. An accepted postulate in geotechnical engineering is the belief that the effective stresses in a soil mass actually control or govern the engineering behavior of that mass.

The response of a soil mass to changes in applied stresses (compressibility and shearing resistance) can be explained consistently on the basis of changes in the effective stresses in that soil mass. The principle of effective stress is probably the single most important concept in geotechnical engineering.

What does the concept mean for fine-grained materials? It is doubtful that the mineral crystals are in actual physical contact, since they are surrounded by a tightly bound water film. On the micro scale, the interparticle force fields that would contribute to effective stress are extremely difficult to interpret and philosophically impossible to measure. Any inference about these force fields comes from a study of the fabric of the soil.

In view of this complexity, what place does such a simple equation as 4.8 have in engineering practice? Experimental evidence and careful analysis has shown that, for saturated sands and clays, the principle of effective stress is an excellent approximation to reality. It is not so good for partially saturated soils or saturated rocks, however.

Whatever it is physically, effective stress is defined as the difference between an engineering total stress and a measurable neutral stress (pore water pressure). The concept of effective stress is extremely useful for understanding soil behavior, interpreting laboratory test results and making engineering design calculations. The concept works, and that's why we use it.

Lateral Earth Pressure at Rest

You will recall from hydrostatics that the pressure in a liquid is the same in any direction--up, down, sideways or any inclination--it doesn't matter. But this is not true in soils. Stresses in situ are not hydrostatic. The ratio of horizontal to vertical stresses can be expressed through the effective stress concept and a coefficient as follows:

$$\sigma_h' = K_o \sigma_v' \tag{4.11}$$

where: σ_h' = effective horizontal stress

σ_v' = effective vertical stress

K_o = coefficient of lateral earth pressure at rest

This coefficient is very sensitive to the geologic and engineering stress history, as well as the unit weights of the overlying soil layers. The value of K_o is important in stress analyses, in assessing the shearing resistance of particular soil layers and in many foundation engineering problems. The K_o in a natural deposit can be as low as 0.4 or 0.5 for sedimentary soils (even less for peat) that have never been preloaded, or as high as 3.0 or more for some very heavily preloaded deposits.

Seepage Effects

When water flows through soils, it exerts forces called "seepage forces" on the individual soil grains. These seepage forces affect the intergranular or effective stress in the soil mass. Water flows through porous media because of a hydraulic gradient or driving head. As the driving head increases, the seepage forces increase and gradually overcome the gravitational forces acting in the soil mass. Eventually a quick condition (quick meaning alive) or boiling will occur.

If you dewater a braced excavation pit next to a body of water (e.g., a lake), you create a driving head and seepage forces on the sands at the bottom of the excavation because the excavation is surrounded by higher water. There will come a critical head where the bottom will become quick, a condition and not a material, where the effective stresses fall to zero.

Holtz and Kovacs (1981) point out that, contrary to popular belief, it is not possible to drown in quicksand unless you really work at it, because the density of quicksand is much greater than that of water. Since you can almost float in water, you should be able to float in quicksand.

Another phenomenon related to quicksand is "liquefaction." This can happen when a loose, saturated deposit of sand is subjected to dynamic loads of very short duration (such as those that occur during earthquakes), pile driving and blasting. The loose sand tries to densify during shear, and this tends to squeeze the water out of the pores. Under static loading, the sand normally has sufficient permeability so that the water can escape and any induced pore water pressures can dissipate.

But in this situation--because the loading occurs in such a short time--the water doesn't have time to escape, and the pore water pressure increases. Since the total stresses have not yet increased during loading, the effective stresses then tend toward zero (Equation 4.8), and the soil loses all strength.

It has been found that liquefaction can occur in even moderately dense sands due to a repeated or cyclic application of shear stress--which means that if an earthquake lasted long enough, then even moderately dense sands could possibly liquefy. (Note that pile driving is an example of repeated loadings.)

CONSOLIDATION AND SETTLEMENTS

The interrelations between stress, strain and time for soils is not simple and cannot be treated mathematically with present theory. Soils have another property that complicates matters: they have "memory" (Great Lakes ice has no memory--it melts each spring); consequently, they are "nonconservative." When soils are stressed, they deform, and even when the stress is released, some permanent deformation remains. Deformations in general can be a change of shape (distortion) or a change of volume (compression) or both.

When a soil is loaded, it will compress due to deformation of the soil grains, compression of air and water in the voids, and/or a squeezing out of the water and air in the voids. The amount of compression of the soil mineral grains themselves is small and usually neglected. Soils below the water table are considered to be fully saturated. The water itself doesn't compress. The squeezing out of the water, therefore, contributes most to the volume change of loaded soil deposits.

Squeezing out the water doesn't mean the soil is wrung dry. As the pore fluid is squeezed out, the soil grains rearrange themselves into a more stable and denser configuration, and a decrease in volume and accompanying surface settlement results (i.e., the void ratio gets smaller). How fast this process occurs depends on the permeability of the soil. How much rearrangement and compression takes place depends on the rigidity of the soil skeleton, which is a function of the structure of the soil. Soil structure depends on the geologic and engineering history of the deposit (the character of natural deposits is important; Great Lakes coastal deposits will be reviewed later).

A soil's memory preserves the past stresses and other changes in its structure. When a soil deposit is loaded to a stress level greater than it has ever experienced in the past, the soil structure is no longer able to sustain the increased load, and the structure starts to deform at an increasing rate and to break down. The stress that the soil has sustained in the past is known as the "preconsolidation pressure." The soil is is said to be "normally consolidated" when the preconsolidation pressure just equals the existing effective vertical overburden pressure (σ_v'). If a soil's preconsolidation pressure is greater than the existing overburden pressure, then it is said to be "overconsolidated" (or "preconsolidated").

The "overconsolidation ratio" (OCR) can be defined as the ratio of the preconsolidated stress to the existing vertical effective stress. Soils that are normally consolidated have an OCR equal to 1; soils with an OCR greater than 1 are overconsolidated.

A soil may be overconsolidated for many reasons. The effective stress could be altered by either a change in the total stress or a change in pore water pressure. Dessication of the upper layers of a deposit, due to surface drying, will produce overconsolidation. Geologic deposition and subsequent erosion, for example, is a change in the total stress that will preconsolidate the underlying soils. This can happen from glaciation and from the removal of overburden naturally or by man.

The settlement and rate of settlement of natural deposits can be computed from laboratory test data, but the purposes at hand do not require an examination of consolidation and settlement theories or a knowledge of exactly how these

computations are made. Small-craft harbors involve light loadings, so settlements are of concern only in unusual cases.

FAILURE CRITERION AND SHEAR STRENGTH

The theories used to estimate consolidation of deposits are one-dimensional models. This section describes the reaction of sands and clays to types of loading that are not one-dimensional. If the load or stress in a foundation or earth slope is increased until the deformations become unacceptably large, the soil in the foundation or slope is said to have "failed"--referring to the "strength" of the soil, which is really the maximum or ultimate stress the material can sustain. In geotechnical engineering, the "shear strength" of soils is the major concern because, for most problems in foundations and earthwork engineering, failure results from excessive applied shear stresses.

Failure Criteria

The strength of a material is defined as the maximum or yield stress, or the stress at some strain that is defined as "failure." There are many ways of defining failure in real materials—which is to say that there are many "failure criteria." If the material is brittle, its point of failure is obvious. If it is a work-softening material, the peak of the stress strain curve is sometimes used, or a set amount of strain is used (see stress strain curves in Figure 3.1 for different types of brittle and strain-softening ice.)

Most failure criteria don't work for soils. In fact, the one used doesn't always work so well either. Nonetheless, the most common failure criterion applied to soils is the "Mohr-Coulomb failure criterion."

About the turn of the century, Mohr (1900) hypothesized a criterion of failure for real materials in which he said that materials fail when the "shear stress on the failure plane at failure reaches some unique function of the normal stress on that plane." The functional relationship between shear stress and normal stress at failure is expressed by a limiting or failure envelope of the shear stress. This envelope is called the "Mohr failure envelope."

Enter Monsieur Dr. Coulomb. Besides his famous experiments with cats' fur and ebony rods, Coulomb (1776) was also concerned with military defense works like reventments and fortress walls. At that time, these constructions were built by rule of thumb, and, unfortunately for the French military, many of these works failed. Coulomb became interested in the problem of the lateral pressures exerted against retaining walls, and he devised a system for analyzing earth pressures against retaining structures that is still used today.

One of the things he needed for design was the shearing strength of soil. Since he was also interested in the sliding friction characteristics of different materials, he set up a device for determining the shear resistance of soils. He observed that there was a stress-independent component of shear strength, and a stress-dependent component. The stress-dependent component is similar to sliding friction in solids, so he called this component the "angle of internal friction," denoting it with the symbol ϕ. The other component seemed to be related to the intrinsic "cohesion" of the material and is commonly denoted with the symbol c.

Mohr-Coulomb Strength Criterion

Who first did so is unknown, but the Coulomb formulation was soon combined with the Mohr failure criterion. Engineers traditionally prefer to work with straight lines, since anything higher than a first-order equation (straight line) gets too complicated. So the natural thing to do was to straighten out the curve in the Mohr failure envelope--or at least approximate the curve by a straight line over some given stress range--then the equation for that line in terms of Coulomb's parameters ϕ and c (the so-called "strength parameters") could be written (these parameters can also be "primed" as c' and ϕ' for effective stress conditions.) Thus was born the "Mohr-Coulomb strength criterion," which is by far the most popular strength criterion applied to soils. The Mohr-Coulomb criterion can be written as:

$$s = \sigma \tan\phi + c \qquad (4.12)$$

where: s = the shear stress on the failure plane

σ = the normal stress on the failure plane

ϕ = the angle of internal friction

c = cohesion

This is a simple, easy-to-use criterion that has many distinct advantages over other failure criteria. It is the only failure criterion which predicts the stresses on the failure plane at failure. Figure 4.1 shows the Mohr-Coulomb criterion for a c-ϕ soil, a ϕ-soil (no cohesion), and a c-soil (cohesive).

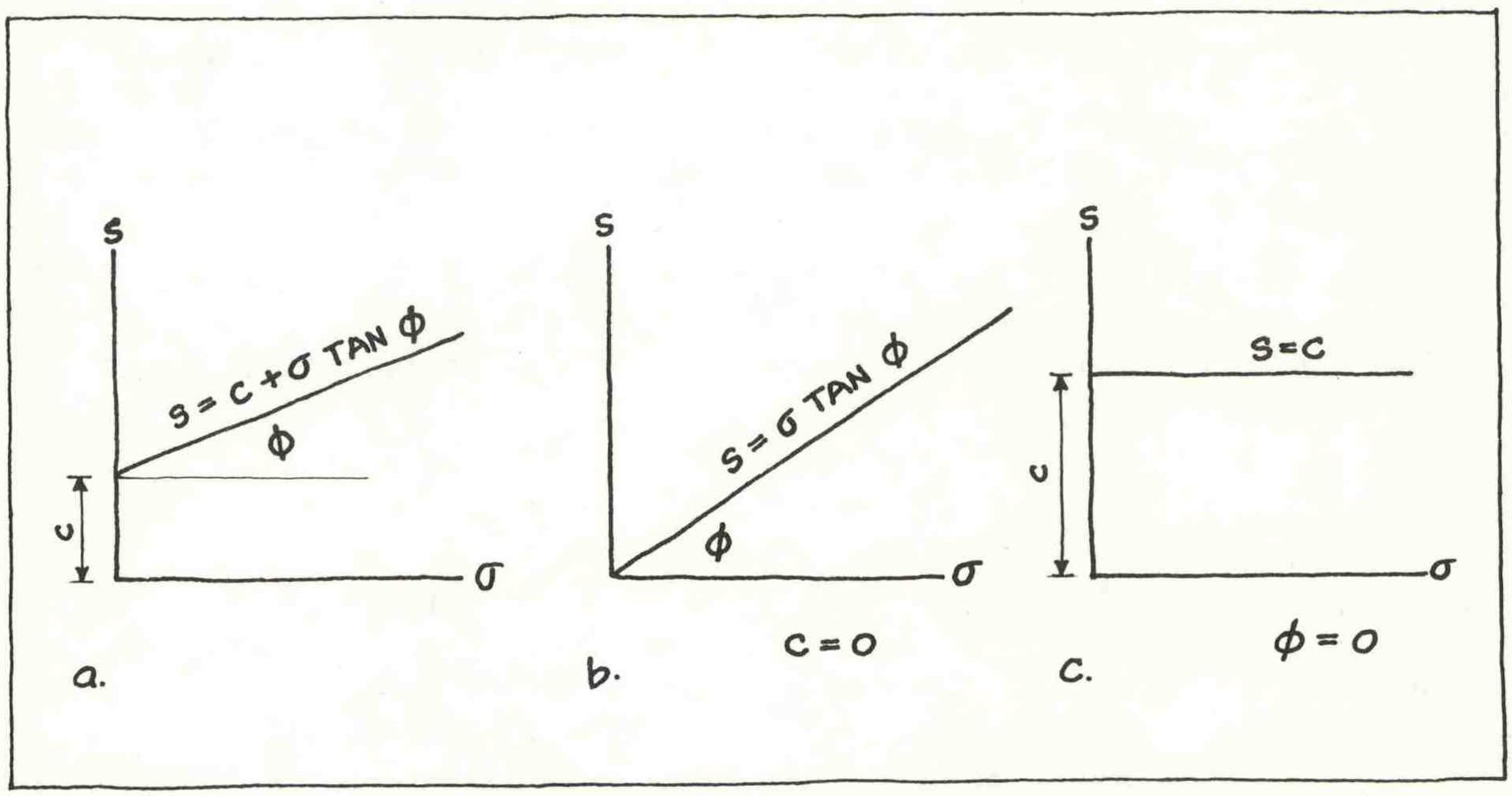


FIGURE 4.1: Mohr-Coulomb strength criterion for (a) c-ϕ soil, (b) ϕ-soil (cohesionless) and (c) c-soil (cohesive).

Direct Shear Test

There are several kinds of tests for determining the Mohr-Coulomb strength parameters. The inexpensive, fast and simple direct shear stress is probably the oldest, because Coulomb used a type of shear box more than 200 years ago to determine the necessary parameters for his strength equation. The specimen container or "shear box" is separated horizontally into halves. One half is fixed; with respect to that half, the other half is either pushed or pulled horizontally. A normal load is applied to the soil specimen in the shear box through a rigid loading cap. Dividing the shear force and the normal force by the nominal area of the specimen produces the shear stress as well as the normal stress on the failure plane (which has been forced to be horizontal with the apparatus).

Triaxial Shear Tests

The triaxial test, though more complicated than the direct shear test, is much more versatile. A cylindrical soil specimen, usually encased in a rubber membrane, is placed in a cell chamber that permits pressurizing the sample to replicate in situ conditions. At given pressures, axial loads are applied until failure. Drainage from the sample can also be controlled, and often the volume change of the sample during a drained test, or the induced pore water pressure during an undrained test, is measured. The failure plane is not forced—the specimen is free to fail on any weak plane or, as sometimes occurs, to simply bulge.

Drainage conditions in the triaxial test are models of specific critical design situations for stability in engineering practice. These are commonly designated by a two-letter symbol. The first letter refers to what happens "before shear"—that is, whether the specimen is consolidated. The second letter refers to the drainage conditions "during shear." The three permissible drainage paths in the triaxial test are the consolidated-drained (CD), consolidated-undrained (CU), and the unconsolidated-undrained (UU). The unconsolidated-drained test defies interpretation and is therefore meaningless (it models no real engineering design situation: drainage would occur during shear, and the effects of the confining pressure cannot be separated from the shear stress.)

These triaxial and other strengths and penetration tests are described in the following paragraphs. Some details about the test procedures are presented, but the applicability of the several tests to actual field conditions is what is most important here.

Consolidated-Drained (CD) Test

In the consolidated-drained test, the specimen is consolidated under some state of stress appropriate to the field or design situation. This test is most suitable for sandy soils. When the consolidation is over, the "C" part of the CD test is complete. During the "D" part, the apparatus' drainage valves remain open and the stress difference is applied very slowly so that essentially no excess pore water pressure develops during the test. Excess pore water pressure is excess above initial hydrostatic and is the pressure that would be induced by quickly loading a saturated sample. The CD test

models conditions for the long-term steady seepage case for embankment dams and the long-term stability of excavations or slopes in both soft and stiff clays.

It is not easy to conduct a CD test on a clay in the laboratory. To ensure that no pore pressure is really induced in the specimen during shear for materials with very low permeabilities, the rate of loading must be very slow. The time required to fail the specimen ranges from a day to several weeks (because of required low strain rates, thermal ice tests similarly require long times). The CD test is also referred to as the S test—the slow test. Because it is possible to measure the induced pore pressure in a consolidated-undrained (CU) test and thereby calculate the effective stresses in the specimen, CU tests are more practical for obtaining the effective stress strength parameters.

Consolidated-Undrained (CU) Test

In the consolidated-undrained test the specimen is first consolidated (with drainage valves open) under the desired consolidation stresses. After consolidation is complete, the drainage valves are closed and the specimen is loaded to failure in undrained shear. Often the pore water pressures developed during shear are measured, and both the total and effective stresses may be calculated during shear and at failure. This test can either be a total stress or effective stress test. This test is sometimes referred to as the R test—the rapid test.

CU strengths are used for stability problems where the soils have first become fully consolidated and are at equilibrium with the existing stress system. Then, for some reason, additional stresses are applied quickly, with no drainage occurring. Practical examples include rapid drawdown of embankment dams and the slopes of reservoirs and canals. In terms of effective stresses, CU test results are also applied to the field situations mentioned in the preceding discussion of CD tests.

Unconsolidated-Undrained (UU) Test

In the unconsolidated-undrained test, the drainage valves are closed from the beginning. Thus, if the sample is 100 percent saturated, no consolidation can occur, even when chamber confining pressure is applied. As with the CU test, the specimen is sheared undrained. The sample is loaded to failure in about 10 to 20 minutes; usually pore water pressures are not measured in this test. This test is a "total stress test," and it yields soil strength in terms of total stresses. This test, sometimes referred to as the Q test--the quick test—is used only with cohesive soils.

Like the CD and the CU tests, the UU strength is applicable to certain critical design situations in engineering practice. These situations are where the engineering loading is assumed to take place so rapidly that there is no time for the induced excess pore water pressure to dissipate or for consolidation to occur during the loading period. It is also assumed that the change in total stress during construction does not affect the in situ undrained shear strength. Examples include the end of construction of embankment dams and foundations for embankments, piles and footings on

normally consolidated clays. In these cases, often the most critical design condition is immediately after the application of the load (at the end of construction), when the induced pore pressure is the greatest but consolidation has not yet had time to take place. Once consolidation begins, the void ratio and the water content naturally decrease and the strength increases, so the embankment or foundation becomes increasingly safer with time.

Undrained Shear Strength and Unconfined Compressive Strength

In natural deposits of sedimentary clays, the undrained shear strength has been found to increase with depth and thus is proportional to the increase in effective overburden stress with depth.

Another factor that strongly affects the undrained shear strength of clays is stress history; the difference in behavior between normally consolidated and overconsolidated clays is represented by empirical relationships with the overconsolidation ratio (OCR).

Finally, the problem of the residual strength of soils should be mentioned. When stiff overconsolidated clays work-soften under large strains, the ultimate strength, called the residual strength, is less. Needless to say, determining shear strengths of clay soils is very complex.

Theoretically, an "unconfined compression test" can be conducted to obtain the UU-total stress strength. This is a special case of the UU test, with the confining or cell pressure equal to zero (atmospheric pressure). A triaxial cell is not required for the unconfined compression test: the cylindrical sample is compressed uniaxially with no confinement--like the tests on ice samples. Practically speaking, for the unconfined compression test to yield the same strength as the UU test, several assumptions must be satisfied. The specimen must be intact, homogeneous fine-grained soil; it must be sheared rapidly to failure, and it must be 100 percent saturated (otherwise, compression of the air in the voids will occur, causing a decrease in the void ratio and an increase in strength). The unconfined compression test should not be used on partially saturated or fissured samples.

From statics it can be shown that, in the absence of a confining pressure, the maximum shear stress occurs on a plane making a 45° angle with the loaded surface and that it equals half the failure load applied. In other words, the shear strength (cohesion) is equal to about half the unconfined compressive strength. For very soft materials, the failure may not occur along a diagonal plane; the sample bulges instead, and failure is generally assumed when the axial strain has reached 20 percent. This strength test is far and away the most common laboratory strength test used in the U.S. today for the design of shallow and pile foundations in clay.

Penetration and Other Tests

Besides the unconfined compression or the UU triaxial tests, other methods can be used to obtain the undrained shear strength of cohesive soils. Due to all the problems associated with sampling and laboratory testing, it is sometimes better to measure the strength directly in the field. The two most common field tests for soft clays are the vane shear test (VST), where two crossed

blades on a vertical rod are pushed into the clay at the bottom of a bore hole and the rod torqued to fail the clay, and the Dutch cone penetrometer test (CPT), where a 60° cone is pushed into the clay (or sand) and the point resistance and the friction on a friction sleeve attached above the cone are measured. These two tests are not widely used in the U.S.; instead, the standard penetration test (SPT) is most often used for granular and sometimes cohesive soils.

The standard penetration test is a very important procedure used for sampling and penetration testing. A standard "split-spoon" sampler is driven by a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler 12 inches deep is called the "standard penetration resistance," or "blow count" (N). The split-spoon is a cylinder which, when withdrawn from the bore hole, can be uncoupled and opened along longitudinal seams and the "disturbed" soil sample recovered. Split-spoons are used in sand and clays, though better clay samples--so-called "undisturbed" samples--are obtained by pushing an intact thin-wall sampler (a Shelby tube) into the deposit and then bringing it to the surface. The recovered clay sample is later extruded in the laboratory.

As discussed in Chapter 10, N-values from the SPT procedure are used to correlate with many soil parameters. N-values are generally used beyond their inherent capabilities for prediction, but they are used nevertheless. They are very much a part of conventional soil exploration programs, even though they have limitations in mixed soils like Great Lakes glacial tills and they are often misleading.

There are other field and laboratory tests. One of them the simple pocket penetrometer (PP) test. The pocket penetrometer, which can literally be carried in a pocket, is a small solid cylindrical probe that is pushed a prescribed distance into a cohesive soil sample (or the wall of a test pit, etc.). The penetrometer has a calibrated spring that registers penetration resistance and hence the unconfined compressive strength, usually called the "approximate" unconfined compressive strength.

Shear Strengths and Limiting Equilibrium

Before concluding this section on failure criteria and shear strength, a couple more items need to mentioned. The shear strength of soils is a most important aspect of geotechnical engineering. The bearing capacity of shallow or deep foundations, slope stability, retaining wall design and, indirectly, pavement design are all affected by the shear strength of the soil in a slope, behind a retaining wall or supporting a foundation or pavement. Structures and slopes must be stable and secure against total collapse when subjected to maximum anticipated applied loads.

Thus "limiting equilibrium" methods of analysis conventionally are used for design of structures and slopes, and these methods require a determination of the ultimate or limiting shear resistance (shear strength) of the soil. The deformations needed to mobilize strength must be considered, since structural failures may occur at significantly smaller strains than the 20 percent value often associated with loose sand or normally consolidated clay lab tests.

The converse of strength mobilization involves consideration of allowable strains. The shear strength of a soil was defined earlier as the ultimate or

maximum shear stress the soil can withstand. Sometimes the limiting value of shear stress was based on a maximum allowable strain or deformation. Very often, this allowable deformation actually controls the design of a structure because, with the large safety factors used, the actual shear stresses in the soil produced by the applied loads are much less than the stresses causing collapse or failure.

Strength and deformation both must be considered in geotechnical engineering practice.

Sensitive Clays

Only test results on "well-behaved" sands and clay soils have been discussed here so far; special soils--such as cemented sands, stiff fissured clays, highly sensitive ("quick") clays and organic soils—were not considered in any detail. This approach is admittedly classical ("textbook"), and I hope it was not overly simplified.

However, "sensitivity" needs to be discussed. Sensitivity is the ratio of the undisturbed, or in situ, natural strength of clay to its remolded strength, or strength after being worked. Sensitivity is usually based on the ratio of the undisturbed to the remolded unconfined compressive strengths. Highly sensitive clays are rare in the United States, but sensitive clays exist in other parts of the world, especially eastern Canada and Scandinavia.

Canadian Leda clays are often very stiff in their natural state, but their strengths are so low when they are thoroughly remolded that their sensitivity ratio is about 1500--which Holtz and Kovacs (1981) describe as going beyond "quick" at greater than 50, beyond "extra quick" greater than 100, and thence to "greased lightning." Upon being disturbed, this clay changes from load-supporting to a pourable slurry—at the same water content.

Total Stress vs. Effective Stress Approaches

The two fundamentally different approaches to the solution of stability problems in geotechnical engineering are: (1) the "total stress approach" and (2) the "effective stress approach." In the total stress approach, no drainage is allowed to take place during the shear test, and it is assumed--and admittedly it is a big assumption—that the pore water pressure and thus the effective stresses in the test specimen are identical to those in the field. This method of analysis uses the total, "undrained shear" strength of the soil.

The second approach to calculate the stability of foundations, embankments, slopes, etc., uses the shear strength in terms of effective stresses. This approach requires measurements or estimates of the excess hydrostatic pressure, both in the laboratory and in the field. Then, with some knowledge of the initial and applied total stresses, the effective stresses acting in the soil may be calculated. Since shear strength and stress-deformation behavior of soils are believed to really be controlled by effective stress, this second approach is philosophically more satisfying. This method of analysis uses the "drained shear strength" or shear strength in terms of effective stresses. The drained shear strength is ordinarily only determined by laboratory tests.

5. Introduction to Foundation Design

This chapter introduces some basic aspects of foundation design of structures and elements in small-craft harbors. Mostly it covers basic design approaches and the special problems stemming from the presence of ice and water. It does not cover the detailed design of conventional items such as spread footings, bulkheads, engineered fills, cut slopes, etc.

Since the properties of steel and concrete--or carefully selected natural materials like timber and stone--can be determined reliably, the problems associated with design almost can be solved by direct application of theory or the results of model tests.

On the other hand, as Terzaghi and Peck (1948) remind us, every statement and conclusion pertaining to soils in the field involves many uncertainties. In extreme cases, the concepts on which a design is based are no more than crude working hypotheses that may be far from the truth. In such cases, the risk of partial or total failure can be eliminated only by using what may be called the observational procedure.

The observational procedure consists of making appropriate observations early enough during construction to detect any signs that the real conditions are departing from those assumed by the designer, and then modifying either the design or method of construction in accordance with these findings. (This procedure also can be used after construction in anticipation of improving performance with the next construction.)

Terzaghi and Peck also note that a complicated theory serves no practical purpose until the results are condensed into graphs and tables that permit rapid evaluation of the final equations on the basis of several different assumptions. (I hope to do this; meanwhile, complex theories are presented here only in abstract form.)

The material in the next two sections is taken largely from Vesic (1975) and Bowles (1982), and others as cited. It deals with soils, not rock.

BEARING CAPACITY--AN INTRODUCTION TO THE BASIC THEORY

It is known from observations of the behavior of foundations subjected to load that bearing capacity failure usually occurs as a shear failure of the soil supporting the footing. The three principal modes of shear failure under foundations have been described in the literature as general shear failure, local shear failure and punching shear failure (Figure 5.1).

General shear failure (Fig. 5.1a) is characterized by the existence of a well-defined failure pattern consisting of a continuous slip surface from one edge of the footing to the ground surface. In the stress-controlled conditions under which most foundations operate, failure is sudden and catastrophic. The failure is accompanied by substantial tilting of the foundation. Under strain-controlled conditions (e.g., when the load is

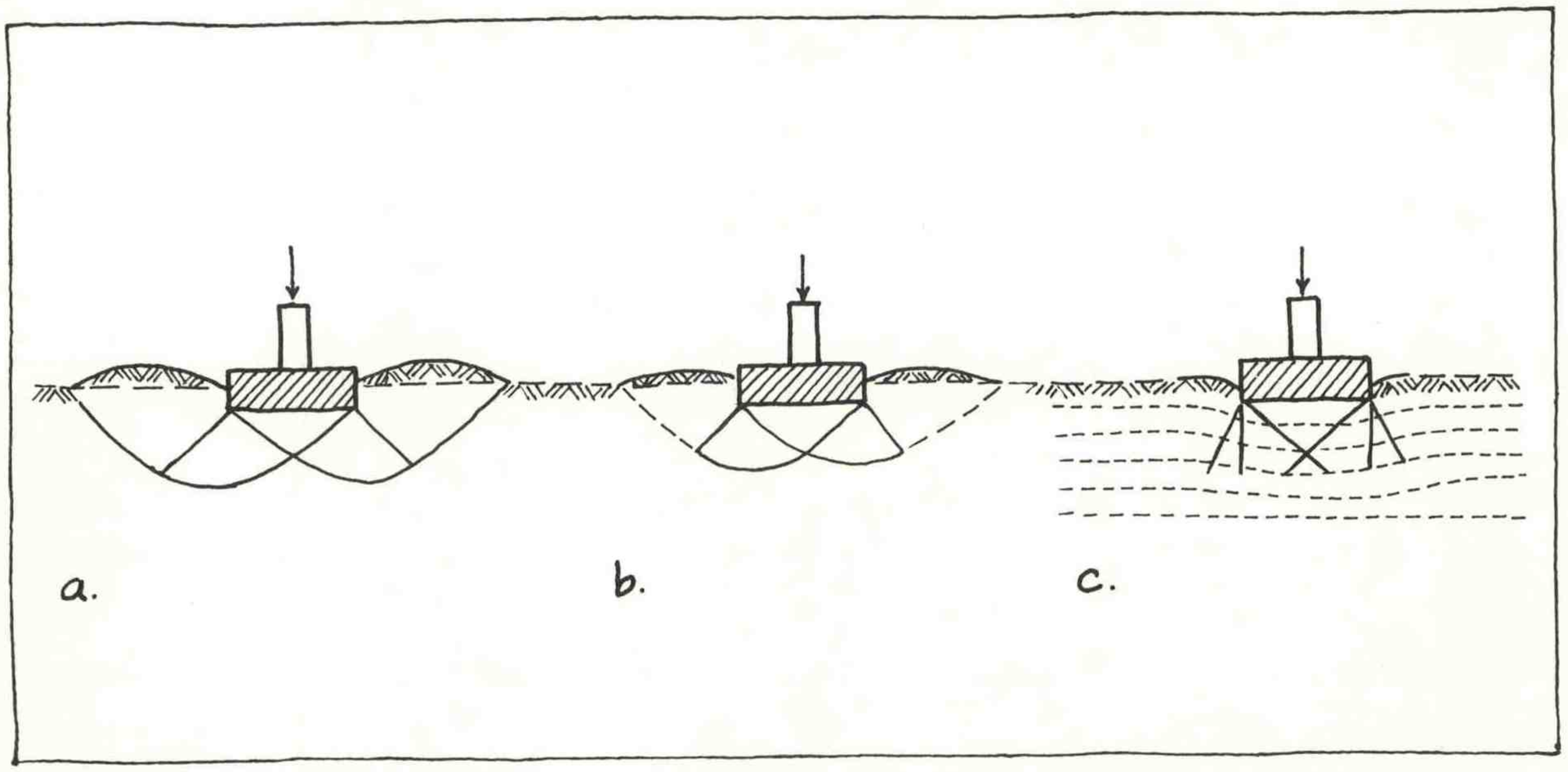


FIGURE 5.1: Modes of Bearing Capacity Failure: (a) General Shear Failure, (b) Local Shear Failure, (c) Punching Shear Failure

transmitted by jacking), a visible decrease of load necessary to produce footing movement after failure may be observed. A tendency for bulging adjacent soil can be recorded through most of the loading process on both sides of the footing, though the final soil collapse occurs only on one side.

Local shear failure (Fig. 5.1b) is characterized by a failure pattern that is clearly defined only immediately below the foundation. This pattern consists of a wedge and slip surfaces, which start at the edges of the footing, as in the case of general shear failure. The soil tends to bulge visibly on the sides of the footing; however, the vertical compression under the footing is significant, and the slip surfaces end somewhere in the soil mass. The slip surfaces may appear at the ground surface only after considerable vertical displacement of the footing (say, up to half the footing width). Even then there is no catastrophic collapse or tilting of the footing, which remains deeply imbedded, mobilizing the resistance of deeper strata. Thus, local shear failure has some characteristics of both general shear and punching shear modes of failure and represents a transitional mode.

Punching shear failure (Fig. 5.1c) is also characterized by a failure pattern that is not easy to observe. As the load increases, the vertical movement of the footing is accompanied by compression of the soil immediately underneath. Continued penetration of the footing is made possible by vertical shear around the footing perimeter. Except for sudden small movements ("jerks") of the foundation in the vertical direction, there is neither visible collapse nor substantial tilting. Continuous increase in vertical load is needed to maintain footing movement in the vertical direction.

The mode of failure to be expected in any particular case depends on a number of factors that only have been partially explored so far. Generally, the failure mode depends on the relative compressibility of the soil in the particular geometrical and loading conditions.

The mode of failure to be expected in any particular case depends on a number of factors that only have been partially explored so far. Generally, the failure mode depends on the relative compressibility of the soil in the particular geometrical and loading conditions.

If the soil is practically incompressible and has a finite shearing strength, it will fail in general shear. On the other hand, if a soil of given strength is very compressible, it will fail in punching shear. The soil type alone does not determine the mode of failure. For example, a footing on very dense sand can also fail in punching shear if the footing is placed at great depth or if it is loaded by a transient, dynamic load. Similarly, the same footing will fail in punching shear if the very dense sand below is underlain by any compressible stratum, such as loose sand or soft clay.

The "failure" of a loaded footing is clearly defined only in the case of general failure. In cases of local and punching, the point of failure is less clearly defined and often difficult to establish. How can the estimated ultimate capacity be computed, and what factors of safety should be applied for design?

Ultimate and Allowable Bearing Capacities

The capacities of theoretical prediction of the ultimate failure load are, strictly speaking, limited currently to relatively incompressible soils or to the general shear failure mode. However, common practice uses the available solutions for compressible soils as well, with possible reduction for the effects on compressibility.

Opinions differ regarding how to compute ultimate bearing capacity correctly. Over the past 40 years, a large number of equations/procedures have been proposed (though none in the last 10 years), but presently the bearing capacity equation is usually written as follows:

$$q_{ult} = cN_c + qN_q + (1/2)\gamma BN_\gamma \qquad (5.1)$$

where: q_{ult} = ultimate bearing capacity

c = soil cohesion

q = surcharge when footing base is below the ground surface; equal to depth times soil unit weight

γ = soil unit weight

B = width of footing

N_c, N_q, N_γ = bearing capacity factors

This solution for ultimate bearing capacity is based on a number of assumptions. The footing is assumed to be an infinite strip of width B. This is justified, strictly speaking, for a footing length-width ratio L/B of 10 and practically for a ratio greater than 5. Corrections for L/B ratios less than 5 and other shapes need be made. The soil mass is assumed to be of semi-infinite extent and is homogeneous (semi-infinite, or half-space, means the plane of the ground divides the infinitely extending soil mass below it, from

the blue sky above it). It has shear strength properties defined by a straight-line Mohr-Coulomb envelope, with strength characteristics c and ϕ, and a stress-strain curve of a rigid-plastic body. The shear strength of the overburden is neglected.

The bearing capacity factors--N_c, N_q and N_γ--are assumed functions of the strength parameter, ϕ. A great variation in proposed solutions to the bearing capacity problem exists in the literature, including differing values for these factors. While the variation in N_c and N_q values proposed remain relatively insignificant, the differences in N_γ are substantial, ranging from about a third to double the values shown in Table 5.1.

Table 5.1 and Equation 5.1 are used as follows. Suppose a 10-foot-wide long crib structure is placed 1 foot into a sandy harbor bottom with properties γ (bouyant) = 58 pounds per cubic foot, c = 300 pounds per square foot and $\phi = 30^\circ$:

$$q_{ult} = cN_c + qN_q + (1/2)\gamma BN_\gamma$$

$$= (300)(30.1) + (1)(58)(18.4) + (1/2)(58)(10)(22.4)$$

$$= 9,030 + 1,070 + 6,500$$

$$= 16,600 \text{ pounds per square foot}$$

The ultimate bearing capacity is 16,600 pounds per square foot. This value must now be divided by a safety factor to obtain the allowable soil pressure.

TABLE 5.1: Bearing Capacity Factors (Vesic 1975)

Angle of Internal Friction, ϕ	N_c	N_q	N_γ
0	5.1	1.0	0.0
5	6.5	1.6	0.5
10	8.4	2.5	1.2
15	11.0	3.9	2.7
20	14.8	6.4	5.4
25	20.7	10.7	10.9
30	30.1	18.4	22.4
35	46.1	33.3	48.0
40	75.3	64.2	109
45	134	135	272
50	267	319	764

Factors of Safety

The assessment of adequate safety of a component of a structure is a complex problem of optimization which can be properly resolved only with due considerations of serviceability and economy of the structure, as well as probability and consequences of failure. There are no generally accepted, consistent criteria that can be recommended for use in engineering design today.

The selection of safety factors for design cannot be made properly without assessing the degree of reliability of all other parameters that enter into design, such as design loads, strength and deformation characteristics of the soil mass, etc. Use Table 5.2 as a guide for reasonably homogeneous soil conditions. It is assumed that, where appropriate, all foundations will be analyzed also with respect to maximum tolerable total and differential settlement; if settlement governs the design, higher safety factors must be used. Table 5.2 is for shallow foundations--foundations in which the depth of the footing base below the ground surface is less than the footing width.

For our example of a marina crib wall, a safety factor of 3 would be appropriate. Therefore, the allowable soil pressure would be about 5,500 pounds per square foot (16,600 divided by 3).

Net Pressures vs. Gross Pressures

Allowable soil pressures--are they net pressures or gross pressures? You cannot necessarily know which is intended unless it is indicated by the person who makes the allowable soil pressure recommendation.

Net pressure is in excess of the existing overburden pressure that can be safely carried at the foundation depth. Gross pressure is the pressure that can be carried at the foundation depth, including the existing overburden pressure.

TABLE 5.2: Minimum Safety Factor for Design of Shallow Foundations (after Vesic 1975)

Characteristics of the Structure	Soil Exploration	
	Thorough Complete	Limited
Maximum design load likely to occur often; consequences of failure disastrous	3.0	4.0
Maximum design load may occur occasionally; consequences of failure serious	2.5	3.5
Maximum design load likely to occur	2.0	3.0

The bearing capacity equation is based on gross soil pressure (q_{ult}), which is everything above the foundation level. Settlements are caused only by a net increase in pressure over the existing overburden pressure. So, if the allowable pressure is based on the bearing capacity equation, the pressure is a gross pressure; if it is based on settlement considerations, it is a net pressure.

Technically, then, the 5,500 pounds per square foot allowable in this example should be reduced by the one-foot embedment, or by 58 pounds per square foot. In engineering practice, the allowable soil pressure would be about 5,400 pounds per square foot. Note also that it is customary to make this reduction this way--from the soil pressure derived <u>after</u> dividing the ultimate by the chosen safety factor, not before.

FACTORS AFFECTING BEARING CAPACITY

Equation 5.1 is a bearing capacity equation derived with the assumptions listed above and for approximate failure surfaces in the soil for a unit width of an infinitely long strip footing so that a plane-strain soil condition could be assumed. What effect will foundation shape have for a rectangular footing?

Shape Factors

The engineering approach to evaluation of the effect of foundation shape has been mostly semi-empirical. The bearing capacity factors for a long strip footing are modified by dimensionless parameters called shape factors. For rectangular footings the shape factors are:

$$s_c = 1 + (B/L)(N_q/N_c) \qquad (5.2a)$$

$$s_q = 1 + (B/L)(\tan\phi) \qquad (5.2b)$$

$$s_\gamma = 1 - (0.4)(B/L) \qquad (5.2c)$$

Shape factors are multipliers on the bearing capacity factors. Their use is illustrated with the previous example, but instead of a long crib, now assume the crib is 25 feet long:

$$s_c = 1 + (B/L)(N_q/N_c)$$
$$= 1 + (10/25)(18.4/30.1) = 1.24$$

$$s_q = 1 + (B/L)(\tan\phi)$$
$$= 1 + (10/25)(\tan 30^\circ) = 1.23$$

$$s_\gamma = 1 - (0.4)(B/L)$$
$$= 1 - (0.4)(10/25) = 0.84$$

$$q_{ult} = cs_c N_c + qs_q N_q + (1/2)\gamma B s_\gamma N_\gamma$$
$$= (1.24)(9,030) + (1.23)(1,070) + (0.84)(6,500)$$
$$= 11,200 + 1,320 + 5,460$$
$$q_{ult} = 17,980$$
$$q_{allow} = 17,980/3 = 6,000 \text{ pounds per square foot}$$

The allowable soil pressure is increased from 5,500 to 6,000 pounds per square foot, or about 10 percent. The method of analysis itself has errors on this order of magnitude.

Inclined and Eccentric Loadings

If our structure has a horizontal load, or an eccentric load, the bearing capacity estimate is somewhat more complicated. Failure can occur either by sliding of the footing along its base, or by general shear of the underlying soil. I will preclude sliding for now and examine only the supporting capacity of the soil. Soil supporting capacity can be calculated with inclination factors, which modify bearing-capacity factors just as did the shape factors. These inclination factors are complex in form and not presented here.

Theoretical and experimental investigations show that the following method is generally safe. The eccentricity of the load (e) is computed for one or both directions (x and/or y) and is equal to the overturning moment divided by the vertical force. The width and length of footing (B and L) are replaced by $B' = B - 2e_x$ and $L' = L - 2e_y$. B' is substituted in the bearing capacity equation for B.

For example, assume our 10-foot by 25-foot crib has a total vertical weight of 5,000 pounds per lineal foot, and there is a 2,000-pound-per-lineal-foot lateral load in the short direction that produces a 12,000-pound-foot-per-lineal-foot overturning moment.

$$e_x = \frac{M}{P} = \frac{12,000}{5,000} = 2.4 \text{ feet}$$

$$B' = B - 2e_x = 10 - 2(2.4) = 5.2 \text{ feet}$$

and $L' = L = 25$ feet (no overturning in the long direction)

$$q_{ult} = cs_c N_c + qs_q N_q + (1/2)\gamma B' s_\gamma N_\gamma$$
$$= 11,200 + 1,320 + (1/2)(58)(5.2)(0.84)(22.4)$$
$$= 11,200 + 1,320 + 2,840$$
$$q_{ult} = 15,360$$
$$q_{allow} = 15,360/3 = 5,100 \text{ pounds per square foot}$$

The ultimate load on the footing would be computed from:

$$P_{ult} = (q_{ult})(B')(L) \text{ or } L' \text{ if doubly eccentric}$$

Another approach to the lateral load would be to compute the trapezoidal soil pressure on the footing and see how it compares with the allowable:

$$p = \frac{P}{B}\left(1 \pm 6\,\frac{e}{B}\right) \tag{5.3}$$

where: p = soil pressure per lineal foot

P = axial load per lineal foot

B = footing width

e = eccentricity of load (moment divided by axial load)

$$p = \frac{5{,}000}{10}\left[1 \pm 6\left(\frac{2.4}{10}\right)\right]$$

$$= 500\,[1 \pm 1.44]$$

$$p = 1{,}220 \text{ and } -220 \text{ pounds per square foot}$$

In the example, the toe pressure (at the front of the crib) is 1,220 pounds per square foot (which is low), and the heel pressure (at back of crib where load is applied) is -220 pounds per square foot, or a small tension. This cannot exist (or at least we assume no tensile strength for the sandy bottom), and instead of a trapezodial soil pressure diagram, it is a triangular one where the toe pressure is a bit more than 1,220 pounds per square foot--still of no concern in this case.

Later sections will examine ice forces on cribs and walls and consider soil pressures, overturning and sliding.

Depth Factors

The above analyses for determining the ultimate load neglect the shearing resistance of the overburden. This is normally justified by the fact that the overburden is weaker than the bearing stratum. In some cases, however, the expected increase of bearing capacity due to shearing resistance of the overburden cannot be neglected. The effect of overburden is considered with "depth" factors, which are dimensionless parameters indicating the increase in individual terms of the bearing capacity equation.

Approximate values for three cases of deep footings, either square or circular, are given by Equation 5.4 for cohesive and cohesionless deposits:

$$q_{ult} = 9.3c + q \qquad \phi = 0^\circ \tag{5.4a}$$

$$q_{ult} = 58q \qquad \phi = 30^\circ \tag{5.4b}$$

$$q_{ult} = 496q \qquad \phi = 45^\circ \tag{5.4c}$$

where: q_{ult} = ultimate bearing capacity

c = soil cohesion

q = surcharge equal to the depth times the soil unit weight

Vesic (1975) states that the equations give results that are in fair agreement with observed point bearing capacities of driven piles in sand. Equation 5.4 indicates the increase in bearing capacity due to the "depth" effect, which occurs in conditions where the method of placement of the foundation (driving) causes significant lateral compression. Vesic also states there exists good evidence that this effect is practically nonexistent if the foundations are drilled in or buried and backfilled, or if the overburden strata are relatively compressible. For this reason, you are advised against introducing depth factors in design of shallow foundations.

Approximate Bearing Capacity Relationships for Clay

Note for the clays ($\phi = 0$) the following simple approximate relationships, in terms of c alone, can be made from Equations 5.1, 5.2a and 5.4a, and Table 5.1, by neglecting the surcharge term ($N_q = 1$) and eliminating the width term ($N_\gamma = 0$):

$$q_{ult} = 5c \qquad \text{long footings near surface} \qquad (5.5a)$$

$$q_{ult} = 6c \qquad \text{square footings near surface} \qquad (5.5b)$$

$$q_{ult} = 9c \qquad \text{pile point bearing capacity} \qquad (5.5c)$$

Also note that the allowable bearing capacity of footings in clay can be "roughed-out" as being equal to the unconfined compressive strength, assuming a safety factor of about 3; then the allowable bearing capacity equals about 2c, or c equals about half the allowable bearing capacity. As previously stated, c also equals about half the unconfined compressive strength, so the unconfined compressive strength is approximately equal to the bearing capacity of a cohesive soil--a statement frequently heard.

Soil Compressibility

Earlier, it was emphasized that all preceding analyses of ultimate loads are based on the assumption of incompressibility of soil, and that the analyses should be applied, strictly speaking, only to cases in which general shear failure of the soil is expected. A lack of rational methods for analyzing bearing capacity failure in the other two modes (punching shear failure and local shear failure) exists.

To satisfy the immediate needs of engineering practice, Terzaghi (1943) proposed the use of the same bearing capacity equation and factors with reduced strength characteristics c and ϕ. Terzaghi recommended that two-thirds c be used and that two-thirds to three-quarters ϕ be used (2/3 for smaller ϕ and 3/4 for larger ϕ). Such an approach may give satisfactory answers in some soils, though not always safe ones. Also, studies have since shown that there is a decrease in apparent values of bearing capacity factors with increase of footing size.

Foundation Roughness

The roughness of the foundation base has little affect on bearing capacity (vis-a-vis strain compatibility of base and soil) as long as the applied external loads remain vertical. In the case of inclined loads, the foundation roughness may limit the maximum horizontal component of the load to be transmitted across the contact surface of the base. However, experience indicates that most cast-in-place concrete foundations, by the way they are constructed, possess roughnesses defined by friction angles equal to or greater than ϕ of the underlying soil.

Schultze and Horn (1967) investigated the value of the angle of base friction between concrete and sandy gravel under high normal stresses. They horizontally moved model blocks of concrete (about 10 square feet in size) above and below a water level and at different foundation depths. Several of their data points are for normal loads of 1,000 to 2,000 pounds per square foot, with overburden of 75 to 150 pounds per square foot and with no overburden, and for submerged conditions. They indicate a friction angle of about 30°. The tangent of the friction angle is the coefficient of friction. Based on these tests, we can conservatively say that the coefficient of friction of concrete to sandy gravel, below the water table, is about one-half. (I presume a rock-filled timber crib also would be on the same order).

Schultze and Horn also state that the foundation depth, even without applying any lateral earth resistance (so-called passive resistance) to moving the concrete block laterally in the soil, affects the base friction favorably (i.e., increases it) because of the corresponding overburden pressure around the foundation base.

Submergence

Generally the submergence of soils will cause loss of all apparent cohesion, coming from capillary stresses or from weak cementation bonds. At the same time, the effective unit weight of submerged soils will be reduced to about half the weight of the same soils above the water table. Thus, through submergence, all three terms in the bearing capacity equation may be considerably reduced. For the water table at or near the level of the foundation base, submerged unit weight should be used.

Static vs. Dynamic Leadings

All the analyses of bearing capacity in the preceding paragraphs are conceived for static loading conditions. Tacitly assumed is that the footing load is increased gradually until failure at a loading rate slow enough that no viscous or inertial effects are felt. This assumption applies to conditions of most ordinary footings, which carry a certain dead load and are presumed to fail by a single application of excessive static live load. The rate of application of these loads affects, under these conditions, the bearing capacity only to the extent that it may be related to the rate of drainage of excess pore water pressure created in the supporting soil by the application of the loads. The selection of the shear strength parameters c and ϕ to be introduced in the analysis shall be made so as to take care of that effect.

Studies show that the conventional static analyses of bearing capacity can be used for footings subjected to moderately rapid loadings--if the strength

parameters introduced in the analysis are modified for strain rate effects. Footings subject to impact and vibratory loads still require a dynamic approach for analysis.

Nonhomogeneity of Soils

In discussing the effects of nonhomogeneity on bearing capacity, distinction must be made between two basic kinds of nonhomogeneity that can be encountered: erratic soil profiles, where judgment and worst-case design approaches would be used, and regular soil profiles, where soil strength increases or decreases with depth. The latter type of nonhomogeneity is treated with modified bearing capacity factors. However, it would not be logical to place a gravity structure on a thin sand layer overlying soft clay or peat in a harbor bottom, nor to found the structure on a bottom of loose sand, soft clay or peat. We would undercut the bottom and perhaps refill or try to improve the harbor bottom in-place, or we would avoid the bottom and select a deep foundation system.

Two recent harbor bottom in-situ improvements were reported at the 8th European Conference on Soil Mechanics and Foundation Engineering. DeWolf et al. (1983) describe ground improvement in the current construction of the new outer harbor rubble mound breakwater at Zeebrugge, Belgium. For about a quarter of the breakwater length, the upper 12 to 20 feet of subsoil consists of loose sands over soft clays. The top loose sands and soft clays are being dredged and replaced with relatively coarse sea sands. The sea sand is dumped in the dredged excavation and compacted by vertical vibrocompaction using a patented vibroprobe named "Starprofile."

The Starprofile vibrating probe is made from three steel plates, each about 1.5 feet wide and 6.5 feet high, welded together to form a star with three 120° rib segments. The probe is hung in an electric or hydraulic block and is lowered into the dumped sand from a jack-up platform by means of a heavy crane. The energy of the vertically vibrating probe is transferred to the soil mainly through the series of ribs working as individual pounders. The vibrating probe is introduced into the filled material and to a depth of about 6.5 feet into the natural soil layers underneath the dredged area. Satisfactory densification has been obtained in 30-foot water depths with probe "pricks" in gridded 20- to 40-square-foot bottom areas.

Järviö and Petäjä (1983) reported on techniques for improvement of bearing capacity and deformation characteristics of silty fine sand in a Helsinki shipyard harbor. The water depth was about 30 feet, and the silty fine sand was at or below the silt-content threshold given in the literature as where vibrocompaction is considered to be effective. They were quite successful in driving tapered concrete compaction piles (10" x 12" butt by 4" x 4" tip by 15' long). Suitable heavy Vibrocompaction or Terra-Probe equipment was not available in Finland at the time. The piles were driven through the water with a driving "follower" and were spaced on a 4-foot grid. Upon completing the compaction of the soil, they placed a 3-foot lift of crushed stone, which served as the sub-base for a concrete base of a raft caisson supporting their shipyard pier.

Settlements

Small-craft harbors usually involve light vertical loadings, and except in unusual cases, settlements are not a real concern. We wouldn't build on soft,

unsuitable soils. And if we did have these soils and heavy loads, we would use a deep piled foundation or a shallow foundation system specifically designed on a settlement analysis basis rather than an allowable bearing capacity approach for the ultimate load capacity.

For an idea of the settlements on sand (not soft clay or compressible materials), I use a simplified approach given in Duncan and Buchignani (1976) and Meyerhof (1965). When a footing on sand is loaded, it settles immediately due to the volume change and distortion in the sand beneath the footing.

One way to estimate these immediate settlements is by the standard penetration test N-values. When a load is applied over a limited area on clay, some settlement occurs immediately. In saturated clays, some immediate settlement occurs as a result of distortion, or a change of shape beneath the loaded area. But there is no immediate volume change, because time is required for water to drain from the clay (consolidation). Immediate settlement can be estimated by using elastic theory and derived charts for solution aid, and consolidation can be estimated by using well-established theories and procedures.

A simple method suggested by Meyerhof (1965) can be used to estimate the upper limit for immediate settlement on sands:

$$\rho_i = \frac{5p}{(N - 1.5)C_B} \qquad (5.6)$$

where: ρ_i = maximum immediate settlement (inches)

p = bearing pressure (tons per square foot)

N = SPT blow count

C_B = width correction

The width correction C_B varies as the width of the footing B. For B less than or equal to 4 feet, $C_B = 1.0$; for B = 8 feet, $C_B = 0.9$; and for B greater than or equal to 12 feet, $C_B = 0.8$. Assume our 10-foot by 25-foot crib has a vertical weight of 1,000 pounds per square foot (and no lateral forces) and a sand blow count of 10:

$$\rho_i = \frac{(5)(1{,}000/2{,}000)}{(10 - 1.5)(0.85)} = 0.35 \text{ inch}$$

The maximum long-time settlement can be estimated by applying a time rate factor. This factor is 1.0 at one month, 1.2 at one year, 1.4 at 10 years, and 1.5 at 30 years. At the end of several decades, the sand settlement would be about half an inch for the 1,000 pounds per square foot bearing pressure. If the load was higher, say 4,000 pounds per square foot (2 tsf), the settlement would increase to a couple of inches.

In a review of the design and performance of spread footings and rafts in relation to the prediction and control of settlement, Meyerhof (1965) noted that allowable bearing pressures for a given settlement of shallow foundations on sand and gravel, when estimated from penetration tests (standard and cone), are rather conservative. In a marina, a couple of inches of settlement can be tolerated, provided it is uniform and not isolated in one place.

RETAINING STRUCTURES, WALLS, AND BULKHEADS

This section highlights information on retaining walls, sheet pile structures and cellular structures. Generally, it is based on Winterkorn and Fang's Handbook on Foundation Engineering (1975). This reference, and many others, may be consulted for all the complexities of the theoretical and empirical design procedures used in practice. The intent here is to present a few concepts, like lateral earth pressures, and to list a few points helpful in harbor structures design.

Lateral Earth Pressures

The at-rest earth pressure coefficient introduced earlier, K_o, expresses the ratio of horizontal stress to vertical stress for an earth mass at rest. Jaky (1948) expressed this as $K_o = 1 - \sin \phi$. Jaky studied materials (wheat, rye, oats, barley, corn, beans and peas) in silo pressure tests. These materials were presumed to be granular in nature. Jaky also studied brick, concrete and wood fillings. The angle of wall friction was also noted to be always smaller than the angle of internal friction. In practice, passive and active earth pressure coefficients are also used. All three are related through deformation conditions. Lateral earth pressures will be examined here in the context of retaining walls.

To estimate the soil pressure on the back of retaining walls, the most important consideration is the deformation conditions imposed on the soil by the retaining wall, which exerts an overriding influence on the soil pressure. The earth pressure theories and deformation assumptions used to compute soil pressures are based on shear strength properties c and ϕ. It is important to realize that the range of values for c and ϕ may vary widely, depending on the loading conditions. If earth pressure calculations are to have any meaning, the choice of shear strength parameters must be based on proper consideration of the loading conditions to be encountered in the field.

The deformation conditions are generally controlled by the type of retaining structure adopted. The retaining structure may be of great rigidity and unyielding. When the wall does not move, then the earth pressure on the back of the wall is equal to that in the soil mass itself. This earth pressure is the at-rest pressure. An example is a basement wall braced at the top by a floor.

If the retaining structure is permitted to move away from the soil--either laterally or rotationally about some point, allowing a lateral expansion of the soil--the earth pressure decreases with increasing expansion. Further expansion will cause a shear failure of the soil in which a sliding wedge tends to move forward and downward (Figure 5.2a).

At this state of failure, the earth pressure is at the minimum value and additional deformation does not reduce the earth pressure any further. This minimum earth pressure is known as the "active earth pressure." The amount of movement or rotation required is between 0.1 and 0.4 percent of the height of the wall.

On the other hand, if the retaining structure is forced to move backward toward the soil, causing a lateral contraction of the soil, the force required to start the movement is greater than the earth pressure against a rigid and

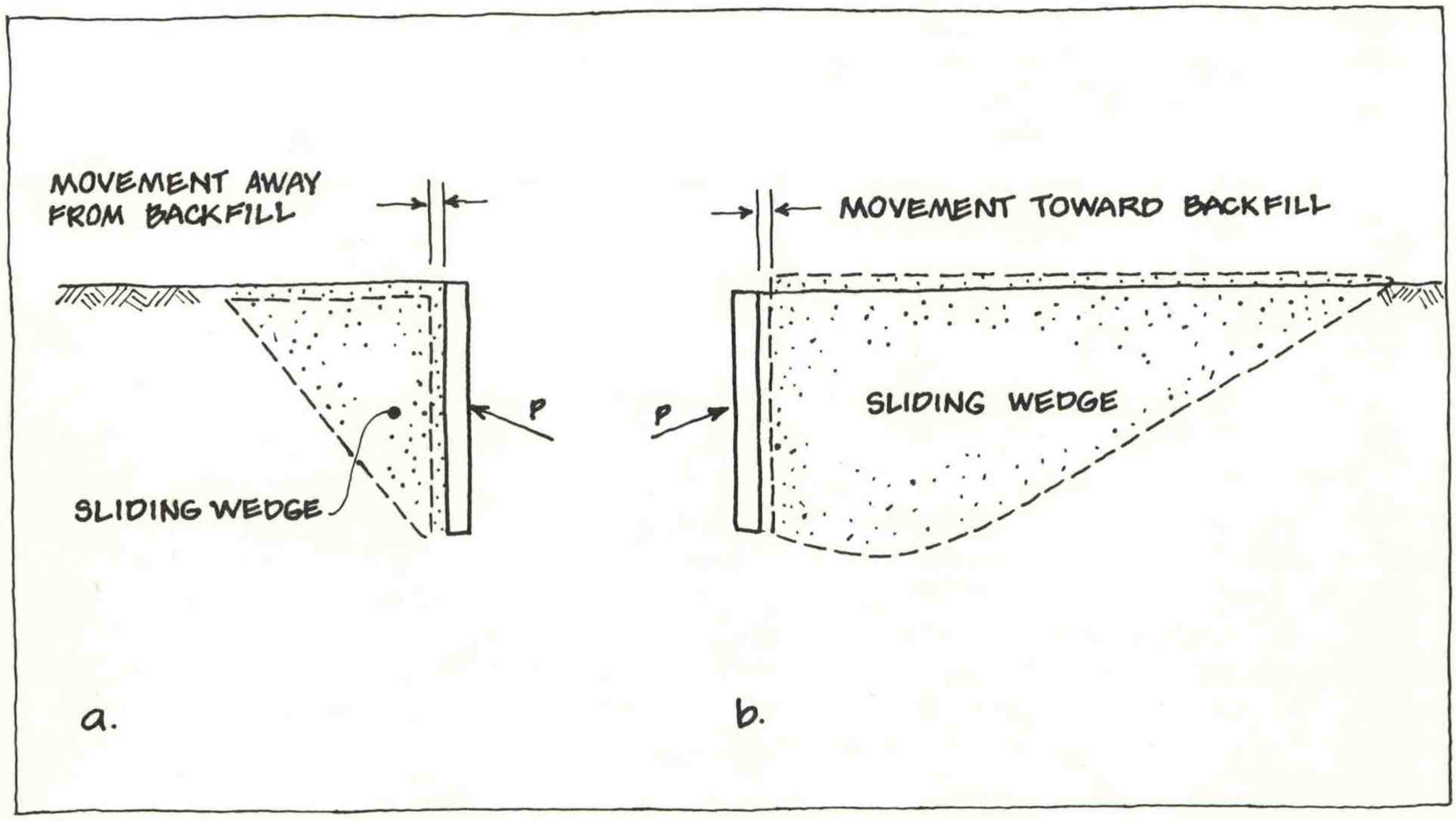


FIGURE 5.2a: Active Earth Pressure. 5.2b: Passive Earth Pressure.
(after Teng 1962)

unyielding wall. A larger force is required to move a greater distance (by about 50 to 100 times the active yielding) until a state of failure is reached where a sliding wedge is formed (Figure 5.2b). This wedge of soil moves backwards and upwards with respect to its original position. At this state of failure, the earth pressure is at a maximum value, known as "passive earth pressure" or "passive resistance." At this state, no greater force is required to introduce further movement of the wedge. (You wouldn't normally force a wall into the earth; but if a rigid piling is laterally loaded and rotates in the soil, you can visualize the development of passive resistances on the piling shaft.)

Typical values for soil unit weights and strength parameters are listed in Tables 5.3 and 5.4, but actual values for final design should be determined on the basis of tests. Any filling used behind walls should be clean, hard and free-draining, such as sand, gravel, broken stone or rock fill. Clays and silts should not be used as a filling material, because they produce high lateral pressures--two to three times as much.

Equations 5.7a through 5.7e give the active, passive and at-rest horizontal intensity of earth pressures against vertical walls with horizontal cohesionless and cohesive backfills or natural ground surfaces. If the backfill is partly or wholly saturated, then the pressure on the wall is sum of the pressure from the soil particles (intergranular pressure) and the pressure of water (pore water pressure). If drains are not provided, the pore water pressure is equal to the hydrostatic pressure, which results in large pressures on the wall and is uneconomical.

TABLE 5.3: Cohesionless Soils—Typical Values (Cornfield 1975)

Sands and Gravels Density	Natural Unit Weight (γ) (lbs./ft.^{3})	Submerged Unit Weight (γ') (lbs./ft.^{3})	Angle of Internal Friction (ϕ°)
Loose	90–125	55–65	30
Medium	110–130	60–70	35
Dense	110–140	65–80	40

TABLE 5.4: Cohesive Soils—Typical Values (Cornfield 1975)

Consistency	Cohesion (c) (lbs./ft.^{2})	Saturated Unit Weight (γ) (lbs./ft.^{3})	Submerged Unit Weight (γ') (lbs./ft.^{3})
Very soft	Under 375	90–100	30–40
Soft	375–750	90–110	30–50
Firm	750–1,500	105–110	45–65
Stiff	1,500–3,000	115–135	55–75
Very stiff	Over 3,000	120–140	60–80

EQUATIONS FOR HORIZONTAL INTENSITY OF EARTH PRESSURES AGAINST VERTICAL WALLS

$p_a = K_a \gamma h$ active pressure cohesionless soils (5.7a)

$p_p = K_p \gamma h$ passive pressure cohesionless soils (5.7b)

$p_a = \gamma h - 2c$ active pressure cohesive soils (5.7c)

$p_p = \gamma h + 2c$ passive pressure cohesive soils (5.7d)

$p_o = K_o \gamma h$ at-rest pressure cohesionless/cohesive soils (5.7e)

where: p_a = active pressure

p_p = passive pressure

p_o = at-rest pressure

K_a = active pressure coefficient

K_p = passive pressure coefficient

K_o = at-rest pressure coefficient

γ = unit weight of soil

h = depth into the soil mass

c = soil cohesion

Table 5.5 lists active and passive pressure coefficients for cohesionless, granular soils in terms of the angle of internal friction ϕ and the angle of wall friction δ.

For calculating active pressure, δ may be taken as one-half the value of ϕ. However, δ is sometimes neglected; that is, it is taken as zero when there is some doubt about the actual soil properties at the site. The error is on the safe side.

In calculating passive pressure, it is usual to take $\delta = (2/3)\phi$ for anchored sheet pile walls. However, downward resistance may not be sufficient to mobilize such a value of wall friction in the case of anchorages unless they have sufficient dead weight, so it is usual to take $\delta = 0^\circ$ for the design of anchorages. The possibility of a similar situation occurring in a cantilever retaining wall should also be considered.

Table 5.6 lists at-rest pressure coefficients K_o, which can be estimated with reasonable accuracy only in the case of normally consolidated soils.

TABLE 5.5: Active and Passive Pressure Coefficients for Cohesionless Soils (Cornfield 1975)

Angle of Internal Friction (ϕ)	Active Pressure Coefficient (K_a)	Passive Pressure Coefficient (K_p)
	Angle of Wall Friction, $\delta = 0^\circ$	
30°	0.33	3.0
35	0.27	3.7
40	0.22	4.6
	Angle of Wall Friction, $\delta = 10^\circ$	
30	0.31	4.0
35	0.25	4.8
40	0.20	6.5
	Angle of Wall Friction, $\delta = 20^\circ$	
30	0.28	4.9
35	0.23	6.0
40	0.19	8.8

TABLE 5.6 At-Rest Pressure Coefficients for Cohesionless/Cohesive Soils (Wu 1975)

Soil Description	At-Rest Pressure Coefficient (K_o)
All types, normally consolidated	$1 - \sin \phi$
Compacted clay, hand tamped	1.0 - 2.0
Compacted clay, entire backfill machine tamped	2.0 - 6.0
Clay, overconsolidated	1.0 - 4.0
Sand, loosely dumped	0.5
Sand, compacted	1.0 - 1.5

To illustrate the computation of earth pressures, I will compute the pressures against a frictionless wall at a depth of 10 feet for a clean loose sand and for a compacted clay. Assume $\phi = 30^\circ$ and c = 1,000 pounds per square foot. First, for the sand:

$$p_a = K_a \gamma h = (0.33)(115)(10) = 380 \text{ pounds per square foot}$$
$$p_p = K_p \gamma h = (3.0)(115)(10) = 3,450 \text{ pounds per square foot}$$
$$p_o = K_o \gamma h = (0.5)(115)(10) = 580 \text{ pounds per square foot}$$

And for the clay:

$$p_a = \gamma h - 2c = (110)(10) - (2)(1000) = -900 \text{ pounds per square foot}$$
$$p_p = \gamma h + 2c = (110)(10) + (2)(1000) = 3,100 \text{ pounds per square foot}$$
$$p_o = K_o \gamma h = (2)(110)(10) = 2,200 \text{ pounds per square foot}$$

Note several things: For the sands, the earth pressures range over an order of magnitude between active and passive states. For the clays, a negative active soil pressure was computed from $p_a = \gamma h - 2c$. This equation is for in situ cohesive soils and not for compacted clay wall backfill (which, as indicated, is poor practice). No tension in the soil is assumed, and an estimate of the active clay pressure can be obtained by dropping the 2c term. In this example, p_a = 1,100 pounds per square foot. The effect of this procedure--of computing $p_a = \gamma h$--is to use a pressure coefficient of unity; that is, the soil has no shear strength and its lateral pressure is equal to the vertical pressure (hydrostatic conditions).

With these earth pressure analysis methods, the stability and internal stresses (moments and shears) for loaded structures in contact with the earth can be computed. Standard statically determinate and statically indeterminate

methods of structural analysis are used. Unfortunately, many of the soil-structure interactions are statically indeterminate, and computation of stability and stresses must additionally involve more assumptions about the interaction behavior.

Tiebacks and Anchors

Tiebacks can be used to counter earth pressures and other forces. A tieback is a structural element which uses a grout anchor in the ground (or rock) to secure a tendon that applies a force to a structure. Figure 5.3 depicts one type of tieback system.

Weatherby (1982) states that permanent tiebacks are routinely installed in noncohesive soils with a standard penetration resistance (N) greater than 10 but are not routinely installed in soft to medium soils because their long-term loading capacities are questionable.

Tiebacks in normally consolidated clays with unconfined compressive strengths less than one ton per square foot and remolded strengths less than a half-ton per square foot may be creep-susceptible. Tieback tendons can easily be protected from corrosion.

To establish load holding characteristics and establish confidence in long-term performance, a tieback test program is recommended if permanent tiebacks are to be anchored in a cohesive soil, or in a noncohesive soil with a standard penetration resistance less than 10 blows per foot.

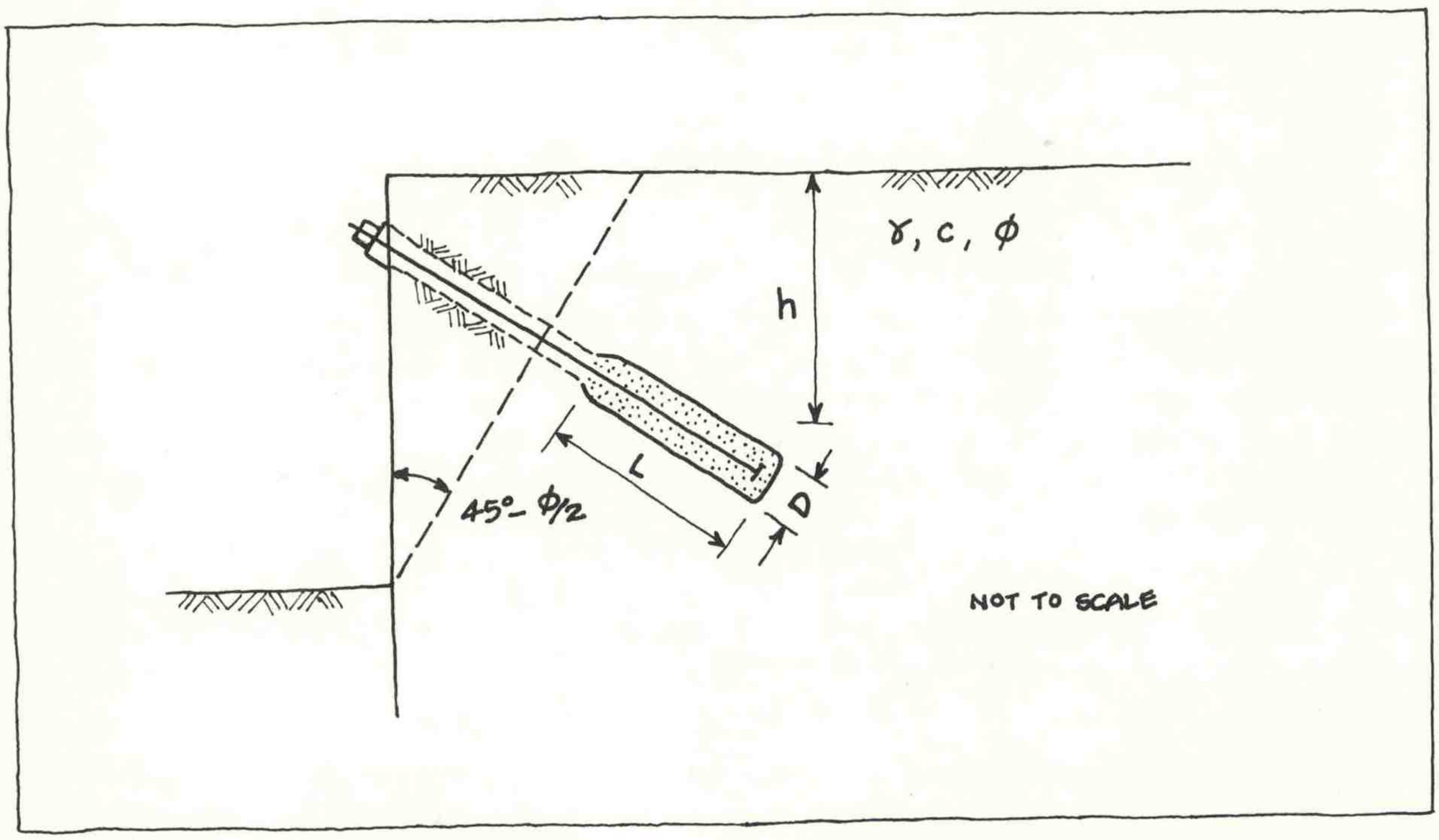


FIGURE 5.3: Grouted Tieback Anchor in a Predrilled Hole.

The capacity of soil tiebacks are estimated by using empirical relationships developed for the particular tieback type. Bowles (1982) gives the following formula for tieback anchor resistance (also see Fig. 5.3):

$$F = \pi D \gamma h L K \tan \phi + c_a \pi D L \qquad (5.8)$$

where: F = tieback anchor resistance

D = average grout diameter

γ = unit weight of soil

h = average depth of grouted length

L = grouted length

K = earth pressure coefficient

ϕ = angle of internal fricton

c_a = soil adhesion, taken to be about 0.7 to 0.9 c.

The use of $K = K_o$ can be justified if the grout is placed under pressure, which is most often the case; otherwise use $K = K_a$. Values of K greater than K_o are not recommended because of soil creep.

Wu (1975) reports that, in current design practice, the bond strength between anchor grout and the surrounding soil varies from 1,000 pounds per square foot (or 0.25 times the unconfined compressive strength) for stiff clay to 2,000 pounds per square foot for dense sand to 3,000 pounds per square foot for sound rock. It is the recommended practice to proof-test the anchors to at least 1.5 times the design load; a linear load-displacement relationship up to this load may be used as an acceptance criterion. Alternatively, the anchors may be required to hold the design load without appreciable relaxation.

Weatherby (1982) reports that a 50-ton design load tieback installed in sand costs between $1,000 and $2,500, and that a 50- to 70-ton design load tieback installed in clay costs between $1,000 and $3,500. Also, the design loads that are used for tiebacks range from 25 to 150 tons.

Because of costs and the design loads, it would seem that earth tiebacks would not really be a choice for overcoming ice uplift forces. However, all rock materials can generally be considered as suitable ground into which to found anchors. Large resistances can be developed. Table 5.7 lists typical bond stresses for rock anchors.

Crib Walls and Gravity Structures

Crib walls may be built of timber, steel or precast concrete members. The inside of cribs are filled with rocks and soil, and if the crib has a bottom, the whole unit acts as a gravity wall. If there is no bottom, the crib behaves like a filled cellular structure.

When computing the safety of the crib with respect to overturning, sliding and uplift, consideration must be given to the soil inside the crib. If the crib tips forward, the soil inside may not move as an integral part of the wall;

TABLE 5.7: Typical Bond Stresses for Rock Anchors
(Prestressed Concrete Institute 1974)

Type of Rock	Ultimate Bond[a] Rock and Grout Plug; Sound, Not Decayed (pounds per square inch)
Soft shales[b]	30-120
Slates and hard shales	120-200
Sandstone	120-250
Soft limestone[b]	150-220
Dolomite limestone	200-300
Concrete	200-400
Granite and basalt	250-450

a The bond between grout and the steel strand is about 450 pounds per square inch.
b Bond strength must be confirmed by pullout tests that include time creep tests.

rather it may move downward with respect to the wall, and shear stresses will develop between the soil and the wall. If the crib is being lifted, the soil may run out of the bottom.

Figure 5.4 is a photograph of a "failed" steel cell canister structure that has been removed and is on shore. The canister is a piece of corrugated metal culvert pipe and was set vertically into the harbor bottom and filled with sand. There was a concrete bottom poured and a concrete cap on the top to support a marina head pier. Ice shoving and lifting caused the bottom to slip out and then the sand fill to run out. The structure was no longer serviceable; it had "failed."

The use of several standard methods of analysis is necessary for design of cells. They are detailed in Dismuke (1975) and include two methods that are based on the assumption that the failure of the cell is an internal shear failure--either horizontal shear or vertical shear--and a method that assumes a base failure whereby the cell fill acts as a unit. These methods should be checked for bottomless gravity crib walls.

PILE FOUNDATIONS--AN INTRODUCTION TO ANALYSIS AND DESIGN

The introductory material in this section is taken mainly from Poulos and Davis (1980) and Vesic (1977). Poulos and Davis authored a book on pile foundation analysis and design, and Vesic wrote a synthesis-of-practice manual on the design of pile foundations. Not surprisingly, these authors do not agree on all scores, nor do they cover quite the same material. Material was

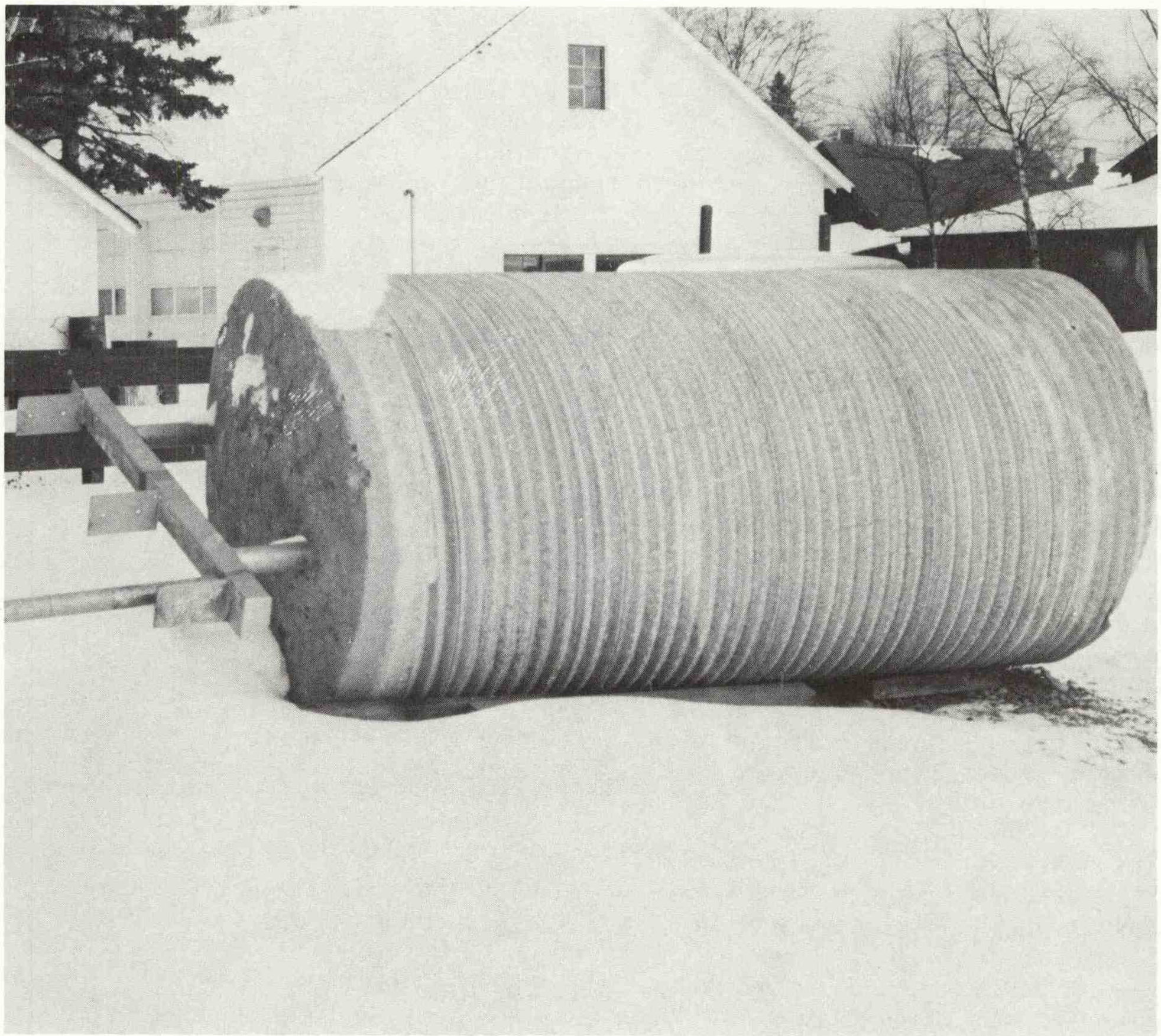


FIGURE 5.4: "Failed" Steel Cell Canister Structure.

also taken from Bowles (1982), Meyerhof (1976), Peck et al. (1974), Winterkorn and Fang (1975), and others as cited.

This section emphasizes friction piles—tension piles in particular—and, to a lesser extent, laterally loaded piles. Design of bearing piles design is not the main point of this manual; the main point is to develop deep foundations to resist major uplift loads and, to some extent, lateral loads as well.

Lateral loads, though occasionally bothersome, are not paramount. Furthermore, at this time little is known about lateral ice forces on individual piles, so there is no point to being finicky about lateral loads on marine piles. What is known about soil resistance to laterally loaded piles is not very precise either.

This introduction to pile foundations begins with several axioms and general principles.

General Behavior and Principles

Real soils often can be treated as elastic over a limited range of stresses, provided that the elastic parameters are determined for this stress range. When used in this manner, with due discretion and a measure of engineering judgment, elastic-based theory has had considerable success in predicting the deformation of both shallow and deep foundations. Since elastic theory allows consideration of stress transmission through a mass, it can be used to analyze the interaction between two or more piles and, therefore, to examine the behavior of groups of piles.

Our analysis is limited to single piles. Head pier piles connected with structural framing will act indeterminately and present an intermediate case between free-end piles and fixed-end piles, frequently joined in a concrete pile cap.

Design based on empiricism alone tends to focus attention on imperfectness because of inadequacy, because recorded experience generally only distinguishes between unsatisfactory and trouble-free performance, and rarely between economical and uneconomical design. Only by understanding the behavior of the engineering structure in an analytical as well as empirical sense can an engineer reasonably expect to achieve designs that are neither inadequate nor overadequate.

In reality, the loads on foundations determine their movement, but this movement affects the loads imposed by the structure; there is inevitable interaction between structure and foundation.

Piles embedded in soil (especially if closely spaced), provide a reinforcement to the soil, increasing its load capacity and modifying its deformation behavior in much the same way that steel reinforces concrete.

At the present state of knowledge, it is generally only possible to consider failure as something that occurs mainly at the interface between the side of the pile and the soil, ignoring details of failure within the soil, though for the pile base, ordinary bearing capacity theories may be applicable. For vertical failure, the shear stress at the pile shaft-soil interface attains a limiting value (possibly varying with depth and soil type); for horizontal failure resulting from lateral load or movement, the normal stress at the interface attains a limiting value (again, possibly varying with depth).

Vesic states that the soil always fails in the same manner: punching shear under the point, accompanied or preceded by direct shear failure along the shaft. Computation of the ultimate load is quite difficult and a "general solution" is not yet available. For design purposes, the ultimate load is separated into two components: the base or point load, and the shaft or skin load. Theories for determinating point load based on the plasticity theory are now considered inadequate and are being replaced by linear or nonlinear elasto-plastic theories. (Good grief!)

The theoretical approach for evaluation of skin resistance is similar to that used to analyze the resistance to sliding of a rigid body in contact with the soil. Equations are available to calculate the point and skin resistances; however, the calculations require detailed knowledge of strength and deformation characteristics of the soil strata and also the variation of unit weight

and water content within those strata. For most structures, the cost of obtaining the information is prohibitive. In addition, it is normally preferable to estimate unit resistances directly from such field tests as the static cone penetration test, the standard penetration test or the pressuremeter test.

Bowles (1982) states it is highly probable that, in the usual range of working loads, skin resistance is the principal mechanism in all but the softest soils overlying solid strata (i.e., bearing doesn't get mobilized--rather the load is carried on the shaft).

Pile Installation Effects

The installation of piles affects the surrounding soil mass, and the extent to which installation changes the properties of the soil around the piles is a major concern.

Consequently, the method used to install a pile may have profound effects on its behavior under load. The standard methods for installing concrete, steel and timber piles include driven piles, bored or cast-in-place piles, driven and cast-in-place piles, and screw piles.

Driven piles will be discussed here, because that is what is used in small-craft harbors and because driven piles develop greatest uplift resistance--excluding bell-bottomed caissons, etc. The effects of driving are different in clays and in sands.

The effects of driving in clay deposits have been classified into four categories. The first is the influence on soil shear strength and pile capacity. The undrained strength of a clay is initially decreased considerably because of driving, but in time a significant amount of strength is regained. Driving piles into clay will cause a loss in undrained strength because of remolding at constant water content. Strength will increase with time as structural bonds are partially restored and excess pore water pressure dissipates. The remolding of the soil mass virtually stops two diameters from the face of the pile surface. The time it takes for strength to be regained is a function of the rate of pore pressure dissipation. Full or nearly full strength is regained in 100 to 1,000 hours.

The second category is the influence of pore pressures developed during driving. In the vicinity of the pile, very high excess pore pressures are developed--in some cases nearly 1.5 to 2.0 times the in situ vertical effective stress; at the tip, it might be 3.0 to 4.0 times. Some have found significant negative friction and downdrag because of reconsolidation of soil around the pile. Pore pressure decreases rapidly beyond two diameters from the pile face for normal clays and beyond four diameters for sensitive clays.

Third is the effect of the dissipation of excess pore pressure. A consolidation analysis provides a reasonable estimate of the rate of increase of load capacity (adhesion increase). Dissipation of excess pore pressures is assumed to occur radially, and a degree of consolidation of 100 percent (when the excess pore pressure has dissipated) occurs after 10 to 100 days. This estimate is used to determine the time between driving and load testing. (The estimate of the time for adhesion build-up is important in ice uplift

pressure resistance during the first winter for newly driven pilings.) The total and effective stresses adjacent to the pile just after driving may be related directly to the original undrained strength of the soil, and is essentially independent of the overconsolidation ratio (OCR). The final stress state after consolidation is similar to a K_o test, except that the radial stress is now the major principal stress.

Peck et al. (1974) say experience indicates that the driving resistance of friction piles in clay is likely to be low because of disturbances of the clay structure, whereas after driving the strength may increase markedly over a period of time. The gain in strength or "freeze" (also occasionally referred to as "fetch up" or "seize") may be caused partly by gradual reorientation of absorbed water molecules on clay particles (thixotropic processes) and partly by consolidation of the highly stressed clay immediately surrounding the piles.

As the freeze is likely to constitute the greatest part of the capacity of the pile, and as it is not related to the phenomena of stress transmission in the pile during driving, dynamic pile driving formulas or analyses based on the wave equation (to be discussed subsequently) are likely to give erroneous conception of the capacity. This conclusion has been amply demonstrated by experience.

Lastly is the effect of displacement caused by driving. Pile driving generally causes a heave of the clay surrounding the pile, followed by consolidation of the clay. It is a matter of a few inches, and both soil and pile heave can be estimated (though this is relatively unimportant in widely spaced marina piles).

When a pile is driven into sands, on the other hand, the soil is usually compacted by displacement and vibration, resulting in permanent rearrangement and some crushing of particles. Thus, in loose soils, the load capacity of a pile is increased as a result of the increase in relative density caused by the driving. The density decreases above the pile tip because of tensile strains equal to about half the compression strains below the tip. The zones affected are three to five diameters for straight and tapered shafts.

When groups of piles are driven in a loose sand, the soil around and between the piles becomes highly compacted. If the pile space is sufficiently close (less than six diameters), the ultimate load capacity of the group may be greater than the sum of the capacities of the individual piles. But if the sand is so dense that pile driving causes a loosening rather than a compaction of the soil, the group's efficiency may be less than 1. The standard penetration resistance (N) values can increase 1 to 2 times within a group of piles on 4- to 5-foot centers each way. Experience with vibratory compaction in fine uniform sands indicates an increase in relative density but no change in N-value, hence some ambiguity about this type of soil behavior exists.

Adding to Poulos and Davis' descriptions, Vesic states that when such piles are driven into saturated stiff clay, there are significant changes in the secondary structure (closing of fissures) extending to a distance of several diameters around the pile, with remolding and a complete loss of the effects of previous stress history in the immediate vicinity of the pile. If the surrounding soil is cohesionless silt, sand or partially saturated clay, pile driving may cause soil densification, which is most pronounced in the immediate vicinity of the pile shaft and gradually diminishes in intensity

over a zone extending from one to two pile diameters around the shaft. The driving procedure is also accompanied by increases in horizontal ground stress and changes in vertical stress in the pile vicinity, some or all of which can be lost by relaxation in creep-prone soils.

Bowles (1982) notes that increases in pile capacity over time as a result of consolidation processes may be marginal in very stiff and overconsolidated clays; in fact, pile capacity may decrease slightly with time as the high lateral pressures dissipate via creep. There are reports of increased load capacity for piles in sand, with the principal gain occurring in about a month. This strength increase cannot reasonably be attributed to dissipation of excess pore pressure, but rather may be due to local factors causing grain adhesion to the pile and dissipation of residual driving stresses.

A closed-end pipe or a full-section rectangular or square pile causes much larger lateral stress increase than an open-ended pile or a steel H-pile. In loose, cohesionless deposits, driven piles may cause considerably more soil densification than vibrated or jacked piles. In very dense, cohesionless strata, jetting with water can be used, at least part of the way, to remove an equivalent volume of soil and to ease driving to the desired depths. Piles so formed would be classified as partially or fully nondisplaced. Their placement causes little or no change in lateral ground stress, so such piles develop less shaft friction than displacement piles of the same size and shape.

In the case of piles with developed profiles, such as open piles or H-piles, the unit bearing resistance and shaft skin resistance are usually expressed in terms of fictitious areas, defined as areas contained within the outer perimeter of the profile. This is associated with observations on the formation of a soil plug within the interior of the pipe or between the outer flanges of the H-profile. In the case of cohesionless soils, where the ratio of base resistance to developed shaft skin resistance may be high, it is advisable to verify that the assumed plug can transmit the reaction from pile point to the main body of the pile by friction. (There are no simple, clear-cut ways to analyze these soil plugs.)

Occasionally, the large end of a pile is driven butt down--as in the case of a soft soil overlying a firm stratum, for example. The soft soil will flow back against the sides of the shaft and the butt will rest on the firm stratum, giving increased bearing area. Piles driven butt down are a reverse-taper type of pile and conceptually should develop larger uplift resistances from the soil mass. Whether this happens or not has yet to be demonstrated and documented. A second perceived benefit of butt-down driving is that the ice tends to slip on the reverse taper as the ice sheet rises, but this is highly unlikely unless the pile is wrapped or coated successfully because the ice fails ice-to-ice in the sheet and never "sees" the developed pile surface.

In sum, the effects of driving piles in sands and clays are many and not well quantified, and they are difficult to deal with specifically in a design sense. As for installing bored piles rather than driven piles, the effects in clay have been studied largely in relation to the adhesion between the pile and the soil. The adhesion has been found to be less than the undrained cohesion prior to installation, mainly because of a softening of the clay immediately adjacent to the soil surface. Drilling fluids (muds) and casings also cause problems in developing adhesion. There is relatively little qualitative information on the effects of installing bored piles in sands.

TABLE 5.8 Customary Range of Working Load on Driven Piles (after Peck et al. 1974)

Type of Pile	Working Load (tons)
Timber (8" tip diameter)	10–30
Concrete (10" diameter)	20–60
Steel pile, concrete filled	
10 3/4" x 0.188"	30–50
10 3/4" x 0.250"	40–70
12 3/4" x 0.250"	50–80
14" x 0.312"	60–90
Steel H-section	
HP 10 x 42	50–75
HP 12 x 53	50–95
HP 14 x 89	100–160
HP 14 x 117	150–200

Working Loads and Ultimate Loads

Table 5.8 lists some customary ranges for working loads on piles. Working loads are something less than ultimate capacities and may be determined by using factors of safety (such as 2 to 6) on ultimate loads, through settlement criteria and other empirical methods. Lengths to achieve these working loads vary with soil deposit types, pile column-strengths and other factors.

The two usual approaches to calculating the ultimate load capacity of piles are the "static" approach, which uses the normal soil mechanics methods to calculate the load capacity from measured soil properties, and the "dynamic" approach, which estimates the load capacity of driven piles from analysis of pile-driving data.

Ultimate Load Capacity--Static Approach

The net ultimate load capacity (Q_u) of a single pile generally is accepted to be equal to the sum of the ultimate shaft resistance and base resistance, less the weight of the pile:

$$Q_u = Q_p + Q_s - W \tag{5.9}$$

where: Q_u = net ultimate load capacity
Q_s = ultimate shaft resistance
Q_p = ultimate base resistance
W = weight of pile

It is an implicit assumption of Equation 5.9 that the shaft and base resistance are not interdependent. This assumption cannot be strictly correct, but it is correct enough for practical purposes.

Vesic indicates that modern research on pile behavior has established that full mobilization of skin resistance requires a relative displacement between the pile shaft and surrounding soil of 0.25 to 0.40 inches, regardless of pile size and length. At the same time, mobilization of ultimate point resistance of a pile requires a displacement of approximately 10 percent of the pile-tip diameter for driven piles and as much as 30 percent of the pile-tip diameter for bored piles.

But assume Equation 5.9 is acceptable. If we concentrate on ultimate uplift resistance, Q_p is discarded, and W is positive rather than negative or dropped from consideration. As mentioned earlier, I will focus on developing uplift resistance, not vertical pile capacity, for gravity loads.

Before discarding Q_p, I should note that the ultimate base or point resistance is usually evaluated from bearing capacity theory; that is:

$$Q_p = A_p[cN_c + qN_q + (1/2)\gamma BN_\gamma] \qquad (5.10)$$

where: A_p = area of base

(The remaining factors are as defined in Equation 5.1)

Equations 5.4a, 5.4b and 5.4c give approximate values for bearing capacity for sand and clay deposits. In evaluating the bearing capacity equation, the parameters are either undrained or drained, depending on whether short-term or long-term ultimate capacity is to be computed.

The ultimate shaft resistance term in Equation 5.9 (Q_s) is of major interest. Q_s can be evaluated by integrating the pile-soil shear strength over the surface area of the shaft. But just as a straight line is used to represent the Mohr-Coulomb failure envelope (because engineers don't like equations higher than the first order), an integral sign won't be used to determine Q_s. Assume that if the unit skin resistance of the pile shaft (f_s) is known, a pile length and appropriate area for the shaft can be factored to find $Q_s = L \times A \times f_s$. The f_s term is the parameter to be evaluated, but the shaft area can also be a problem for a shape like an H-pile.

For a steel H-pile, two modes of failure of the shaft are possible: (1) the development of the limiting pile-soil shear strength along the entire surface area of the pile (the "painted area"), and (2) the development of the limiting pile-soil shear strength along the outer parts of the flanges, plus the development of the full shear strength of the soil along the plane joining the tips of the flanges (i.e., the soil within the outer boundaries of the pile effectively forms part of the pile shaft). Since which mode of failure will apply is unknown, it is best to use the lesser of the two values.

Unit Skin Resistance

As previously stated, the theoretical approach for evaluation of unit skin resistance (f_s) generally is similar to that used to analyze the resistance

to sliding of a rigid body in contact with soil. It is assumed that f_s consists of two parts: "adhesion" (c_a), which should be independent of normal stress (σ_h) acting on the foundation shaft, and "friction," which should be proportional to that normal stress. Thus, in any particular stratum in contact with the foundation shaft:

$$f_s = c_a + \sigma_h \tan \delta \qquad (5.11)$$

where: f_s = unit skin resistance
c_a = soil adhesion
σ_h = normal stress
δ = angle of pile material friction, or skin friction

In this equation, tan δ represents the coefficient of friction between the soil and the shaft, which--according to experience with piles of normal roughness, Vesic says--can be taken as equal to tan ϕ', the coefficient of friction of the remolded soil in terms of effective stresses.

Kezdi (1975) gives surface friction angles and coefficients for various pile materials and soil types. Some of them are listed in Table 5.9.

Vesic states that the pile-soil adhesion (c_a) is normally small and for design purposes can be neglected. This will be true for a granular deposit, but for a cohesive deposit, the skin friction (f_s) will be related to the soil cohesion (c), which, at least empirically, is related to soil adhesion (c_a) to a pile material.

The normal stress on the pile shaft is conventionally related to the effective vertical stress--at the corresponding level prior to placement of the pile--by a coefficient of skin pressure (K_s), defined as the normal stress on the shaft divided by the effective vertical stress, written as:

$$f_s = K_s \tan \phi' \sigma_v' \qquad (5.12)$$

TABLE 5.9 Surface Friction Angles and Coefficients (after Kezdi 1975)

Soil Type	Precast Concrete		Steel	
	δ°	tan δ	δ°	tan δ
Sandy gravel (clean)	30	0.58	28	0.55
Damp sand	31	0.60	28	0.55
Saturated sand	30	0.58	26	0.49
Coarse silt, wet	23	0.43	22	0.40
Silt, wet	21	0.36	20	0.36

Please watch out here--I am talking here about the vertical effective stress and the lateral stress perpendicular (normal) to the vertical pile shaft. The coefficient K_s depends mainly on the initial ground stress conditions and the method of placement of the pile, but it is also affected by pile shape (particularly taper) and length. K_s can be equal to or smaller than K_o and can even be as large as K_p.

For piles driven into normally consolidated soft-to-firm clays, K_s is equal to or slightly larger than K_o. The skin resistance may be initially low because of the existence of pore pressure set up by pile driving and a corresponding reduction in effective overburden stress. However, as the pore pressures dissipate and as the effective overburden stress approaches its initial value, the skin resistance of many clays may, after a sufficient period, become approximately equal to their undrained shear strength. This long-established fact has led researchers into comparing the skin resistance with the undrained shear strength for all clays.

In sum, concentrate on skin friction capacity rather than point bearing pile capacity. So far, it is tacitly assumed that skin friction for down-loads is the same as for up-loads. The skin friction somehow varies with cohesion, adhesion, effective vertical stress, the coefficients of friction for soil and other materials, undrained and drained parameters, lateral earth pressure coefficients, and pile characteristics such as method of placement, shape and length. It is complicated. Later sections will review some of the literature and research results and controversies and present my recommendations on how to evaluate and design for uplift. For now, for a feel for the skin friction (f_s), I recommend you think of it in the approximate terms I have previously given for marinas (see Wortley 1982).

Preliminary estimates of required pile penetration can be made assuming skin friction equal to about a third of the effective vertical stress (up to depths of about 20 pile diameters, beyond which the skin friction may not increase with depth) for granular deposits, and about equal to the undrained shear strength for cohesive deposits.

Ultimate Load Capacity--Dynamic Approach

The second usual approach to calculating the ultimate load capacity of piles, the dynamic approach, is the most frequently used method of estimating the load capacity of driven piles. The method uses driving formulas or dynamic formulas. The formulas relate ultimate load capacity to pile "set" (the vertical movement per blow of the driving hammer) and assume that the driving resistance is equal to the load capacity of the pile under static loading. The formulas are based on an idealized representation of the action of the hammer on the pile in the last stages of its embedment.

The primary objectives of using a pile-driving formula usually are either to establish a safe working load for a pile by using the driving record of the pile, or to determine the driving requirements for a required working load. The working load usually is determined by applying a suitable safety factor to the ultimate load calculated by the formula. Pile driving formulas take no account of the nature of the soil, so safety factors must vary.

You've probably heard of the "wave equation"--which pertains to soils, not harbor waves. The wave equation method examines the transmission of

compression waves down the pile, rather than assuming that a force is generated instantly throughout the pile, and it obtains a better relationship between ultimate pile-load and pile-set than can be obtained from a simple driving formula. This relationship allows an assessment to be made of the driveability of a pile with a particular set of equipment. Moreover, this approach also enables a rational analysis to be made of the stresses in the pile during driving and can therefore be useful in the structural design of the pile.

The judicious selection of a compatible hammer-pile-soil system may optimize driveability and minimize installation cost. It is in pursuing this aim that the wave-equation analysis probably enjoys its greatest success.

Pile Settlement

The analysis and design of piles includes traditional methods of settlement analyses on assumptions of stress distributions or on empirical correlations. This dimension won't be brought into the analyses here, as it is not a problem for marinas.

Lateral Loads and Deflections

In the design of pile foundations for lateral loads, two criteria must be satisfied: an adequate factor of safety against ultimate failure, and an acceptable definition of working loads. As in other fields of soil mechanics, these two criteria generally are treated separately, and the design is arranged to provide the required safety margins independently.

In many cases, the ultimate load will be reached at very large deflections, especially in the case of relatively flexible piles. In such cases, it may be desirable to carry out a complete elastoplastic analysis, but in marina construction this isn't really necessary. For semirigid piles, the methods subsequently described are assumed to be generally applicable, even though the pile is rather flexible.

The methods for estimating the ultimate lateral resistance of relatively slender vertical floating (not bottom-bearing) piles with negligible base resistance will be used. The methods consider the statics of a pile subjected to a horizontal force, a moment, or both.

Satisfactory theoretical solutions for piles under lateral loads with significant base resistances have not yet been obtained. This is important, as some harbors have soft sediments underlain by competent strata into which piles can be driven--and there is no analytic model, other than to assume the overlying deposit has no effect. In designing pile foundations to resist lateral loads, the criterion for design in many cases is not the ultimate lateral capacity of the piles but rather the maximum deflection of the piles. For marina ice loads, the opposite most often would be the case (i.e., failure of the pile material and the soil mass is more of a concern than deflections).

In the past, design methods for determining lateral load deflections have frequently made use of empirical information, such as full-scale load tests and model studies. More recently, theoretical approaches for predicting

lateral movements have been developed extensively. However, subgrade-reaction theory and elastic-continuum theory to analyze pile load-deflections will not be discussed here. Such analyses seem unwarranted, except in unusual cases.

This completes my review of geotechnical and ice engineering. Other sections of this manual will build on the principles and concepts covered here.

Part Two:
PRELIMINARY DESIGN



6. Small-Craft Harbor Site Characterization

This chapter discusses some of the parameters necessary to characterize a small-craft harbor site. An understanding of these parameters is needed for satisfactory design. Other parameters also may be necessary, but they are so site-specific that they are not included here.

A site parameter, or a parameter, is defined as "a quantity that is constant in a particular calculation (mathematical sense) or case, but varies in other cases." Characterization of a site involves consideration of the soil, the water, the ice, the climate, etc. These parameters are not literally constants but descriptors, which vary in time and within defined ranges--for example, the ranges of thickness of ice, air and water temperatures, etc.

In Chapter 1, a small-craft harbor was referred to as a complex situation of ice-water-soil-structure interaction. This is the interaction to be understood: characterize first, provide for next; start from the "bottom" with the soil and work "up."

SITE GEOLOGY AND HISTORY

Most of this discussion is taken from Peck et al. (1974), with some from Martin (1965). Though rather long, it is relevant to an understanding of Great Lakes geology.

A program of subsurface exploration (discussed in the next section) for any foundation project must be adequate to disclose the essential character of the deposit, particularly its possible variations from point to point. Economy and the limitations of time dictate that no greater expenditure than necessary should be made to produce the desired results. This end cannot be achieved if the engineer does not have at least a rudimentary knowledge of the anatomy of various kinds of deposits. Such knowledge will assist the engineer in interpreting information as it is obtained from the field and laboratory and in recognizing the stage at which further information would not be worth the added cost.

Probably the most variable deposits are those asociated with glaciation. This is particularly relevant because the basins of the Great Lakes are the result of glacial erosion. The area was covered by glaciers, and the events of the glacial epoch with respect to the work of the foundation engineer are of extraordinary significance. These events illustrate that the engineer dares not assume uniformity of the subsurface conditions and must learn the character of the deposit at each site to be able to forecast the most unfavorable conditions that may be encountered.

From a geological standpoint, soils can be divided into two major groups: "transported" and "residual." Transported soils no longer cover the rock material from which they were derived. They may be further classified according to the mode of transportation and deposition: alluvial soils, transported by running water; lacustrine soils, deposited in quiet lakes;

marine soils, deposited in sea water; aeolian soils, transported by wind; colluvial soils, deposited primarily through the action of landsides and slopewash; and last but not least, glacial soils. Residual soils have been developed from the parent rock over which they now lie. Deep deposits are common in the southeastern United States, for example, but are rare in the northern half of the U.S. and in Canada because the continental glaciers removed most of the products of weathering that had formed on the bedrock.

In a very general way, soils tend to be arranged in profiles, or systems of layers. The most significant of these are profiles of weathering and profiles of depositon. In many instances, one of the former is superimposed on one of the latter, and a rather complex system of soil layers may be found near the surface.

Deposits Associated with Glaciation

The great continental glaciers covered much of the land surface north of the 40th parallel. The ice excavated, transported and redeposited loose rocks and soils. The materials laid down by glaciers are collectively known as "drift." Those deposited directly out of the ice are called "till." The meltwater flowing away from the ice also carried debris and deposited it in broad sheets known as "outwash." The concentration of meltwater into torrential streams of temperature-dependent variable rates of flow gave rise to "glaciofluvial deposits." In some instances, the meltwater was dammed between high ground and the glacier itself, forming lakes in which sediments known as "glacial lake deposits" were laid down.

Carrying drift, the ice of an active glacier continually flows toward its outer edges. Melting takes place near the edges, and the drift is concentrated at the bottom of the ice, where part of it becomes fixed to the frozen ground. The ground-fixed drift constitutes "ground moraine," consisting primarily of till of erratic composition. A small amount of ground moraine is deposited as an ice sheet grows, and a larger amount as it shrinks. Where repeated growth and shrinkage of a glacier occurs, several distinct sheets of till are laid down. Figure 6.1 represents a cross section of the earth beneath downtown Chicago, which has at least three successive ground moraines.

If the edge of the ice remains stationary for at least a few years, the drift accumulates in a ridge at the face of the glacier. These ridges are known as "terminal" or "end" moraines. They are long and narrow in plan, and may be tens of feet thick. They consist largely of till, but they may be stratified in places once occupied by pools of meltwater and may contain deposits of outwash of irregular shapes.

Glacial tills vary widely in texture, plasticity and engineering properties. Texture ranges from coarse to fine, and these tills are frequently well-graded even though the clay-size fraction varies widely. The strength of tills may vary both vertically and horizontally, as shown in the generalized cross section in Figure 6.1.

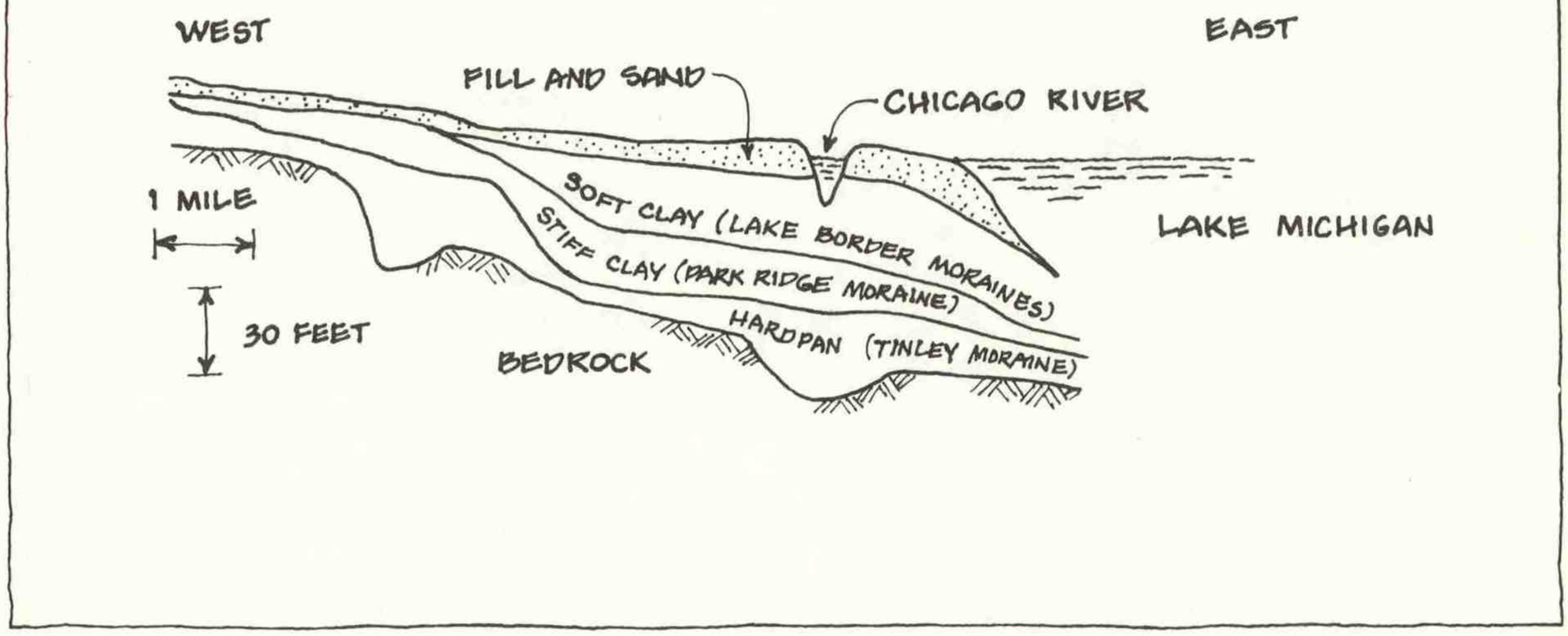


FIGURE 6.1: Simplified Cross Section of Glacial Deposits beneath Lake Street in Chicago: Successive Ground Moraines. (after Peck et al. 1974)

All types of moraines are likely to contain some waterlain clays and silts deposited in temporary ponds. They also may contain uniform sands and gravels laid down in channels and tubes in the ice. Some moraines are composed of clay till having exceptionally uniform water content, whereas others may exhibit extremely erratic variations. Morainic areas are likely to be poorly drained, especially if the till was deposited during one of the more recent glacial advances. In poorly drained pockets, deep beds of peat are often encountered.

During the warm seasons, tremendous quantities of water flowed from the faces of the continental glaciers, carrying coarse material short distances and transporting sands, silts and clays greater distances. Temporary channels quickly became choked with debris and new ones were created. The resulting glaciofluvial deposits--especially if formed close to the glaciers--consist of lenses of coarse-to-fine materials, some loose and some dense. They are among the most coarse-grained of sediments and vary widely in resistance to penetration and in grain size.

Peat and marsh deposits are even more common on outwash plains than on till plains. Because of their high compressibility, they ordinarily are avoided or excavated or consolidated (e.g., by preloading).

Many harbors on the Great Lakes exist on estuary-like mouths of rivers. Along the west shore of Lake Michigan, for example, Wisconsin has deepened the mouths of rivers at Milwaukee, Racine, Kenosha, Sheboygan, Kewaunee, Manitowoc, Port Washington, Two Rivers and Algoma. These lake ports could not otherwise exist because of the regular character of the shoreline.

Heterogeneous soil conditions should be anticipated. (Riverine marina locations may also present special problems caused by man's wastes, such as slabs and sawdust from lumber mills.)

In contrast to the glaciofluvial deposits, those deposits laid down in the relatively quiet waters of glacial lakes show a high degree of uniformity. These deposits are the glacial lake or marine deposits. However, many are laminated, or "varved." As meltwater flowed into the basins, the coarser fraction was dropped near shore, whereas the finer sediments were carried into the open water. During warm periods, both silt and clay settled to the bottom. When melting and inflow ceased during cold periods, the finer clay fraction still in suspension continued to settle. Ice covers on these lakes stopped wave action and aided clay deposition. Banded deposits were formed. Where the bottom was shallow enough to be influenced by currents, the details of laminations sometimes became very intricate and nonuniform.

Glacial lake deposits are common around the Great Lakes and smaller inland lakes in the northeastern U.S. and southern Canada. If never exposed to desiccation (thorough drying), they are likely to be soft, compressible and sometimes quite sensitive. Since the lacustrine deposits are often associated with morained or outwash deposits of fairly high bearing capacity, their low strength has sometimes been overlooked with disastrous results.

Where glacial meltwaters flowed into marine embayments, the saline waters tended to flocculate the silts and clays so that they settled simultaneously and varves were not formed. Many of these deposits were uplifted with respect to sea level because of the isostatic rise associated with the removal of the weight of glacial ice (isostatic having to do with isostasy, the equilibrium of the earth's crust due to movement of the material below the surface).

Subsequently, the saltwater originally in the pores of the soil was gradually replaced by freshwater from rainfall. The physiochemical changes associated with the leaching resulted in the development of unusually high sensitivities. Such quick clays occasionally liquefy and flow on very gentle slopes. Canadian Leda clay is an example, and such quick clays can be found along the St. Lawrence River and its tributaries. Quick clays are also found along some of the major rivers and fjords in Scandinavia.

If you're wondering why I bring up the St. Lawrence River and Scandinavia, the reasons are that, first, the St. Lawrence River is an extension of the Great Lakes, and second, I have made observations on winter ice conditions both there and in Scandinavia. And if you're wondering how geologists and hydrogeologists know glacial processes so well, it is because these processes can be observed today in the Yukon and Northwest Territories. Some glaciers in these areas are wasting with a melt lasting about a millennium (Driscoll 1980).

For various reasons, the levels of the glacial lakes fluctuated widely. When the water level was low, tributary streams eroded valleys in the surrounding lands. As the lakes slowly rose, the mouths of the streams flooded and became filled with sediments, which were often mixed with the remains of plant life.

The entire area may then have been covered by lake sediments or even by till in later glacial advances, and the channels, with their soft filling, may have been completely buried. Such buried channels are very common near the Great

Lakes. They also are found on the seaboard, because periodically during the glacial epoch the level of the ocean was probably up to 300 feet lower than today. A rational interpretation of test borings in the vicinity of buried valleys requires at least a rudimentary knowledge of the geological history of the region.

Windblown Deposits

Closely associated with glacial deposits--especially in the vicinity of major glacial drainageways and outwash areas--are deposits of sand and silt sorted by wind. The sweep of the wind across large sand-covered areas, such as outwash plains, moves the sand and silt-sized particles but leaves the gravel behind. The sand grains are rolled over each other or bounced short distances into the air, and piled up to form dunes, whereas the silt-sized grains are blown away.

"Dune sands" are among the most homogeneous of natural formations. The process of selection by wind sorts the sand into assemblages of very uniform grain size. Windblown silt deposits, or "loess," are usually quite uniform but possess cohesive strengths that, because of weathering and clay coatings on the silt particles, can vary considerably from place to place.

Organic and Shore Deposits

Accumulations of highly organic material may be found in association with almost any type of geological deposit when environmental conditions are appropriate. They are most commonly formed in depressional areas where the water table is permanently at or above the original ground surface and where climatic conditions are favorable to the growth of aquatic vegetation. Consequently, they are frequently found in glaciated regions, coastal areas and river valleys in the temperate to polar regions.

Organic accumulations like "peat," "muck," "muskeg" and "marsh deposits" may vary in depth from a few inches to several tens of feet. Their natural water contents are usually well over 100 percent. They are all highly compressible.

The action of waves and shore currents in lakes and oceans builds up beach and shore deposits primarily of sand and gravel. These deposits may be of relatively uniform grain size and of moderate to high relative density. On the other hand, if the shoreline has fluctuated due to changes in the water level, the deposits of sand may alternate erratically with organic silts and peats. Such formations are known as composite shore deposits. The one illustrated in Figure 6.2 is located near the mouth of the Milwaukee River in Wisconsin.

SUBSURFACE INVESTIGATION

The main message of this section is: Do a good job with subsurface investigation. Don't skimp. It's not the place.

How much field and laboratory work should be done? It depends. It depends on the site geology and history, it depends on the project and likely types of foundations. It doesn't depend on a preconceived, set budget. If the full

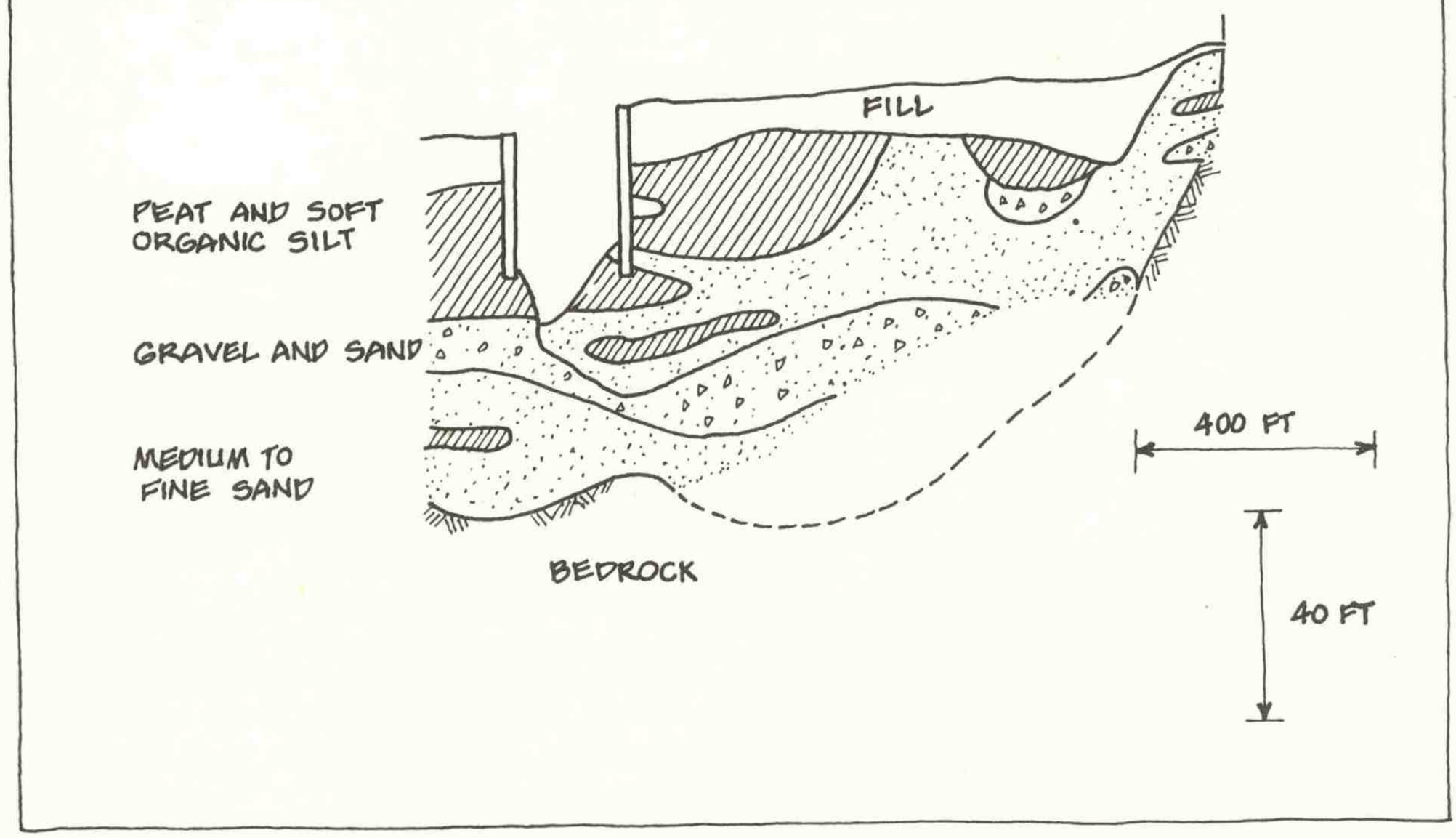


FIGURE 6.2: Cross Section of Composite Shore Deposit Near Mouth of the Milwaukee River, Wisconsin. (after Peck et al. 1975)

extent of the subsurface investigation required was known in advance, it wouldn't be necessary. Cost is obviously a factor, but the geotechnical investigation is so important to doing the project right that it is not the place to skimp.

What's required is a flexibly structured program carried out by skilled drillers and soil testers working under the direction of an experienced soils engineer. Any less, on most projects, is inviting trouble and higher total project costs. The question of what constitutes a subsurface investigation is covered well by Peck et al. (1974).

The preceding section demonstrated that very few natural deposits are evên approximately uniform and many are quite variable. In a variable deposit, obviously, no program of subsurface exploration can lead to more than a rough idea of the average values for the physical properties of the subsurface material and the probable variations from these values.

The nature of the deposit is an important factor in determining the method of soil exploration that will yield the greatest amount of useful information. For example, if the foundation of an important structure is to be established above a fairly homogeneous layer of clay, a considerable amount of testing of undisturbed samples may be justified, because the test results permit a relatively accurate forecast of both the amount and the rate of settlement. Or, if piles are to be driven, tests could establish the strength parameters.

On the other hand, if the same structure is to be located above a deposit composed of pockets and lenses of sand, silt and clay, a comprehensive testing program would not be justified, because it would provide little more information than could be obtained by merely determining the index properties of representative samples. Much more useful information could be obtained at less cost by making an adequate number of penetrometer measurements that would disclose the pattern of the various soft and stiff elements in the subsoil.

Select the magnitude and character of the exploratory program with consideration of the importance of the project under construction. If the job involves only a small expenditure, extensive programs of soil exploration cannot be justified economically. It is cheaper to take advantage of whatever information may already be available and to use a liberal factor of safety in design. (One isn't always "out of the woods" with this approach--you may wind up with an unsatisfactory project for the want of satisfactory subsurface information.)

Develop a program of soil exploration step by step as information accumulates. With this procedure, the maximum amount of information is obtained for a given expenditure, and the program can be terminated as soon as adequate data have been collected. In short, no definite rules can be established for a soil exploratory program.

Preliminary Exploration

Precede the subsurface exploration program with a fact-finding survey. In such a survey, the exploration engineer prepares a digest of all available information on soil conditions near the site and on the behavior of structures in the vicinity that are similar to what is planned.

The preliminary exploration procedure is selected on the basis of the information obtained from the fact-finding survey. Most soil deposits, however, can be appropriately explored by means of a split-barrel sampler ("split-spoon") and standard penetration tests carried out in holes made by augers, rotary drills or wash-boring tools.

Other methods of exploration are not usually considered in the preliminary phase unless it is known that the underlying material consists of bedrock, or of very soft clays, silts or highly organic materials. Moreover, for many projects, no further subsurface exploration is necessary. This is likely to be the case if the loads on the subsoil will be small and a large factor of safety can be used without excessive cost, if the structure can be founded on rock or strata of high bearing capacity, or if an ordinary structure is to be built in an area where much practical experience has been summarized in the form of reliable empirical rules.

Detailed Exploration

If the preliminary exploratory program does not provide sufficient information for design or construction, further investigations are required. Frequently, the properties of fairly uniform deposits of soft clay and plastic silt can be investigated most economically by field vane tests or by obtaining continuous

samples in 2- or 3-inch thin-walled tubes ("shelby tubes") and performing appropriate laboratory tests. Erratic deposits of soft silt and clay can be examined by means of penetration tests combined with enough tube borings to permit interpretation of the penetrometer data. Standard penetration tests ("N-values" or "blows") or dynamic cone penetrometer tests are appropriate for sands.

Depth and Number of Borings

The depth and number of borings are functions of the size of the project, the planned types of foundations, the variability of the site geology and other factors. If you plan to use pilings, certainly the borings have to extend to depths somewhat greater than the estimated pile lengths. If you plan to use bottom-resting gravity structures, the borings have to extend to depths two to three times the estimated widths of the structures. If soft compressible strata are being encountered, the borings should extend past these strata to firm materials (unless you're somewhere like New Orleans, where the fluvial deposits are hundreds of feet thick!).

If borings encounter rock and the conditions are such that the structure may be founded in rock or that rock may be a problem in driving piles, obtain cores for a depth of 5 to 10 feet to make sure that sound rock has been reached rather than a boulder or a piece of detached rock. If borings encounter rocks that are not cored and the boring is moved over and started again, it is important to mention this fact in presenting the results--floating boulders in soil deposits cause many problems in the installation of pilings.

The borings must be performed "over water"--or "over ice" if done in the winter (which is a good idea if you can plan ahead and have a sound ice sheet). The borings cannot be done on shore because it cannot be assumed that they will depict soil conditions in the harbor. As explained previously, glacial deposits are too heterogeneous.

How many borings? One boring for every 100,000 square feet is a good number. This works out to a grid pattern of about 300 feet on a side. Something on this order would be reasonable for planning a program.

Presentation of Results and Recommendations

The soils engineer should prepare a foundation engineering report that presents the results of all field and laboratory tests in a format useful and readily comprehensible to the marina designer and owner. Subsequently, give prospective contractors the results. The contractors will be doing the work, and they need to know whatever you know.

Besides clearly presenting the data, the soils engineer's report should substantiate recommendations on the foundation types and their predicted ranges of performance and costs. If the soils engineer is not experienced in the design of harbor structures, especially for ice conditions, give him a copy of this manual.

No matter how complete the program of soil exploration and testing may be, a large margin of uncertainty always remains regarding the exact nature of the

subsurface conditions at a given site. This fact is of outstanding practical importance. The engineer often must wait to obtain the final data on soil conditions until what happens in the field can be observed. Soil tests performed on a few samples from an erratic deposit do not provide a satisfactory basis for design, because the engineer is interested in the behavior of the deposit as a whole rather than of a few specimens taken from it.

LAKE LEVELS, WIND SETUPS, SEICHES AND CURRENTS, AND WATER TEMPERATURES

As mentioned earlier, one of the most significant and damaging ice forces results from changes in water levels. These changes cause the ice sheet to move up and down, tearing and pulling harbor structures. This section addresses these water motions and water temperatures under ice covers.

Large lakes have insignificant tidal variations, but they are subject to seasonal and annual hydrologic changes in water level, as well as changes caused by wind setup, barometric pressure variations and seiches (defined later). Also, some lakes are subject to occasional water level changes caused by regulatory control works (Coastal Engineering Research Center 1977). In the Great Lakes, water level control is possible at Sault Ste. Marie, at Chicago and at Niagara.

The question of Great Lakes tides is interesting and apparently not fully resolved. Some areas of the Great Lakes have small tides, on the order of two, perhaps three inches (Martin 1965; Hutchison 1957; Mortimer and Lee 1976), but whatever the exact magnitude, the tidal effect is secondary in importance to other effects.

Lake Levels

Water surface elevations of the Great Lakes vary irregularly from year to year. Each year, however, the water surfaces consistently fall to their lowest stages during the winter and rise to their highest stages during the summer. Nearly all precipitation in the watershed areas during the winter is snow or rainfall transformed to ice. When the temperature begins to rise, there is substantial runoff--thus the higher stages in the summer.

Quarterly extreme high and low lake levels are indicated in Table 6.1. Note several things in this table. Average quarterly lake levels are not given. Though they are about the average of the high and low levels, average lake levels are not realistic design criteria for marina structures. In establishing elevations for fixed-height or adjustable variable-height structures, the designer must select levels that will be functional during the boating season, and he must also consider winter ice levels and the effects of both high ice and low ice. Note that the range on all Great Lakes is four to five feet. When ice is high, the horizontally spanned dock members can be engulfed; when ice is low and if, for example, pilings are coated or wrapped up to reduce ice adhesion, unprotected lower portions of these pilings can be gripped, etc.

Also note in Table 6.1 that water level variations in Lake Superior are not as great as in the other lakes. Lake Superior is controlled. In establishing deck elevations for fixed head piers, the Michigan Waterways Commission uses 5.5 feet above Chart Datum; for Lake Superior, 5 feet is customarily used.

TABLE 6.1: Quarterly[a] High and Low Levels from Chart Datum[b] for the Great Lakes (after Corps of Engineers[c])

Extreme Level	Seasonal Level Changes (feet)			
	Winter	Spring	Summer	Fall
	Lake Superior (Chart Datum 600.0)			
High	+1.2	+1.4	+2.0	+1.8
Low	-1.5	-1.5	-0.8	-0.7
	Lakes Michigan and Huron (Chart Datum 576.8)			
High	+3.2	+3.8	+4.1	+3.4
Low	-1.4	-1.2	-0.8	-1.2
	Lake St. Clair (Chart Datum 571.7)			
High	+3.9	+4.4	+4.2	+3.6
Low	-1.5	-0.2	-0.1	-0.6
	Lake Erie (Chart Datum 568.6)			
High	+4.1	+4.8	+4.4	+3.7
Low	-1.0	-0.2	-0.2	-0.8
	Lake Ontario (Chart Datum 242.8)			
High	+3.7	+5.1	+4.6	+3.5
Low	-1.0	-0.2	-0.4	-1.1

[a]Quarters: Winter is January-March, etc.
[b]Chart Datum is feet above mean water level at Father Point, Quebec (International Great Lakes Datum 1955).
[c]Detroit District, U.S. Army Corps of Engineers, Attn: NCEED-L, P.O. Box 1027, Detroit, MI 48231

Complete, monthly hydrological data covering many years is available from the Great Lakes Environmental Research Laboratory, 2300 Washtenaw Ave., Ann Arbor MI 48104. (See reference, GLERL 1983).

Wind Setups

Wind stresses affect the Great Lakes and raise water levels at one end and lower them at the other. Sudden changes of water levels can vary between a few inches to many feet (Table 6.2).

Lake Erie, shallowest of the Great Lakes, is subject to greater wind-induced surface fluctuations (wind setup) than any other lake. Table 6.2 reports sudden short-term maximums recorded at several gage sites. Even though these high and low water elevations won't stay long, usually only hours, significant water level changes do occur, and they must be considered in the design of structures. Figure 6.3 shows a Lake Erie dock after high water receded.

Seiches and Currents

An interesting, unusual and somewhat hard-to-define phenomenon of the Great Lakes is the "seiche." The dictionary defines it as:

> seiche (saysh), n. An occasional, rhythmical movement from side to side of the water of a lake, with fluctuation of water level, thought to be caused by sudden local variations in atmospheric pressure (derived or taken from Swiss-French "seiche")...

Schwab (1978b) states that the principal cause of local water level elevation or depression on the open lakes and large bays is the effect of atmospheric disturbances on the water mass. High winds and barometric pressure gradients act as external forces, deforming the water surface. When the force is removed, the water mass dissipates the potential energy of the deformed surface by damped oscillation. The deformation of the water surface is known as a storm surge or wind tide; the subsequent oscillation is called a seiche.

TABLE 6.2: Sudden, Short-Term Maximum Deviation[a] from Chart Datum (after Coastal Engineering Research Center 1977)

Lake and Gage Location	Rise (feet)	Fall (feet)
Lake Superior at Marquette	4.1	2.5
Lake Michigan at Calumet	6.4	3.3
Lake Huron at Harbor Beach	5.2	2.6
Lake Erie at Buffalo	10.5	4.4
Lake Erie at Toledo	8.1	7.1
Lake Ontario at Oswego	6.2	1.8

a Deviations presumably are from wind setup but may also include other effects, such as seiches. More detailed data on wind setup may be obtained from the Great Lakes Environmental Research Laboratory, 2300 Washtenaw Ave., Ann Arbor MI 48104.

Several other rather common definitions are that a seiche is a standing wave oscillation of an enclosed or semi-enclosed water body that continues, pendulum fashion, after the cessation of the originating force. It is a short-term rise and fall of the water level caused by either persistent strong winds piling up the water at one end of a basin, or changes in barometric pressure over the lake, and sometimes by a combination of both. The period of a seiche is a few minutes in a bay or harbor, and about 10 hours for a Great Lake. (A seiche can be easily observed in a marina in the summertime, even on a very calm day, by watching for a drop in the water surface with respect to some fixed object like a sloping launching ramp or a piling.)

In a chapter on hydromechanics, Hutchison (1957) states that the word "seiche" is used to express a stationary oscillation of a lake or a large independent part of a lake. If a wind that has piled up water at one end of a lake suddenly dies down, a current will flow from leeward to windward, momentarily restoring the lake to its former level. The current will not have lost its energy when the surface is level, however, but will continue, piling water at the former windward end. This will cause a new current in the direction of the original wind drift. As gradient currents, these currents are independent of depth except near the bottom, where the stress on the basin will gradually slow the movement. The current system thus constitutes an oscillation about a nodal line determined by the shape of the basin. As with other oscillating



FIGURE 6.3: Lake Erie Marina Covered with Ice Left by Receding Water Setup.

systems, there is a tendency for harmonics to form. The simple case described above is a uninodal seiche, but seiches that are binodal, trinodal, etc., are well known. In several cases, the higher harmonics--certainly up to the octinodal--have been recorded.

So a seiche is a slosh--and a rather complicated one at that. In a master's thesis on the jacking of marina piles by ice in the Great Lakes, Roblee (1983) analyzed data from 33 Great Lakes harbors. A site characteristic studied (for possible correlation with pile lifting damage) was seiche action at specific harbors. The following examination of lake level flucuations is based on Roblee's analyses and Schwab (1978a and 1978b), Schwab and Rao (1977), Rao and Schwab (1976), Rao et al. (1976), and Mortimer and Fee (1976).

Roblee states that the evaluation of the energy associated with various lake oscillations at a given site involves the painstaking process of reducing large quantities of water level data into their frequency components. This task has been addressed by members of the Great Lakes Environmental Research Laboratory, who authored some of the papers cited above. Their publications examine the first several modes of basin oscillation for each of the Great Lakes through a numerical modeling procedure supported by actual field data. The results are presented in nondimensional relative terms by means of "contour" maps (Figure 6.4), where a value of 100 represents the largest magnitude of oscillation for a particular mode . To evaluate the actual amplitude for a given site using this method, one must know how much energy

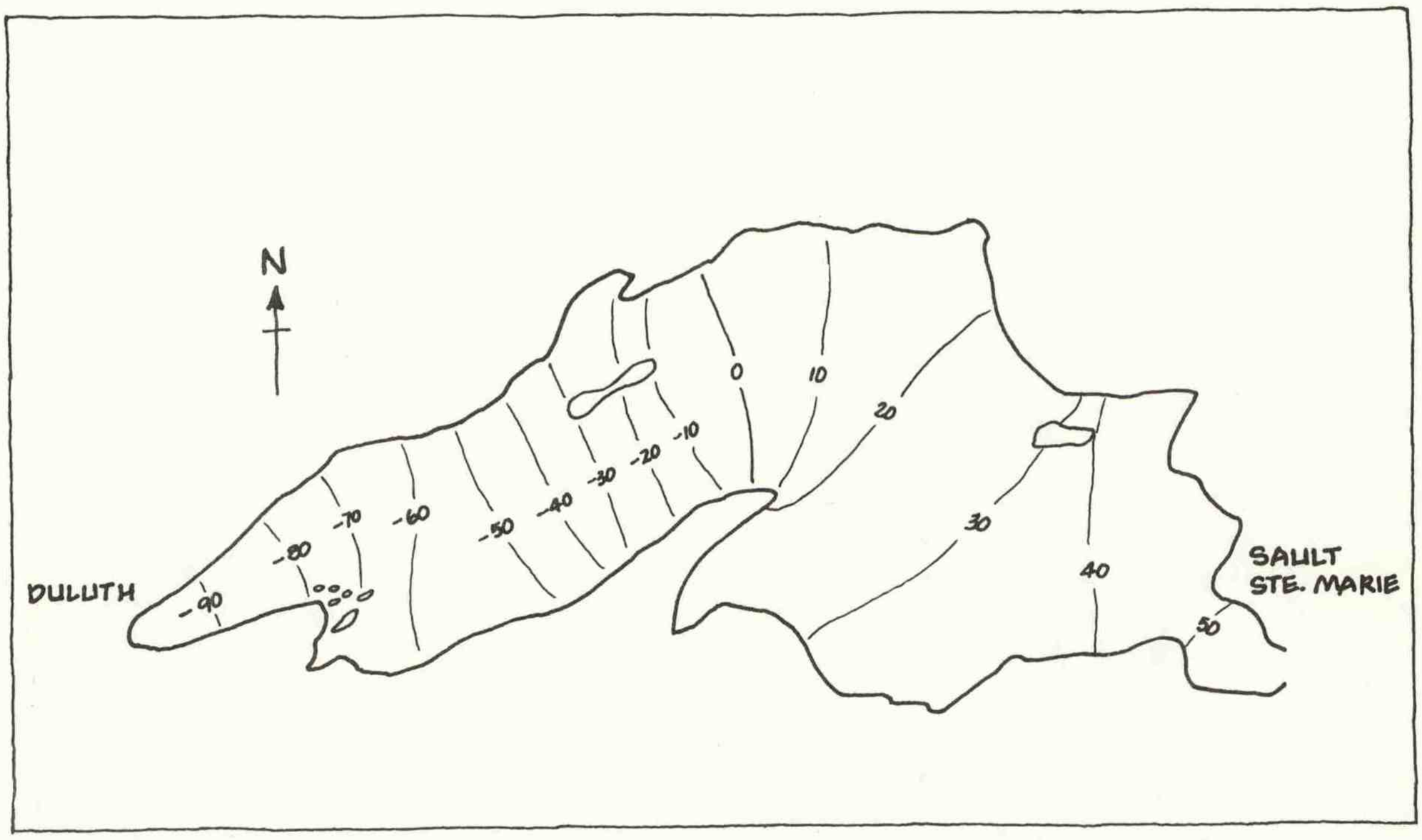


FIGURE 6.4: Basin Oscillations, First Mode, Lake Superior
Contour 100 = about 43 inches. (after Rao and Schwab 1976)

is within a given frequency band. This evaluation is difficult because the amplitude varies with time as a function of prevailing meteorological conditions.

Rao and Schwab (1976) note that the first mode exhibits a single nodal line (see Fig. 6.4). The maximum water fluctuation on the western end is about twice that on the eastern end. The second mode (not shown), with two nodal lines, has maximum water level fluctuations on the ends of the basin and less than a third of this value in the center. Even though these oscillations take place mainly along the longitudinal axis of the lake, as would be the assumption in a channel calculation, the two-dimensional shape of Lake Superior reflects itself in the asymmetric distribution of water levels on either side of the axis.

To obtain a very approximate estimate of the surface variability due to the first several modes at a given site, Roblee (1983) tried using equilibrium storm surge values for a given prevailing wind as the maximum amplitude of the fundamental oscillation of a given basin. For modes two through five, maximum values were obtained by dividing the primary mode's maximum value by the mode number. Wind conditions of about 70 miles per hour were used in estimating equilibrium storm surge values. These storm surge amplitudes, divided by the mode number, were used to correspond to the maximum value of 100 on the appropriate modes' "contour" map. For example, the first mode map for Lake Superior sets contour 100 equal to about 43 inches. Contour 90 at the western end then represents about 39 inches. For mode 2, contour 100 is estimated to represent about 21.5 inches, etc.

While the values obtained this way are not an accurate reflection of the real site conditions, Roblee used them to identify and separate likely active and inactive sites from among the 33 studied in detail. Also used were past observations at each of the sites on how "active" they were. The presence of pieces of ice rubble around fixed pilings--which break off from the ice cover when moved up and down with water fluctuations--was considered indicative of water movement "activity."

Roblee (1983) observed that sites classified as active and relatively inactive both had pilings uplifted. He concluded that any Great Lakes harbor has enough seiche activity to create ice uplift problems for piles. He also concluded that relatively high degrees of oscillation activity seem to lead to more extensive uplift damage, as was evidenced by extensive uplift in some active harbors. These conclusions agree with my field observations--you're not safe from uplift forces if your harbor is a Great Lakes harbor.

Two counter-situations need discussing, but neither negate the above warning. In an area like the Lake Superior Apostle Islands, where there is a lot of seiche activity, one could speculate that all that activity prevents a good firm bond of the ice sheet to the piles. Frequently, a definite working crack encircling the piling can be observed. Unfortunately, during the periods of inactivity, the ice grabs the pile and is all set to lift when the next seiche occurs.

The second counter-situation is for harbors that are several thousands of feet back from a Great Lake and are connected by a narrow channel. Under these circumstances, the ice sheet in the marina rarely oscillates anywhere near as much as the lake because of the small volume of water passing through the

channel. However, it occasionally moves enough to do damage. I therefore believe that all Great Lakes harbors can expect uplift from seiche--unless, of course, you bulkhead off the marina from the lake, a proposal that purportedly has been tried with only modest success and normally would be too expensive.

Hodek and Doud (1975) measured an almost constant fluctuation of the winter water level in a Lake Superior marina at Ontonagon, Mich. The amplitude had an observed maximum of 0.8 feet. The major period varied from 5 minutes to more than 10 hours. Higher frequency water oscillations also were observed, and a change in water level of 3 inches in 10 minutes was noted.

By drilling a hole in an ice sheet, you can see the high-frequency seiche up-and-down "pumping action." I have also watched ice sheets "bounce" by using time-lapse photography (one frame every 4-5 minutes over the course of several weeks). The ice sheets could be seen to oscillate one to six inches, to crack, flood with water and refreeze, and to grip and lift piles 15 feet. (All in all, a great picture show!)

The currents induced by the seiche phenomenon can be reversing ones. Changing winds can change surface currents and hence the direction in which broken ice pieces may move. Broken lake ice can be pushed shoreward, effectively damming up a river or stream discharge point. This causes the river or stream ice cover and flowing water to rise and flood, damaging marina structures as well as adjacent properties.



FIGURE 6.5: Marina Piers Damaged by Ice Movement with River Stage Changes

Figure 6.5 shows a marina dockage damaged by river ice. The rise in the river was caused by wind setup. The ice rose four feet during a two-day storm with 70-knot winds and then dropped, "breaking the back" of the piers.

In tidal harbors, Gill et al. (1983) report that dock support structures can accumulate exaggerated ice growth due to freezing of water on vertical surfaces following a high tide. Such a growth creates a buildup of ice that must be considered during design to ensure that the dock and foundation can accommodate the added loads. The initial layer of ice on piles and columns forms during falling tide, when a film of water is left on the pile. This layer gradually becomes built up with each tide and forms an "ice bustle."

Depending on the dock structure, these bustles may eventually join and form a lattice of ice ribs between the piles, or they may become incorporated into the larger ice sheet as it grows adjacent to the dock. The exact process depends on the dock configuration and the tidal extent. The growth rates of these ice cylinders or bustles have been measured at greater than an inch a day.

You need not be in a tidal area to see a similar phenomenon--in fact, the dock members need not even be in the water. Figure 6.6 shows dock members (steel bar joist trusses) coated with ice, which developed from spray off Lake Michigan under winds recorded at 80 to 90 mph. The individual diagonals and chords

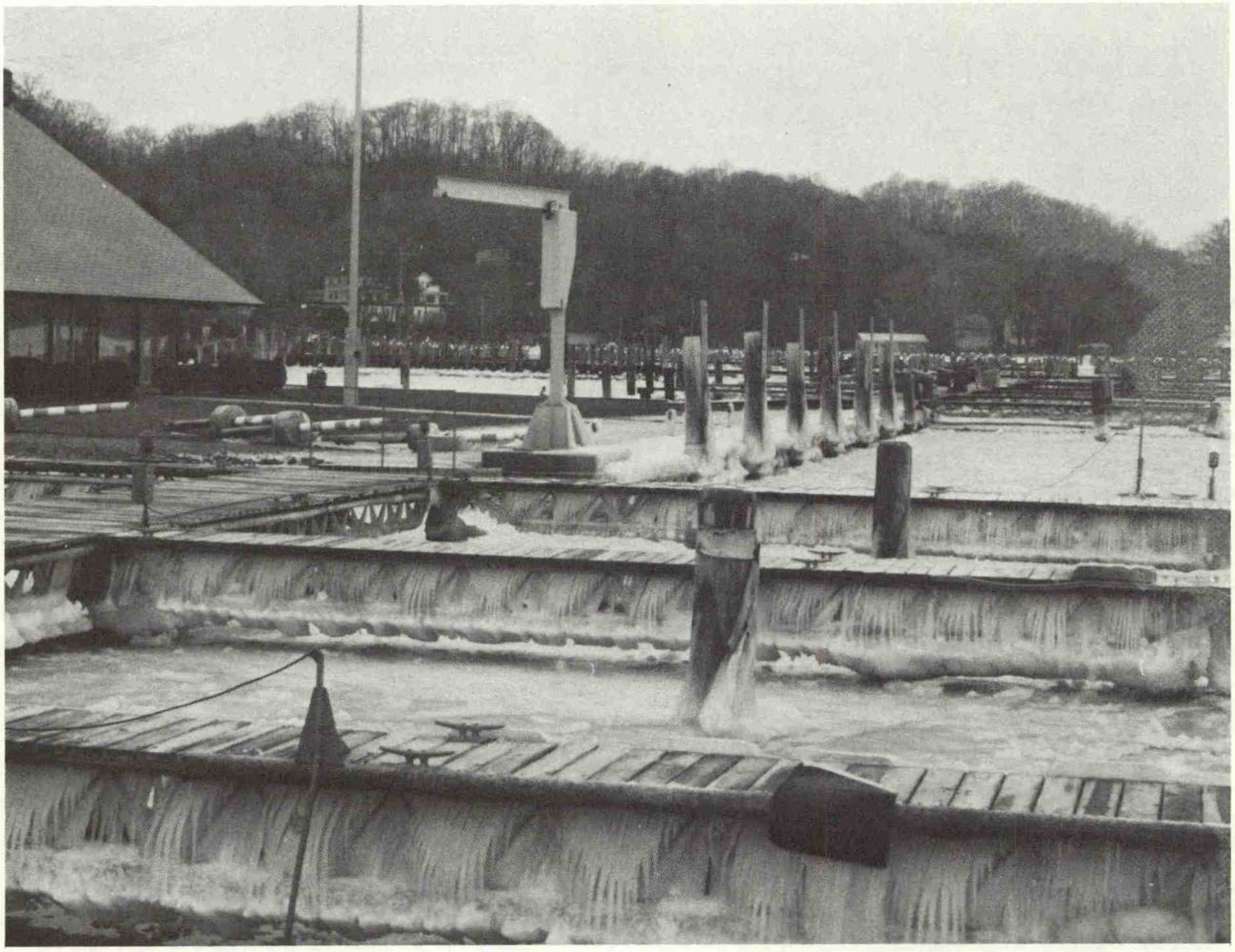


FIGURE 6.6: Steel Bar Joist Truss Dock Members Coated with Ice

became so covered as to form a solid cake of ice spanning parallel bar joists and exerting enough weight to cause several joist spans to drop into the lake.

In sum, the water phase of this interacting system is not at all static. Furthermore, many of the water motions probably are interrelated and therefore difficult to analyze.

Water Temperatures

Harbor water is cold--very cold! The water temperature at various depths has been measured in many harbors. The water temperatures in Great Lakes boat harbors are usually very near the melting point for ice. In many harbors, temperatures above 32 1/4°F to 32 1/2°F are rare. Also, the water is well mixed and hence isothermal. Temperatures in ice-covered rivers have been measured and usually found to be completely isothermal and at the melting point.

My own temperature measurements have been made with a thermistor bridge device, which can be read to 0.1 C° and interpolated to 0.05 C°. The instrument calibration frequently has been checked in a laboratory ice-bath. Field readings are usually 0.0°C to 0.2°C, and hence I say the water is rarely warmer than 32 1/2°F--in fact, often it is colder.

Under comparable ice cover conditions, the temperature measurements at a given harbor are repeatable year after year. For example, the water temperature in the Bayfield, Wis., harbor has always measured 0.0°C, and in a few other unusual harbors, the water has always measured warmer than 0.5°C.

Water temperature is important when we attempt to deice our structures. Even water very near 32°F melts an ice cover—it has to have some heat, as it still is water and not ice. Compressed air bubbler deicers melt ice at Bayfield, even with its cold water.

To measure the temperature of harbor water--if you don't have a calibrated thermistor--use a good quality 12-inch laboratory mercury thermometer (it costs about $75). Cut a hole in the ice and immerse the thermometer in a bucket or jar so when it is retrieved the surrounding mass of cold water insulates the thermometer long enough to read it. My experience with direct digital reading devices has been unsatisfactory mainly because the resolution of temperature near 0.0°C is not fine enough.

Another way to gage the temperature of harbor water is by observing its melting abilities and some empiricism. I have noticed where measured water temperatures are "warm" that alot of ice can be melted out with the same effort that in other "cold" harbors just barely is enough to keep the ice melted from around the harbor structures.

Data on water temperatures from several hundred Great Lakes harbors are available from me through the University of Wisconsin Sea Grant Institute.

Muschell and Lawrence (1980) report that in several instances it was found that water two to four feet below the bottom surface of an ice sheet, in six feet of water, was supercooled to -0.2°C to -0.3°C. This phenomena was not

limited to shallow water, either. In a test in the Straits of Mackinac, they found a uniform temperature of -0.1°C from the ice to the bottom, 89 feet below the surface.

I don't know what to make of their reports. The freezing point depression for "clean-clear" lake water is not this much. But I also have measured temperatures of -0.1°C and -0.15°C under ice covers in harbors. Either the measuring device is in error, or for some reason (perhaps due to currents) the harbor water becomes supercooled. In all likelihood, it is an error of measurement rather than the actual temperature of the lake water.

It really doesn't matter. Just remember that though the lake water under an ice covers is nearly freezing, it does hold heat that can be used to melt ice. Also remember that some harbors hold warm water capable of melting ice easily.

SITE CLIMATOLOGY AND ICE CONDITIONS

Chapter 2 discussed the different types of lake ice and how they form. Figures 2.1 and 2.2 show snow ice over clear ice and the variability of an ice sheet with depth. As noted in Chapter 2, ice not only grows down into the lake, but also thickens on the top. Also, ice can thicken quite rapidly in cold weather and conversely be quite effectively insulated from the cold by a covering of snow.

Hinkel (1983) noted that the rate of growth of an ice cover is strongly influenced by site-specific characteristics other than air temperature but was unable to quantify it, or to make forecasts from these site-specific characteristics.

The thickness of the ice in a small-craft harbor varies from place to place. For example, it usually is thicker near structures where cracking, flooding and freezing have occurred, and it is usually thinner in the structure-free aisles and fairways. Thermal stresses sometimes buckle, or push up, an ice sheet along the aisle axis (Figure 3.7).

In characterizing the ice in a given harbor, it is logical to include the thickness of the ice. It is generally assumed that ice thickness has a lot to do with lifting and shoving forces. This is somewhat correct. But for a number of reasons, some of which have been mentioned already, you needn't be too concerned with exactly how thick. Two ways to determine the ice thickness can be used: to forecast or estimate the thickness with a model, and to actually measure the thickness.

To forecast ice thickness, look at how cold it has been historically. The measure of "coldness" used is the freezing degree-day (FDD), which is defined as the departure of the daily mean temperature from the freezing temperature. For example, if the daily high was 20°F and the low was 10°F, the average would be 15°F, which is a 17-degree departure from the freezing temperature (32°F - 15°F = 17°F). The FDD would therefore be 17°F. A running sum of FDDs is a cumulative measure of the winter's coldness. If this sum becomes negative due to warm weather, a new sum is started on the next day.

Table 6.3 is an 80-year record of FDDs at various sites accumulated on a daily and a weekly basis. The daily basis is termed the mean FDD, and the weekly basis is termed the maximum (extreme) FDD. The weekly sum is computed by stringing together the coldest weeks over 80 years.

Observations show the growth of ice is proportional to the square root of the FDD sum--except very early on, when it grows at a faster rate, and except much later on, when it grows more slowly. A rule-of-thumb equation incorporates a constant of proportionality, the locality factor (α), with this observed relationship to FDDs to derive a thickness of ice. It is:

$$h = \alpha(FDD)^{1/2} \qquad (6.1)$$

TABLE 6.3 Eighty-year Mean and Maximum Freezing Degree Days (F°)
(after Assel 1980)

Station	Mean	Maximum	Station	Mean	Maximum
LAKE SUPERIOR					
Thunder Bay, Ont.	2,500	3,300	Marquette, Mich.	1,400	2,500
Houghton, Mich.	1,650	2,400	Sault Ste. Marie, Mich.	1,650	2,600
Duluth, Minn.	2,250	3,050			
LAKE MICHIGAN					
Escanaba, Mich.	1,400	2,400	Muskegon, Mich.	600	1,400
Traverse City, Mich.	900	2,000	Milwaukee, Wis.	800	1,650
Green Bay, Wis.	1,350	2,300	Chicago, Ill.	500	1,400
LAKE HURON					
Parry Sound, Ont.	1,500	2,550	Bay City, Mich.	750	1,650
Alpena, Mich.	1,150	2,000	Port Huron, Mich.	600	1,550
LAKE ST. CLAIRE-LAKE ERIE					
Buffalo, N.Y.	500	1,200	Erie, Pa.	400	1,100
Port Dover, Ont.	700	1,800	Toledo, Ohio	450	1,350
Detroit, Mich.	500	1,350	Cleveland, Ohio	300	1,200
LAKE ONTARIO					
Kingston, Ont.	1,150	2,000	Oswego, N.Y.	650	1,500
Toronto, Ont.	600	1,500	Rochester, N.Y.	600	1,300

where: h = thickness of ice (inches)

α = locality factor (ranging from 1/3 for heavily snow-covered lakes to 1 for snow-free lakes)

FDD = accumulated freezing degree days (F°)

As for knowing what value to use for α, you can either assume a value if you characterize the snow cover in the marina, or you can derive it historically using actual ice thicknesses, snow conditions and climatic data.

To illustrate this methodology, I will use an actual case to estimate the thickness of ice during a warming trend on Little Traverse Bay near Harbor Springs, Mich. The object was get an idea on how much ice was in the bay when the warm spell occurred and to try to determine if damage to a marina pier was related mainly to thermal shoving of ice or to some other cause.

I obtained records on snowfall, snow on the ground and daily temperatures near the bay from the city of Petosky and Pellston airport. The accumulated FDDs for December through March were 1,100 F°. The average sum of FDDs for nearby cities was 1,275 F° (estimated from Table 6.3 for 1,650 F° at Sault Ste. Marie and 900 at Traverse City). So, the temperatures in Harbor Springs area were about 86 percent of the 80-year mean. (Remember, there is a 50 percent chance that a site will be either colder or warmer than the average!)

In mid-February, two feet of snow was on the ground at Pellston. This snow melted away just a few days later, and there was no snow cover the rest of the winter. The temperature records for Petosky indicated that beginning Feb. 15 the average daily temperature rose above freezing (32°F). For a week the temperatures were in the 40s and 50s, and the average daily temperature averaged 33.5°F from mid-February to mid-March. The first half of February had only four days with lows below 0°F, with -12°F the lowest temperature recorded.

Between December and March, Petosky received a little less than 100 inches of snow. The bay and harbor presumably had less. I assumed a locality factor (α) of 3/4 and from actual FDDs calculated sheet thicknesses of 20 inches at the end of January and 24 inches at the end of February. These calculations agreed well with my actual thickness measurement in Harbor Springs on Feb. 4--55 cm of ice, or about 22 inches. (You may think I back-calculated α to be equal to 3/4, but actually 3/4 is a pretty good number for snowy areas.)

As with water temperatures measurements, measurements of the thicknesses and types of harbor ice in several hundred Great Lakes harbors can be obtained from me through the University of Wisconsin Sea Grant Institute.

These measurements indicate that 3 feet of ice can be expected in northern harbors and 2.5 feet or less in southern harbors of the Great Lakes. For the more southerly harbors, the actual thickness of the ice is frequently a direct function of the amount of snow ice present and not so much the temperature.

The thickest ice I've measured on the Great Lakes was 135 cm (about 53 inches) under snowswept docks at Ontonagon, Mich. Ontonagon is near Houghton, where the maximum FDD is 2,400. The square root of 2,400 and $\alpha = 1$ give a thickness of 49 inches. Cracking, flooding and thickening of the sheet from the top

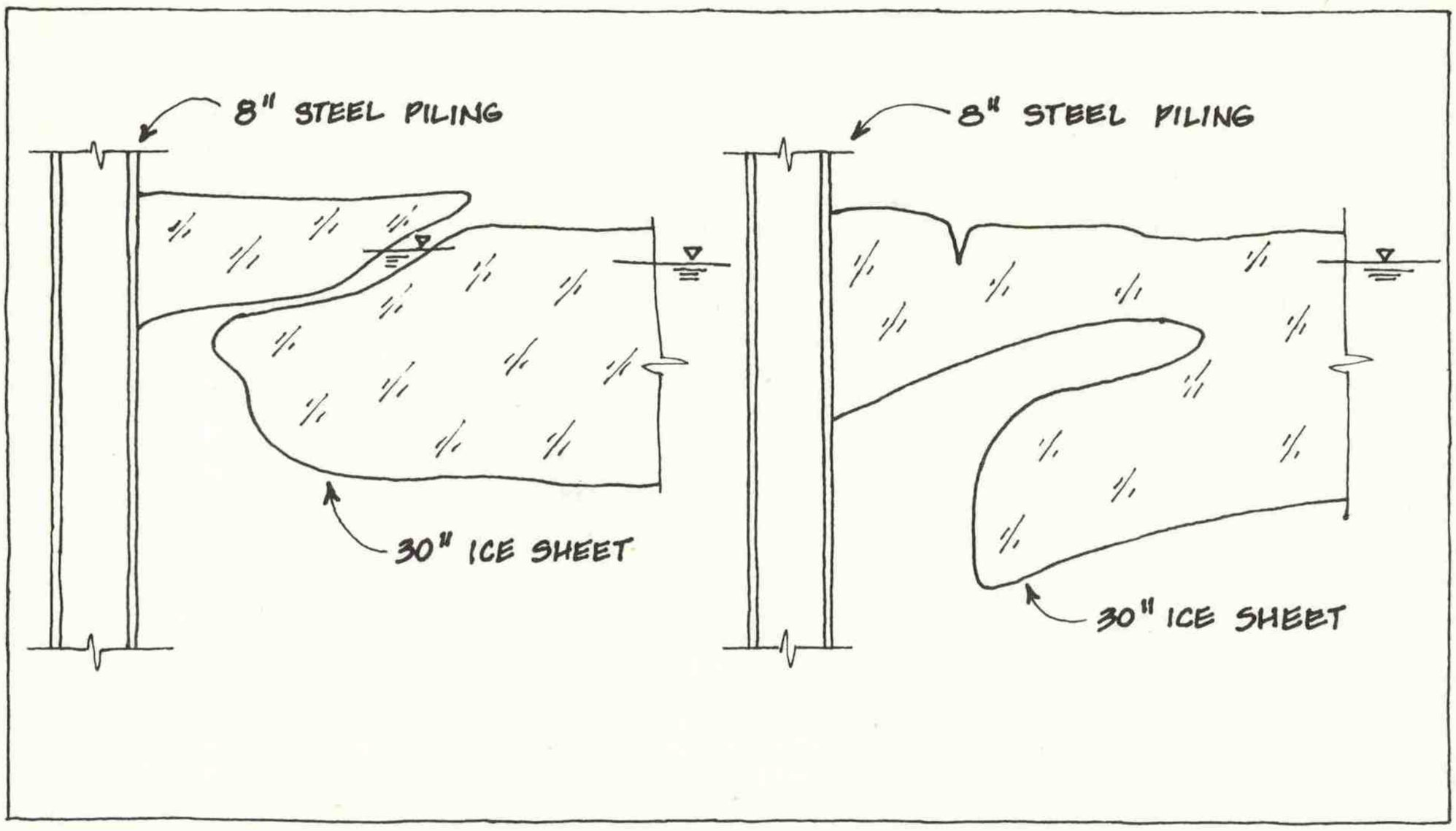


FIGURE 6.7: Half Cross Sections of Ice Sheets Attached to Steel Pilings

down perhaps explains some of the difference between the actual and calculated thicknesses. By comparison, Duluth, also on the south shore of Lake Superior, has a maximum FDD of 3,050 and a thickness of 55 inches; Marquette, Mich., has a maximum FDD of 2,500 and a thickness of 50 inches.

To characterize ice conditions, you must also look at water levels and temperatures. Several harbors that have "warm" water appear to have less uplift problems--perhaps because of sheet thickness, but I suspect more from a thermally-induced loss of the ice's adhesive grip on the pilings. In harbors with a lot of seiche activity, many times the ice is not even frozen to the piling. Under the snow cover will be thin ice or open water, or if the piling is surrounded by ice rubble chunks, there may be ice-pile attachments as shown in Figure 6.7. This figure shows two half-cross sections of a 30-inch sheet ice "frozen to" steel harbor piles. Note that the actual attachment is half the sheet thickness or less.

Currently there is no specific way to characterize ice conditions in a Great Lakes harbor. However, I plan to write a compendium of winter conditions in select Great Lakes harbors. Factors in any such classification system will include thickness, type of ice, snow cover, integrity, water level and temperature.

GENERAL SITE FEATURES CHARACTERIZING SMALL-CRAFT HARBORS

The location of the small-craft harbor site is important because of physical factors like the amount and type of ice, water level activity and subsurface conditions. Location is also important in terms of accessibility in winter.

For example, if mechanical-electrical deicing equipment is used, can it be secured from vandals and can it be serviced by maintenance people? Is the power source reliable or is interruption likely? Are people available who can observe conditions in the harbor during the winter and who can chop ice or take other stop-gap measures if necessary? These are some of the questions that must be asked about the site for a Great Lakes marina.

I have seen practically no vandalism of marinas during the winter, but the possibility remains. Though otherwise secured by fencing during the boating season, in winter a marina is wide open on the ice-covered lake or waterway side. One solution might be to put a line of deicing bubbles across the entrance, thus securing the marina with an open-water "moat." Many marinas permit ice fishing, and I think these anglers perhaps help keep an eye on things. But I know of a marina in a large metropolitan area that, when it gets bitter cold, loses any loose wood decking boards to bonfires.

Heavy snow can cause a marina to become inaccessible for extended periods if the marina's connection to a public roadway is a long, unmaintained service drive. What if there were a fire? Heavy snow and icing conditions can bury a marina or so encapsulate it as to preclude all access.

Orientation of the marina may be a factor with regard to winds--as in blowing sleet and drifting snow. Floating docks can get tipped and twisted as a result. A merchant marine training vessel once became so iced from spray on its windward side that it capsized--went bottom-side up (the students, being students, painted "this side down" on the exposed hull). Will wind-driven ice impinge the docks--either with sharp, jagged plates of broken ice cover, or with blocks? I have seen blocks roughly the size of automobiles (but not as thick) bobbing around in a Lake Superior marina on Memorial Day--after a number of yachts had already been launched for the summer!

Not only can orientation be a factor with ice coming at the marina, but what if the ice moves away from the marina? A marina on a bay on Lake Ontario lost an entire floating dockage system that way. A March storm blew a 2-foot cake of ice away from the shore, carrying a floating dock and some fixed pier construction across the 2-mile bay, never to be seen again. Presumably, the dockage was chewed and ground up. The ice literally bit off the ends of long head piers connecting to the leeward shore!

Besides protection and orientation, the shape of the marina basin is definitely another factor. If it is a rather confined area with a small outlet--and especially if the shorelines are bulkheaded--there will be high lateral forces if sound, intact ice is present during a thaw. Sloping banks may reduce this problem--but just how much is unknown.

There are many other important general site conditions to consider. The point is that the design of a marina or small-craft harbor is site-specific, and the designer must attempt to characterize each site with the kinds of factors mentioned here in mind.

7. Preliminary Design Considerations

After characterizing the marina site, you are ready to make design decisions about the layout of the harbor and structures. You may study several sites, but usually you will be fitting a planned marina into a given site whose characteristics will largely determine whether fixed/adjustable structures, floating structures, movable/removable structures or deiced structures can be used. Marina design requires consideration of environmental conditions as well as such conventional factors as safety, economy, appearance, service life, use and local preferences and needs. These factors are routinely addressed in engineering projects, but marina design for ice conditions requires special attention to one of them: economy.

ECONOMY IN SMALL-CRAFT HARBORS

Economical design of a small-craft harbor means that the functions and serviceabilities of all its components are evaluated in terms of initial costs, operations and maintenance costs, and repair and replacement costs. The Great Lakes are indeed a harsh environment. Cheap docks are not the way to go. Except in unusual circumstances, such as temporary construction, all costs must be analyzed within the project's overall economic framework.

Two somewhat recent analytical tools are now available. They are "life cycle costing" and "value analysis/value engineering." They are not really new concepts; engineers have always used them. What is new is the degree to which the analytic methods have been articulated.

Life cycle costing (or engineering economy) looks at the time value of money and economic choices among alternatives. The methods now used cover not only costs and interest rates, but also assessments, commissions, taxes and inflation rates.

As a methodology, value engineering examines the actual costs associated with functions, including both those functions essential to accomplishing the main task as well as nonessential functions (i.e., adding to or enhancing the construction in some way). Both types of functions are legitimate--value analysis doesn't mean cheapening everything, it means systematically examining what the task is and how to accomplish the project economically and in a manner acceptable to its users.

Value analysis expresses tasks and functions as simple verb-noun couplets. For example, the main task of a university extension continuing education department might be stated as "impart knowledge." Now, what is the main task of a lawn mower? If you answered "cut grass," a value analyst would say you're wrong. Cutting the grass is accomplished with the lawn mower, but you don't cut the grass for the sake of it. The value analyst would probably say the main task of a lawn mower is "groom lawn"--to care for its appearance, to make it neat and tidy. The next step in the value analysis process is to ask how else this task might be accomplished, at what cost and at what level of acceptibility to the lawn's owner or viewers.

For a small-craft harbor, the main tasks of a host of items could be analyzed --the breakwater, head pier, finger pier, mooring pile, deicer, bulkhead, etc. The main tasks of these items are not simple or easy to identify. For example, a deicer can remove all ice, or simply suppress and weaken ice--but why is the deicer to be used? Then ask how else might this task be accomplished and at what cost and acceptibility.

Books, classes and home study aids on life cycle costing and value enginering-value analysis are available from the University of Wisconsin.

"Cost of repairs" is one cost item that needs to be addressed in a marina project economic analysis. Obviously, it is a relevant cost, but one not frequently addressed realistically--probably because of unfounded optimism and a lack of understanding about what can happen. Figures 7.1 and 7.2 illustrate two cost-of-repairs items that need consideration.

The fallen pier shown in Figure 7.1 is the result of a deicing system that failed to keep ice away from the supporting piles. The finger pier is framed as two simple spans of precast concrete channels supported on two pairs of pilings and also supported at the main head pier. Earlier in the season, the flexible intermediate piles were shoved by ice, and because of the narrow, simple support for the heavy concrete pieces, the dock sections fell into the lake.

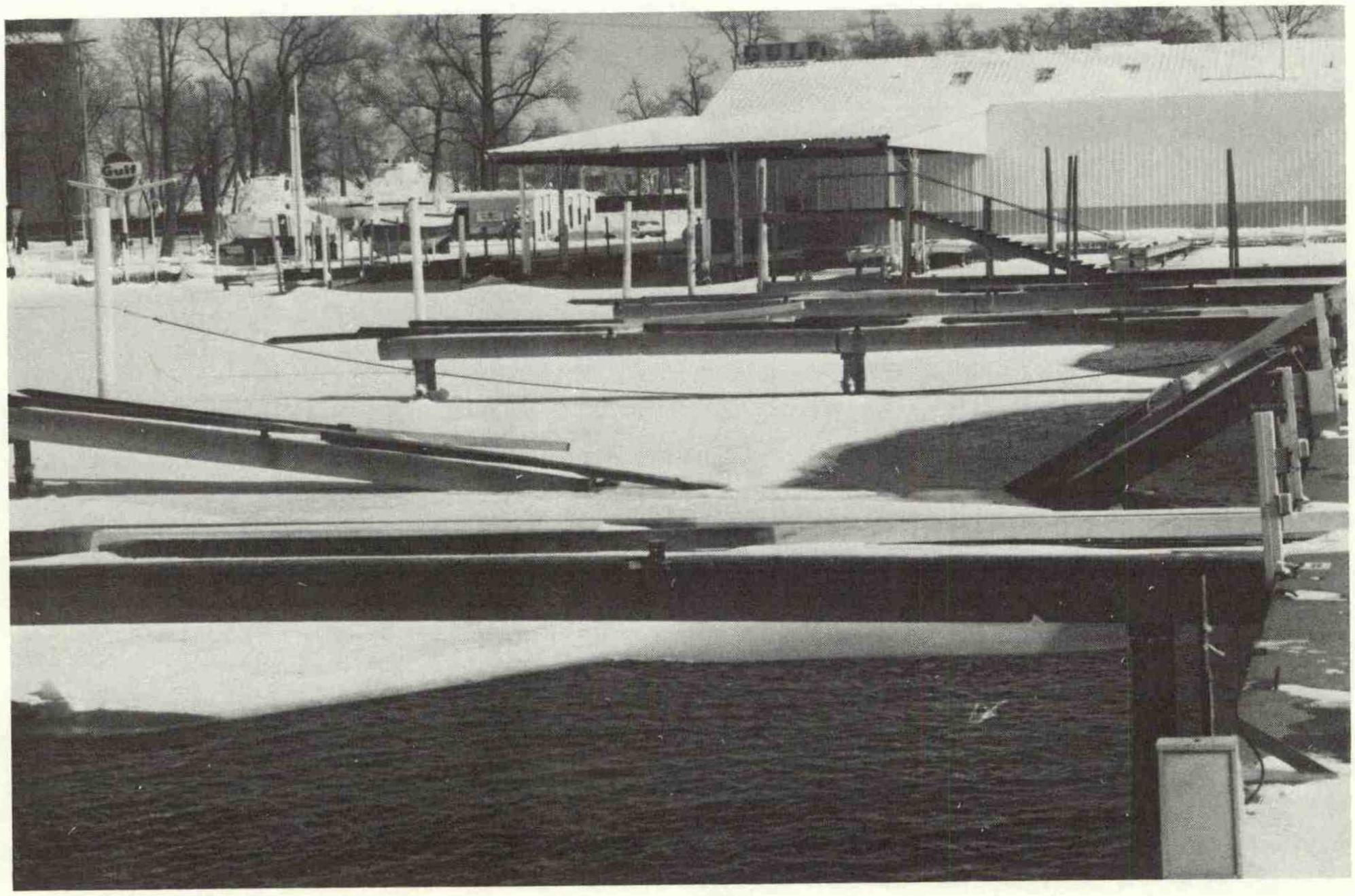


FIGURE 7.1: Damage to "Deiced" Dock

The questions that need to be asked include: what will it cost to repair the damage resulting from a system failure and a design detail inadequacy, and what can be anticipated where several hundred piers of this design exist?"

In Figure 7.2, the designer assumed that deicing was not necessary. But every piling lifted and must be redriven and deiced. The soil conditions were inadequate for the pile penetrations--the outer piles lifted the same amount everywhere, indicating they have pulled free of the harbor bottom. What will it cost to redrive the pilings for more than a hundred such slips?

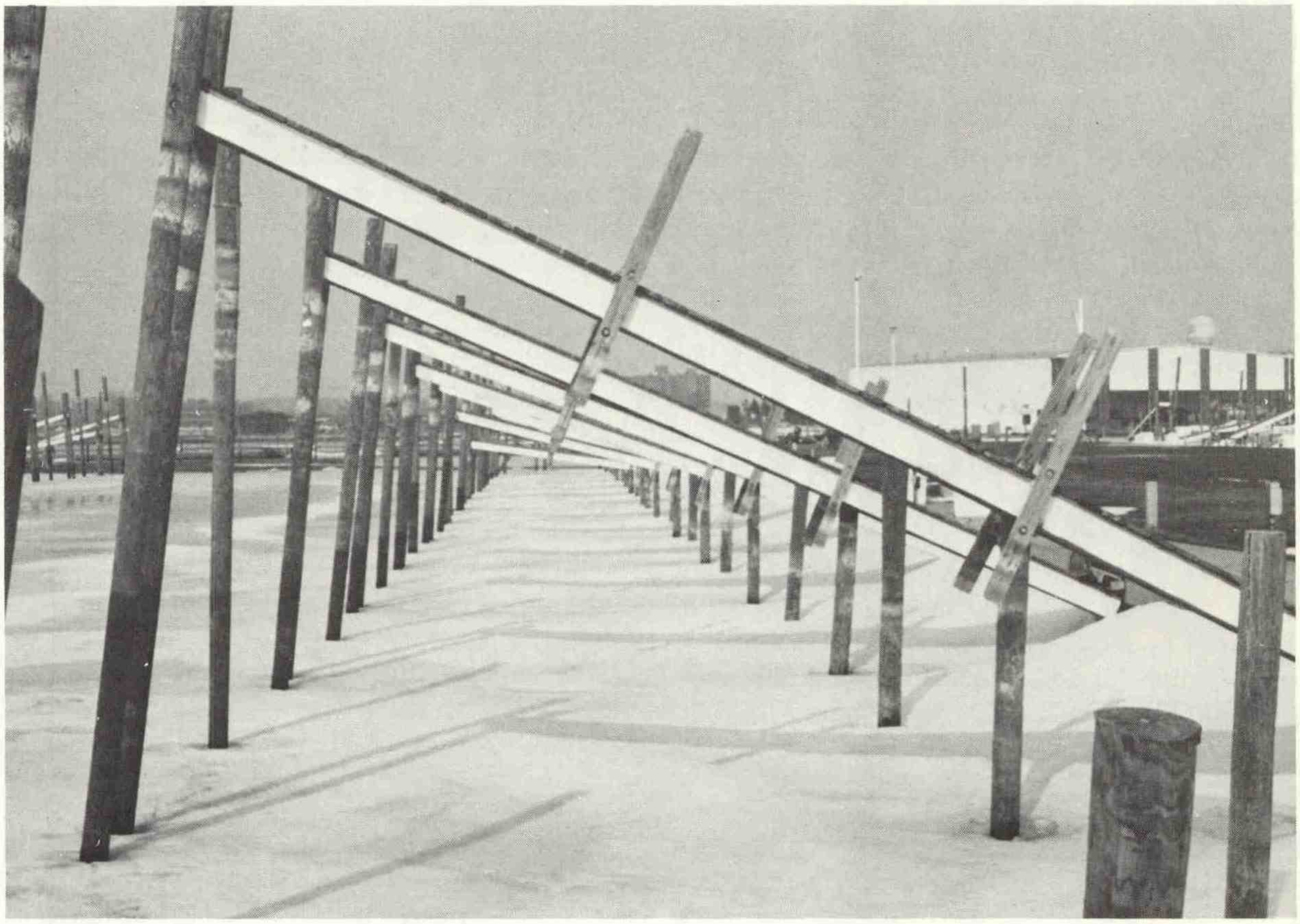


FIGURE 7.2: Damage to "Unprotected" Dock

TYPES OF SMALL-CRAFT HARBOR STRUCTURES

When harbor structures are introduced into the soil-water-ice environment, one of several facility types or a combination of types must be selected. Using a combination of dock types in the same marina may be the best choice to meet all design and cost objectives. For example, main head piers might be crib structures, with finger piers being floating elements. Deicing might be used in areas where good pile penetration is not available, while elsewhere the docks would be supported on deeply driven piles. Slips for large boats could be fixed height (high), while slips for small boats could be floating (lower).

Fixed structures frequently cost more than floating structures, but this is not always true. Many factors enter in--for example, the materials to be used, their life expectancies and maintenance costs, the number of slips to be built, the depth of the water, the type of soil present, etc.

If the harbor is deep, the choice may be floating docks due to the expense of long pilings. A floating system may also be chosen because the mix of boats includes many small boats. The Michigan Waterways Commission uses floating piers for 30-foot slips and occasionally for 45-foot slips. For slips less than 45 feet, they sometimes use adjustable-height finger piers connected to fixed-height head piers.

The most damaging ice conditions for unfixed, floating docks left in the water all winter is shifting and moving ice (Figure 7.3). However, if the harbor is well-protected and quiet, the floating piers can freeze in and melt out with little, if any, problems (Figure 7.4). Docks can be constructed segmentally with easy disconnects to allow removal and land storage (Figure 7.5), or constructed with hinges to allow them to be pulled up and back out of the ice (Figure 7.6).

Fixed structures are structures that are set on the bottom of the harbor or are supported on pilings driven into the bottom. If the structure rests on the bottom, it is referred to as a gravity structure and obtains stability from its size and weight. Pilings obtain stability from size and depth of penetration. Fixed structures can go it alone against ice forces and actions, or they may be partially or fully protected. Booms and deflectors can be used as shields against moving ice, coatings and wrappings can reduce ice forces, and suppression methods can be used to control ice forces and actions. Figure 7.7 shows a fixed dock structure with deicing protection. Incidentally, deicing can and is sometimes used with floating structures. The decision to use deicing also may be influenced by the warmness of the harbor water.

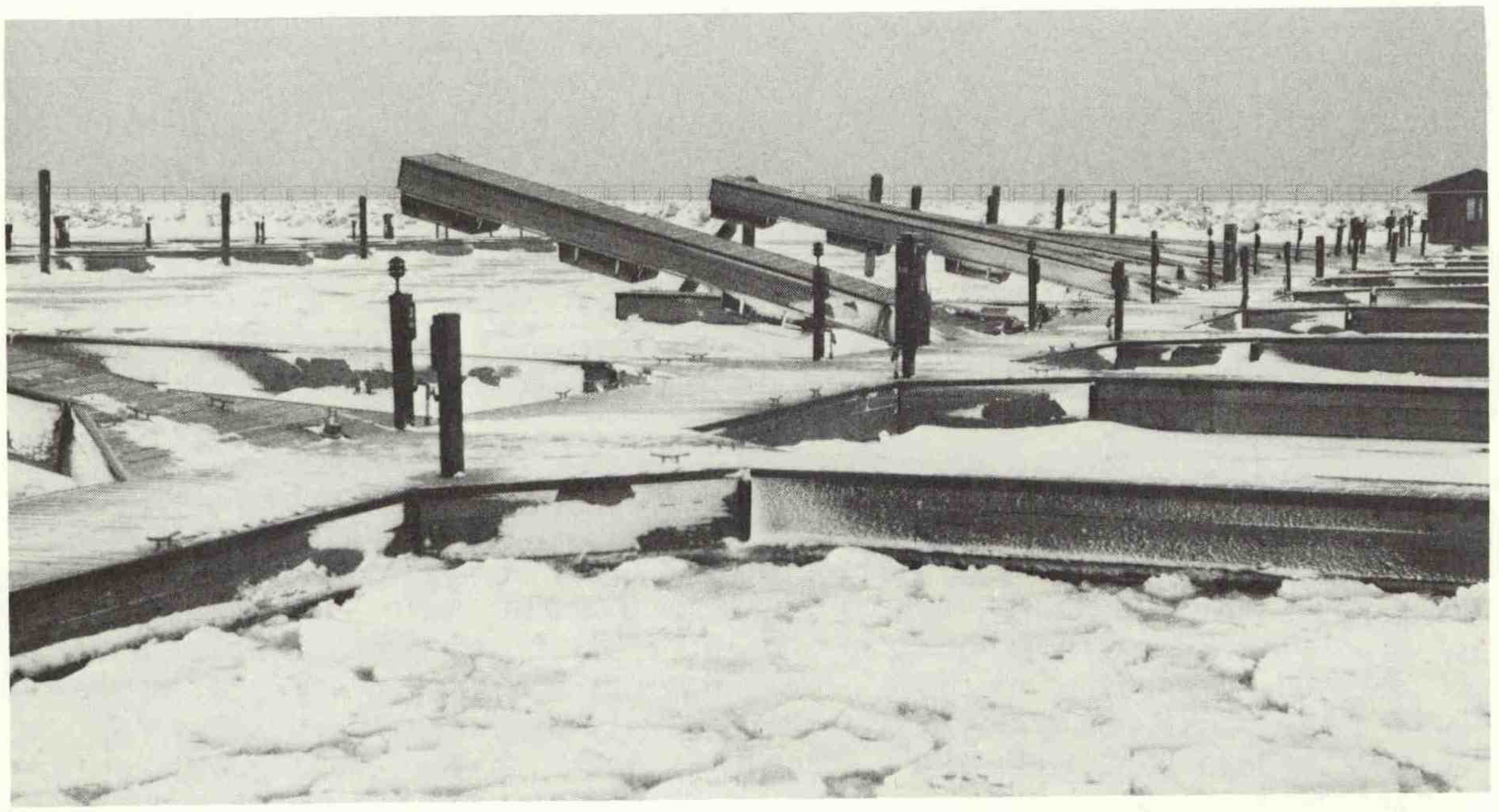


FIGURE 7.3: Floating Docks Damaged by Shifting and Moving Ice

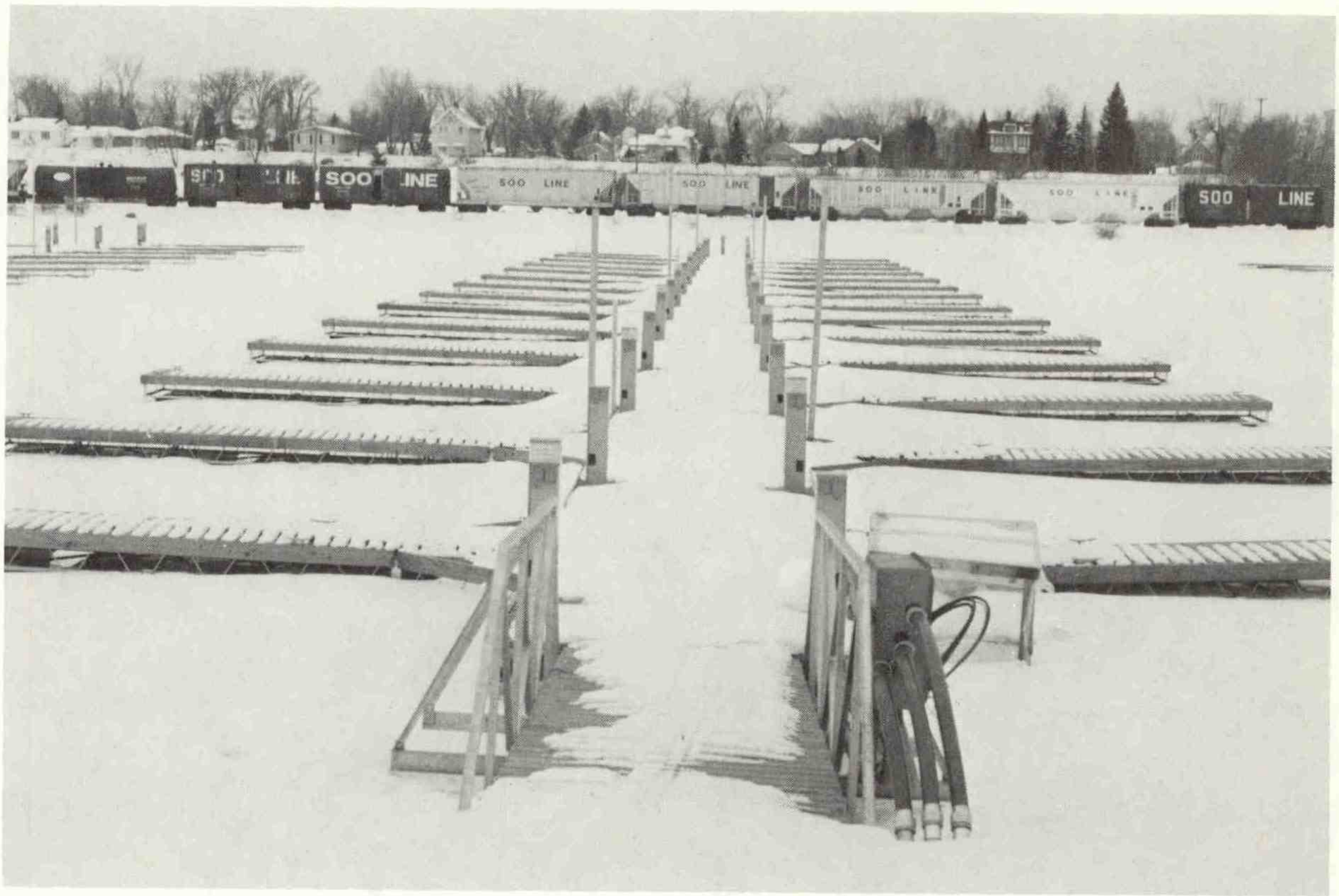


FIGURE 7.4: Floating Docks Wintering in a Quiet, Well-Protected Harbor

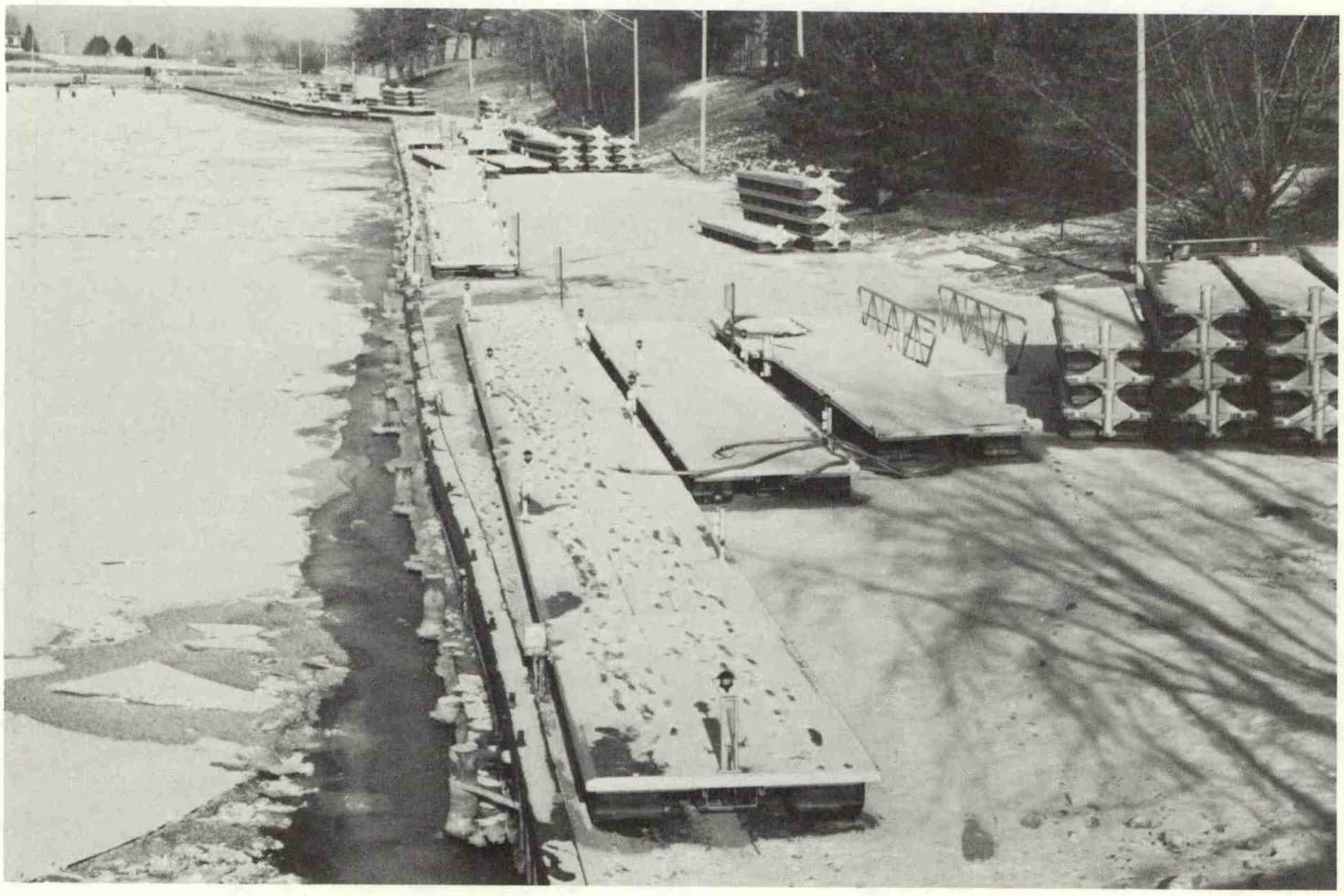


FIGURE 7.5: Floating Docks Removed and Stored on Land

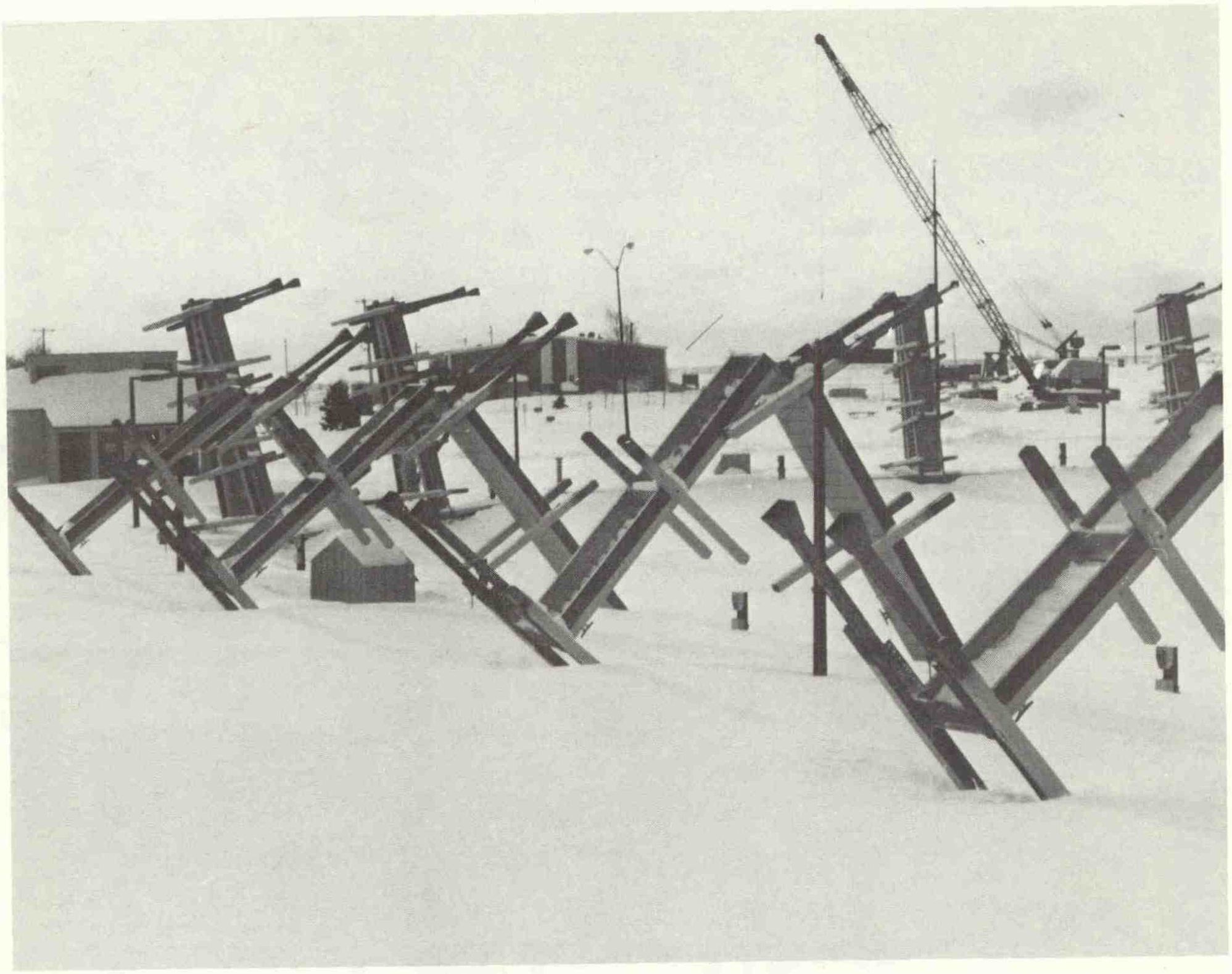


FIGURE 7.6: Fixed Piers Hinged for Winter Removal

Muschell (1981) raised an interesting design concept. He recommended that harbor designers use climatic data in determining damage coefficients and frequency curves for alternative designs. In lieu of designing against all ice forces and actions, the designer would be permitted some annual damages as an economically sound, cost-benefit type of choice. So far this suggestion has not been developed and implemented, but my recommendations about realistically assessing costs of repairs is a step in that direction.

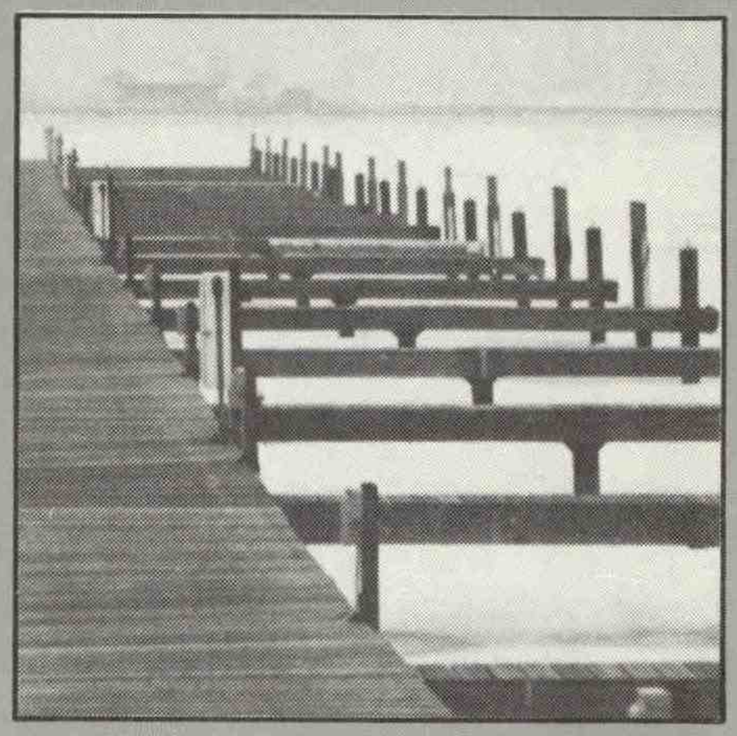
I thought of including a design-decision matrix in this discussion of preliminary design, but I decided it would be rather intricate and of limited use to engineers. As designers, you will think through the options. The next chapter presents technical information and advice on design methods.



FIGURE 7.7: Fixed Piers Protected with Deicing

Part Three:

DESIGN METHODS



8. Design Methods to Suppress and Control Ice

Compressed air bubbler deicing is an effective ice suppression and control method. The air bubbles move "warm" bottom water to melt the underside of surface ice. This method is discussed first. The detailed design and system layout is outside the scope of this manual; I recommend you get assistance from qualified mechanical engineers, contractors and equipment manufacturers.

COMPRESSED AIR BUBBLER DEICING SYSTEMS

Ashton (1974) prepared a monograph on air bubbler systems. He used analytical methods to develop a procedure for predicting the effectiveness of these systems in suppressing ice under various field conditions. Ashton (1975) reported on laboratory experiments on line-source bubbler system heat-transfer coefficients. The material presented in this section is based largely on Ashton's work. This work reasonably predicts observed suppression. The section begins with a description of how a compressed air bubbler deicing system works.

Principles of Compressed Air Ice Suppression

Figure 8.1 is a schematic cross section along the axis of an air diffuser pipe placed on a harbor bottom. The figure also depicts the action of a point-source diffuser.

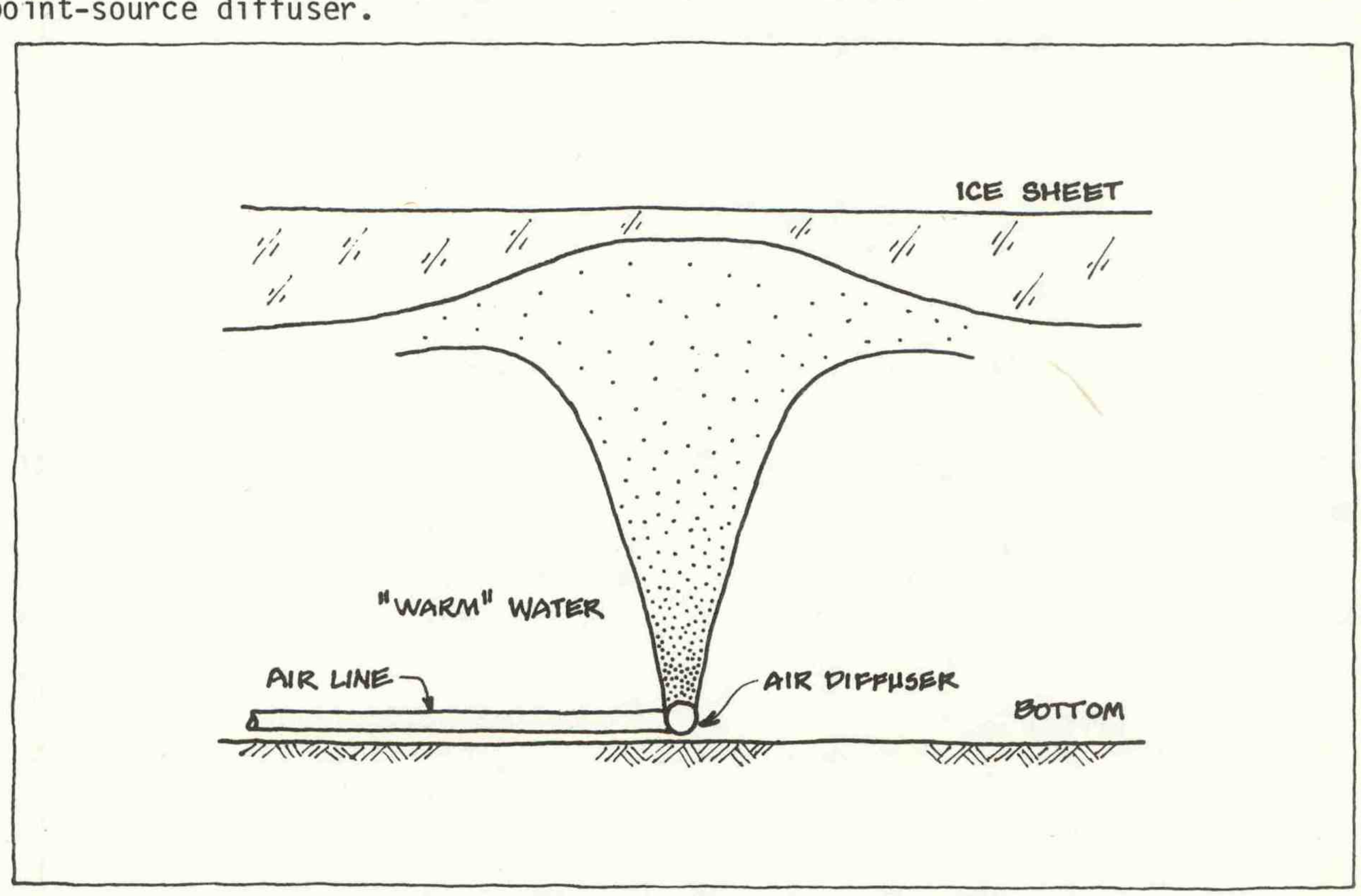


FIGURE 8.1: Compressed Air Ice Suppression System (after Ashton 1974)

Air is compressed by a compressor, which may be located indoors or outdoors in a protective shed. One or more compressors may be used, interconnected through a discharge manifold piping system, which is usually laid on the harbor bottom. Air travels through the manifold piping to perforated diffuser lines, or the manifold itself may have diffuser orifice ports that emit air. The air warmed during compression is normally cooled to near-ambient lake temperature by the time it reaches the diffuser. The air must have enough pressure at the diffuser to overcome the depth of water hydrostatic pressure.

The discharged air expands and cools slightly on leaving the orifice. If it expands too quickly, an icing condition theoretically can develop at the orifice, but with proper air pressures in the system this does not happen. As the bubbles rise, they entrain lateral waters in the rising plume. Many sizes of bubbles are created, but the tinier ones are more efficient because they move more water for the same volume of air. This explains why large bubbles--like those produced by belcher devices--aren't as effective as a continuous stream of tiny bubbles.

The air bubbles themselves don't melt the underside of the ice. Melting results from both the temperature and volume of water moved upward by the bubble plume.

The volume of water needed to melt a cubic foot of ice at near-melting temperatures depends on the temperature of the water, the unit weights of both water (62.4 pounds per cubic foot) and ice (57.2 pounds per cubic foot), the specific heat capacity of water (1 BTU per pound per °F), and the heat of fusion of ice (143.7 BTUs per pound). Assume the water is 33°F:

$$\text{Ratio of volumes} = \frac{\text{Heat required to melt ice}}{\text{Heat content of water}}$$

$$= \frac{(57.2)(143.7)}{(62.4)(1)(33-32)}$$

$$= 132 \text{ cubic feet of water}$$

If the water temperature was 32.5°F, it would take 264 cubic feet of water to melt 1 cubic foot of ice.

At some point near the surface, the bouyant plume spreads as it encounters the ice cover or reaches open water. Upon hitting an ice cover, the bubbles move laterally, tickling the underside of the ice and melting it primarily by convection. This heat loss results in a cooling of the flow of bubbles. The velocity of the flow also decreases as it spreads out, causing a rapid decay in the heat transfer rate.

Finally, the plume imposes a net circulation on the water, and this allows more warm water to be drawn from distant lateral directions. A bubbler system won't work in a swimming pool because the amount of available warm (liquid) water is limited by the pool's volume. Instead, theoretically, the pool's water would freeze all the way to the bottom, assuming no heat inputs to the pool and the ambient air temperature is below freezing.

I doubt that the waters of a typical marina can freeze all the way to the bottom; at least I have not seen any problems of this kind. Bengstsson (1981)

measured heat transfer rates from lake bottom sediments up to 1 BTU per hour per square foot in early winter, decreasing to one-half to one-third of a BTU later. Also, usually there is some circulation of the water, which also helps prevent a freeze all the way to the bottom.

It is often said that a 6-foot water depth is needed to satisfactorily operate a bubbler system. This is about right and perhaps conservative, because with enough air you can melt ice as fast as it forms on any water surface. In shallow water, sometimes grounded ice is found--especially with heavy snowfalls. When augering grounded ice, typically two or three feet of wet spongy ice is encountered before the bottom sediments; there is barely any water under the ice. Obviously, turning on a bubbler system in this case would do little good.

In sum, assume you need about six feet of water, some water circulation and some heat inputs.

Method of Estimating the Air Required to Suppress Ice

The following is a trial-and-error procedure (after Ashton 1974). For a given site and conditions, the quantity of air required (Q_a) is estimated for a tolerated ice equilibrium thickness (n_e). This is the steady state thickness of an ice cover that persists between a continual flow of heat melting the ice from below and a cold atmosphere forming ice from above. In other words, equilibrium thickness is when the ice is melting as fast as it forms.

The design of an ice suppression system requires that a tolerated equilibrium ice thickness be selected. Disproportionate amounts of heat would be required to reduce it to zero or nearly so (i.e., large air flows are needed to eliminate an ice cover completely). The ice itself is an insulator and prevents heat loss. Nonetheless, I prefer a system design that is capable of completely melting out the ice cover under most weather conditions, since with an open water surface you can see that the bubbler system is working. With even a thin ice cover you can't.

In selecting tolerated ice thicknesses, consider first costs and operating costs, the resistance available through soil embedment of the piles to be protected, the magnitude of lateral forces from the thicknesses of ice, the availability of labor to chop ice during severe cold periods, the temperature extremes of the site, and the amount of damage to be tolerated.

The quantity of air required (Q_a) can be estimated from experience and local conditions. Table 8.1 gives heat transfer coefficients (h_b) as a function of water depth (H) and air flow rate (Q_a). The water depth used is from the underside of the ice (from the top surface for suppressed ice) to the diffuser level (generally on the bottom). The heat transfer rate, then, is:

$$q_w = h_b \ (T_w - T_m) \qquad (8.1)$$

Where: q_w = required heat transfer rate (BTUs per hr. per sq. ft.)
h_b = heat transfer coefficient (BTUs per hr. per sq. ft. per °F)
T_w = water temperature (°F)
T_m = melting point temperature of ice (°F)

TABLE 8.1: Heat Transfer Coefficients h_b
(BTUs per hour per square foot per degree Fahrenheit)
(after Ashton 1974)

Water Depth (feet)	Air Flow Rate Q_a (in cubic feet per minute per 100 feet of diffuser)					
	2	4	6	8	10	20
6	169	189	201	211	218	257
10	150	167	178	187	194	228
14	135	151	162	169	175	207

It is reasonable to assume the water's temperature is constant throughout its depth because of normal harbor water mixing and the bubbling action.

Table 8.2 lists the ice equilibrium thickness (n_e) for the required heat transfer rate (q_w) as a function of the ambient air temperature (T_a). It is assumed that no snow cover is present and the average wind speed is 10 miles an hour. If snow is present, the equilibrium thickness will decrease for a given q_w. Conversely, if windier conditions prevail, the equilibrium thickness will increase. Use the average daily temperature of the winter period under consideration for T_a. The average daily temperature is computed by averaging the day's high and low temperatures.

Example Computation of the Compressed Air Required

The following example illustrates the use of Tables 8.1 and 8.2 and Equation 8.1 to determine the quantity of air required to protect marina pilings in the Milwaukee harbor. Actual harbor water temperature measurements are in the range of 32°F to 33°F, occasionally 34°F. The water depth is 8 feet. From Table 6.3, the mean Freezing Degree Day (FDD) total is 800 and the maximum is 1,650. The record cold snap duration is 113 hours (according to a local newspaper). During this period, the temperatures were zero or below. The coldest recorded temperature is about -30°F. The FDD accumulates fairly uniformly over about 100 days from early December to mid-March (Assel 1980). Assuming it was uniform, the average daily mean and maximum (extreme) temperatures are 24°F (32°F - 800/100) and 15.5°F (32 - 1,650/100). Experience indicates that an air flow rate of about 4 cubic feet per minute (cfm) per 100 feet of diffuser hose is reasonable for southern parts of the Great Lakes.

First, calculate the required heat transfer rate (q_w) from Equation 8.1. Assume the temperature of the water is fairly cold, say 32.5°F. Using Table 8.1, with Q_a = 4 cfm and a water depth of 8 feet, the heat transfer coefficient (h_b) is 178 BTUs per hour per square foot per degree Fahrenheit.

$$q_w = h_b (T_w - T_m)$$
$$= 178 (32.5 - 32)$$
$$= 89 \text{ BTUs per hour per square foot}$$

TABLE 8.2: Ice Equilibrium Thickness n_e (inches)
(after Ashton 1974)

Required Heat Transfer Rate q_w (BTUs/hr./ft.^{2})	Ambient Air Temperature T_a					
	20°F	10°F	0°F	-10°F	-20°F	-30°F
25	4	10	16+	16+	16+	16+
50	0	3	6	10	13	15
75	0	1	3	5	7	9
100	0	0	1	3	5	6
125	0	0	0	1	3	4
150	0	0	0	0	1	3
175	0	0	0	0	0	2
200	0	0	0	0	0	1

From Table 8.2, estimate the ice equilibrium thickness (n_e) at ambient air temperatures (T_a) of 24°F and 15.5°F with q_w = 89. At 24°F, n_e = 0, and at 15.5°F, n_e = 0 in. So, in this case an air flow rate of 4 cfm would melt out the ice.

Next, look at the cold snap. It didn't rise above 0°F for 113 hours, or about 5 days. Say the average air temperature was -10°F for this period; for 32.5°F water, the ice equilibrium thickness would then be 3-4 inches. This would be tolerable, as it wouldn't cause lifting or shoving.

Now assume a water temperature of only 32.25°F. Then q_w = 44, and n_e = 0 at 24°F, 3 inches at 15°F and 10+ inches at -10°F. Ten inches of ice in just 5 days is possible--an ice cover can grow more than an inch a day, and it grows fastest as it is first forming.

In this case, an air flow rate of 4 cfm may work--if the marina is attended and some ice chopping is done. It would also be a good idea to have a second compressor working in tandem with the first one and to increase the flow rate some.

A rule of thumb for estimating the quantity of air required is to use 3-4 cfm per 100 feet (0.0046 to 0.0062 m^3/min./100 m) in mild ice climates, and 5-6 cfm (0.0077 to 0.0093 m^3/min./100 m) in severe ice climates.

Line-Source Bubblers vs. Point-Source Bubblers

The preceding analysis is for a line-source bubbler, which typically has orifices spaced at a third to half the depth of the water. It is estimated that a series of point sources (such as in a bubbler system layout protecting

individual piles spaced throughout a marina) would approximate the line-source condition. This, of course, would be a function of how far apart the point sources become.

In general, a reasonable approximation for Q_a in a typical marina would be to add up the air discharges at each point source and divide by the total length of the diffuser—or to assume an orifice every 4-5 feet along the diffuser line and provide the cfm per 100 feet as designed. Theoretically, one bubble source at each piling is adequate because the bubble plume will completely surround the piling. However, it is not unusual to later have to add an orifice (which means cutting another slit in the diffuser line if unperforated vinyl tubing is used) because of a need for extra air, water currents, malfunctions, pressure losses, etc.

Ashton (1979) presents an analysis of point-source bubbler systems. This reference also includes FORTRAN computer programs for line-source and point-source systems.

Figure 8.2 shows line- and point-source bubblers in a marina. Note that the boats are in wet storage. Wet storage requires caution and a good understanding of ice conditions in the harbor. The fees received from wet storage of boats during the winter can exceed the costs of a deicing system and produce net income for the marina.

Lateral Melting

Keribar et al. (1978) discuss how the heat transfer coefficient at the ice-water interface varies with lateral distance from the bubbler axis, flow rate of air through the bubbler, and water depth. The analysis presented above is based on a heat transfer rate directly over the diffuser (at very small lateral displacement) and gives satisfactory results. Local melting near pilings is usually of more interest than predicting ice thicknesses away from the axis of a line-source bubbler.

Some say that bubbles will melt out a width nearly equal to the bubbler installation depth. I have observed that this is correct in mild-to-medium winter conditions and incorrect during cold periods, during which the area melted can shrink almost to nothing. Furthermore, regardless of the severity of the weather, the lateral melting is quite small if the water is very cold. I have measured ice 40 inches thick within 6 feet of the edge of a 2-foot-wide bubbler hole.

Design Principles for Compressors

Once the quantity of air required is known and the system pressure (described subsequently) has been estimated, you can select a compressor or compressors, design the air distribution manifold and diffuser lines, and incorporate controls and miscellaneous appurtenances.

A variety of compressors are available. A high volume of air at a low pressure is required for a marina bubbler system. Consequently, rotary vane compressors (to 15 psi maximum) and straight lobe positive displacement blowers (to 10 psi maximum) are used more today than the less energy-efficient



FIGURE 8.2: Compressed Air Ice Suppression System

high-pressure piston compressors (except for small compressors for limited melting jobs). The rotating compressors tend to make more noise, but since pistons are not involved, the problem of finely atomized oil (bypassed in the piston cylinders) entering the diffuser system and clogging the orifices is avoided.

The selection of air compression machinery will depend on the volume and pressure of air needed, the efficiencies and maintenance associated with the equipment, and the size of the deicing installation, among other factors. Machinery capable of continuous operation is required for most projects.

Two or more compressors can be interconnected for a central air distribution system appropriate for large layouts. A series of small compressors, each independently deicing a local area, can be used on smaller layouts (discussed later). The choice will involve an economic analysis. An interconnected compressor arrangement not only provides supplemental air when one machine can't handle the deicing load, but it also provides back-up capability when mechanical failure or routine maintenance shuts down the compressor. Maintenance is needed every two to three weeks.

Large installations of a hundred or so slips need either a backup power supply, or an emergency operating plan and procedure. Knowing the location of local construction compressors that quickly can be pressed into service might mean the difference between a lot of ice damage and very little.

If the system back-up is not automatic, it will involve people. These people must be present and attentive--or you really have no back-up at all. During some bitter cold weather between Christmas and New Year's Day, the compressor at a large marina failed, and no one noticed it for four days. Then it was too late--lifting had begun and docks were damaged.

Design Principles for Manifold Distribution Systems

Air contains water vapor, and when air is compressed, its volume decreases. If the temperature remains the same, the smaller volume cannot hold as much moisture and the vapor in the air condenses. This condensation can freeze as the compressed air enters the manifold system.

This can be handled several ways. One Norwegian system sequences the air--the air goes from a surge chamber (small pneumatic tank) to an outside fin cooler to dismiss oil vapor and water to dry the air, and then back inside to an air filter and alcohol injector before finally going out into a 1-inch marina distribution system.

The preferred method in the Great Lakes region is to connect the distribution manifold piping to the compressor discharge with a 15- to 20-foot rubber leader hose. The manifold piping should go under the ice and onto the lake bottom in the shortest distance possible. The rubber leader hose is not only flexible, but it loses less heat than galvanized iron or PVC pipe. Also consider that the compressed air system may be turned on in the summer for maintenance, and a PVC connection may be overheated by the air warmed 100°F above the ambient temperature.

The manifold system, which delivers air to the diffuser lines, can be a simple straight run, or a complex network of piping valved and balanced throughout. Manifolding is usually 1.5- to 3-inch pipe to keep pressure losses under a pound per square inch.

Tables are available in which the pressure drop per foot in compressed air lines is expressed as a function of discharge rate and pressure, and pipe size. For example, a 100 cfm air flow at 10 psi pressure in a 3-inch diameter pipe will lose about 1 psi in 2,500 feet; in a 2-inch pipe, 1 psi of pressure is lost in about 300 feet; in a 1.5-inch pipe, about 100 feet.

Manifolds are usually galvanized pipe or plastic PVC Schedule 40 pipe. The plastic pipe can be weighted down by strapping and banding steel reinforcing bars along its length. If the manifold is placed above grade, freezing condensate may be a problem. The air can be dried first, or oversized and pitched manifolds can be used. The manifolds also can be insulated.

Some marinas are using winter-drained, small potable water lines on the docks to distribute compressed air. Large air pressures are needed because of friction losses, and condensation can be a problem. In such cases, heat tapes and insulation have been used on the piping to help prevent condensation.

Design Principles for Diffuser Lines

Air diffuser lines connect to the manifold distribution system. Sometimes smaller PVC pipes with small drilled holes are used. Another type of diffuser line is the popular 3-tube polymerized extruded PVC hose. The three adjoined tubes are each about 5/16 of an inch. The center tube contains a quarter-inch lead bead, which provides ballast and stiffness. One side tube is perforated with quarter-inch slits every 44 inches. The other side tube may also be slit, but usually is not. This tube conducts air from the feed end of the hose to the far end, where it can enter the perforated side and come back. The effect is to reduce the length of the diffuser hose as it is fed from both ends. Nonperforated hoses are available that can be slit to suit specific needs.

The slits--which close when the pressure is turned off--reduce the amount of clogging caused by silt, sand grains and aquatic life. If the system has been shut down, it will be necessary on restarting to purge the diffuser lines of the water that has entered the slits. This can be a difficult task on long lines and at deep spots in the marina. This trapped water must be expelled through the slits, or blown out the end of the diffuser if it has been capped and can be brought to the surface. Purge valves are also available that permit the water to be automatically eliminated without lifting the lines to remove the caps.

The recommended lengths of diffuser lines vary from 75 to 300 feet, depending on how they are fed and on the severity of the local climate. Usual lengths range from 100 to 150 feet.

The diffuser lines are laid uphill from the manifold connection--otherwise much of the air will escape in shallower waters and the deeper orifices will be starved. Remember, about 4.3 psi of air pressure is needed to balance the water pressure at a depth of 10 feet.

Estimating System Pressure Required

Assume the deepest water for the diffuser lines is 10 feet. The estimated air pressure required to overcome the water head is about 4 psi--1 or 2 psi at most to open the slits, plus 1 or 2 psi in the manifold distribution system. Another psi might be needed to handle high water or wind setup. In this case, a 6 to 9 psi discharge pressure would be required, so a blower with at least 6 psi is needed, and it would be preferable to use an 8-10 psi blower. Obtaining the total volume of air desired at an economical horsepower may dictate the choice of an 8 psi over a 10 psi blower, unless you need more room for error. On the other hand, the needed air capacity could also be provided by combining two 8 psi blowers, which might be interconnected through the manifold piping system to give better air balance and duplicity in operation.

System Controls

The operation of the compressor(s) can be controlled with a simple on-off manual switch, a duplex cycling control with thermostats, or even a microprocessor control programmed for given responses. A large Danish marina uses a microprocessor to control the amount of air pumped to each head pier. In a

5 to 6 cfm environment, I believe continuous operation is necessary. Shutting down the system for a few hours or a few days may save a little energy, but it may also result in start-up and other problems. Some intermittent shutdowns may be appropriate at the beginning and ending of a deicing season, but not in the dead of winter. In a milder climate, automatic controls may be used.

Operating Costs

Operating costs obviously will vary widely because of differing weather and water temperatures, utility rates, equipment specified, etc. Annual energy consumption at two northern Michigan marinas with high-pressure piston compressors ranged from 21,000 kilowatt-hours (kwh) to 68,500 kwh for the 70-slip facility, and averaged 37,000 kwh annually for the 102-slip facility. A Lake Erie marina that deices several hundred piers has reported an average consumption of 50,000 kwh annually.

Projected operating costs can be estimated from system parameters. Keep in mind that not every winter is as cold as the winter of record and that winter doesn't last forever--only 100 days or so.

Installation, Maintenance and Monitoring

A compressed air deicing system requires careful installation, maintenance and monitoring. In mid-winter, with several feet of ice in the harbor and subzero temperatures, failure of the deicing system can present insurmountable problems.

The system should be checked and repaired each spring, when the harbor water is clear and bottom sediments are not in suspension. Most often, bottom conditions can be inspected easily. Watch for changes in the bottom contours due to currents and sediments. The system should be operated again in the fall and repaired if damaged during the boating season.

Some operators run their systems at times during the summer to prevent it from being clogged with silt. Operators of large systems generally prefer to do all maintenance work in the fall. Regardless of the approach you take, careful maintenance is essential.

Small Compressed-Air Deicing Systems

Another frequently used deicing system consists of half-horsepower oil-less electric air compressors connected to 250-foot lengths of half-inch polyethene aeration tubing. The compressors are plugged into power receptacles on the marina pier, and the tubing is dropped around the pilings or the area to be deiced. A number of the compressors are placed throughout the small-craft harbor. The tubing is lead-weighted and has die-formed check valves at 2-foot centers. These valves open under 2 to 4 psi pressure and emit bubbles.

Compressed Air Systems in Saltwater, Brackish Water and Rivers

Compressed air deicing will also work in harbors that are not freshwater. In fact, it is said that there are more compressed air installations in saltwater and brackish water than in freshwater. The preceding analysis theoretically is only for freshwater, but it apparently also approximates saltwater situations. Wherever you can get water to rub ice, the ice will

melt. The smaller the temperature differential between the ice and the water, the larger the quantity of air and velocity of air-driven water required. I have been told of coastal areas that were deiced at depths of only 3 feet and in which the diffuser lines were exposed at low tide--obviously, the water was warm and circulating.

Rivers are another story. Suppression systems in ice-covered rivers are usually ineffective, as the river currents destroy or displace bubble patterns. Thermal mixing of river water is a natural process. A river that has formed an ice cover has already dissipated nearly all its heat; otherwise, the cover would not form. Put another way, a bubble plume has an ascending velocity of about a foot per second, and it deflects laterally upon hitting the ice and moves off at a lesser velocity. If the river flow already rubbing the bottom of the ice is 1-2 feet per second, a bubbler won't be of any help.

Historical Note

A manufactured vinyl diffuser tube is perforated every 44 inches. This is due neither to scientific design nor some conversion from another system of units. Years ago, when the manufacturer was first experimenting in the business, it decided to heat the hose before slitting it--and a 4-foot center-to-center slit pattern cools to a 44-inch pattern.

FLOW DEVELOPER DEICING SYSTEMS

The functioning of flow developers is based on the thermal reserve of water and surface currents, which prevent freezing at velocities above 2 feet per second (Eranti et al. 1983). The devices also are called propeller systems, velocity systems, water agitators, water fans and, simply, deicers.

Flow developers most often consist of one-third, one-half or three-quarters horsepower submersible electric motors placed 2 to 5 feet into the water. They are suspended on slings underneath wet-stored boats, on drop pipes attached to marina piers, or on floats that follow tides or support the motors in areas of the harbor where there are no piers.

The motor drives a propeller which can be aligned to produce a vertical flow pattern that spreads out and opens a 50-foot area, or tilted so that an inclined flow opens a rectangular area about 20 feet by 100 feet (figures are according to manufacturers' product literature).

The flow developer moves a lot of water, but only a small portion of it actually melts the ice where suppression is desired. To my knowledge, no methods of analysis for these systems have been published, so experience will be the best guide (unless perhaps you wish to develop the theory for design, based on fluid mechanics and thermodynamics).

Eranti et al. (1983) report that success with flow developers varied widely in Finnish harbors. Their performance depended on local water temperatures and ice conditions. The method was successful with water 1/2°F or more above the freezing point (which varies with salinity; in Finland, the water is about 1 percent salt), and very successful if warm water discharges could be used with the flow developers. They also observed melted-out areas ranging from 35 to 350 square feet for each rated kilowatt of flow developer.

Since the flow developer produces water motion, it prevents freezing in fresh, salt or brackish waters, warm or cold. In cold water, it appears that more ice suppression can be obtained with less energy by using compressed air and natural bubble bouyancy.

My observations and measurements in Great Lakes small-craft harbors indicate that flow developer systems work well in areas where the water is warm (i.e., 33°F to 36°F)--in the "downstream shadow" of a power plant, for example, or in a warm body of water connected to a Great Lake. One marina on such a connecting lake melts lots of ice with a couple of wind-driven shafts that turn large propeller blades.

OTHER DEICING AND SUPPRESSION SYSTEMS

If wind can melt ice, so can groundwater. I know of a marina that uses well water discharged by ordinary garden hoses submerged below the ice sheet and aimed up at its underside. The heat and motion of the well water melts ice, but when the lake level is quite low, the water jet squirts out of a small hole in the ice, the water freezing on everything nearby. Conversely, when the lake is high, the water jet lacks the "oomph" to reach the ice. I don't recommend this system, unless better details can be worked out.

In a somewhat similar effort, winter navigation researchers discharged warm water near the bottom of an ice-covered harbor to see if the water would rise and melt the cover. They report that 57°F water discharged 19 feet below the ice cover rose until it cooled to 39°F, at which point it was denser than the surrounding water and descended. The depth and temperature of the water are significant factors in this approach, ones that must be analyzed carefully.

Anderson (1972) has suggested installing wave machines in the back of a harbor to deice it. The waves would prevent solid ice covers from forming in the harbor and move the fragmented slush ice out of the harbor into deeper water, where incomplete ice cover and currents would remove it. However, this method --along with using chemicals, artificial heaters, and excavating and conveying systems--doesn't appear to be feasible in small-craft harbors. In fact, they have yet to prove useful in large shipping harbors.

In a few instances, insulation has been successfully used to suppress ice growth in rivers and lakes. At least an inch, usually two or more inches of insulation are required, depending on the location. The mechanical problems of placing blankets of insulation on ice covers and melted-out water surfaces have not yet been solved. The insulation has to stay put under wind and wave conditions and also be retrievable at the end of the season. Blizzard winds hit Great Lakes harbors at times, and light insulation is quite difficult to hold down.

ICE CONTROL METHODS

Work is currently being done on developing ice control methods, in the sense that if you cannot suppress ice's growth, you can try to control its effects. Frankenstein and Hanamoto (1983) discuss control methods that may be applicable in some small-craft harbors.

High Flow Air Screens

High flow air screens have been used to keep floating ice away from docking facilities and out of navigation locks. The rising air flow entrains and accelerates a large volume of water, which deflects laterally upon reaching the surface. The resulting surface current is sufficient to prevent ice from moving across or being carried across the air screen by the wake of a passing vessel.

One successful air screen was placed across the 30-foot deep navigation approach to one of the Sault St. Marie locks. A 1,100 cfm compressor operating at 100 psi supplied air to a 2.5-inch manifold and supply line system that had nozzles 0.4 inches in diameter spaced 10 feet apart. Each nozzle flow was about 60 cfm. No air expansion freezing problems were experienced, and the dead-end manifold pressure was about 60 psi. The water surface rise was nearly a foot, and ice could not pass across this hump.

Air screens can also be used during open water seasons for debris control. Mascha and Christensen (1983) report on low pressure air curtains for environmental protection of surface waters.

An air screen may have application in a marina during certain periods. Some design assistance is given in the cited references and in U.S. Army Corps of Engineers (1982).

Ice Booms

Originally, ice booms were used on rivers to prevent or reduce frazil ice. Ice booms are now being used in other ways, such as holding broken ice pieces in place and shielding piers and docks from moving ice. A boom may be made by connecting timbers 1 x 2 x 20 feet in size. Sometimes the timbers are doubled. Some information on design (largely empirical) is contained in U.S. Army Corps of Engineers (1982). The loads on booms may range from 500 to 5,000 pounds per foot. Booms specially set out for the winter could aid in stabilizing a harbor ice cover. Booms can also help with moving ice.

Other Ice Control Methods

Flow developers, akin to air screens, can be aimed to control moving ice. Deflector structures, like pile clusters, are also used in lakes and rivers where the direction of ice movement is predictable. Since they deflect ice and guide it past the facility being protected, the forces on them are less than those of an ice retaining structure. Yet some pile clusters are substantial--6 to 12 large timber piles tied together, spaced 30 feet on centers.

Docks also have been protected by partial encirclement with styrofoam-filled rubber tires tied together to form a combination boom-deflector. These floating tires are a plane of weakness in the ice cover and are believed to reduce lateral and vertical forces.

More work needs to be done with these methods to control ice. The success of such methods will depend on the ingenuity of the designer and the willingness of the marina owner to accept some of the risks attendant with new schemes.

9. Design Methods for Floating Structures

Floating structures are economical, and in many parts of the Great Lakes their use an effective design choice. Fifteen years ago, I thought the same, when as a consulting engineer I designed some floating docks for a Lake Superior marina (Wortley 1972).

This chapter begins with a discussion of that marina project and ice pressure, then reports on field experiments with floating docks, concluding with design recommendations for floating structures.

INTRODUCTION TO BESET SQUEEZE PRESSURES

My Lake Superior project's marina owner had specified floating docks for three reasons: (1) the docks would maintain constant freeboard (the distance between water and deck) despite fluctuating lake levels, (2) workers could build the dock system on land and float it in early summer and (3) only a small number of piles would be required for lateral internal stability as compared with a dock structurally supported throughout its entire length.

The flotation selected for the dock system was hollow (not filled with foam) molded reinforced fiberglass pontoons. The pontoons reportedly had survived ice in two other northern harbors, were successfully tested at the project site the year before in a limited winter experiment, were drafted 5 degrees on the sides (bathtub-shaped) to enable them to pop out of the ice on freezing, and the manufacturer claimed they were able to withstand the -30°F weather and 3-foot ice conditions of the site.

Ice pressure destroyed half the fiberglass pontoons after just one winter in the ice (Figure 9.1). We subsequently found out that the fiberglass pontoons had never before been used in such circumstances.

Workmen refloated the dock with new fiberglass pontoons just in time for the summer boating season. We noticed that the pontoons had been shipped to the site in a nestled stack and thought that, if an undamaged pontoon were submerged under a damaged pontoon, a new shell could be drawn up tightly around it by pumping the water out of the old pontoon. It worked--the dock was simply refloated.

This refloated dock was protected the following winter with a compressed air deicing system. The system was sized to deliver 6 cfm per 100 feet of perforated hose. This worked well. The next year, the "repaired" dock pontoons were field-foamed with polyurethane and now, since they are more rigid, are surviving without deicing in this harbor, which has quiet, but thick, cold ice.

I still don't know why these pontoons cracked and ruptured--other than they simply weren't strong enough to withstand the ice pressures. I speculate that the fiberglass was not uniform in its chemistry and in dimension, especially at the corners of the tubs.

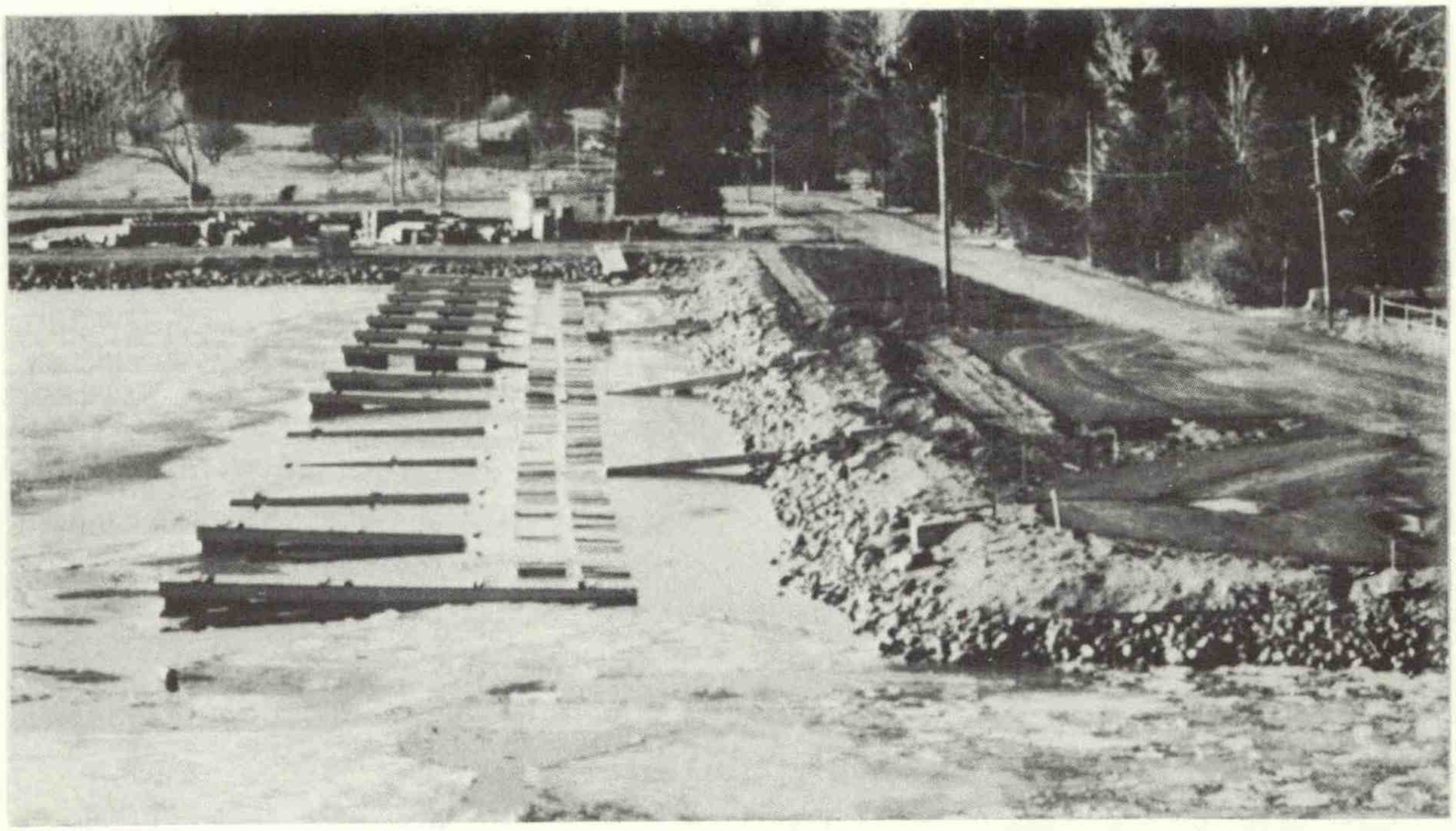


FIGURE 9.1: Hollow (Unfilled) Fiberglass Pontoon System Damaged by Ice

However, I did observe that when there was no snow cover, the ice melted during the afternoon on the sunny side of the tubs, which were green-colored. The absorbed heat melted the ice sheet next to the tubs to a depth of 1 to 2 inches. The surface of this melt water froze rapidly as soon as the sun set. Perhaps as the encased melt water froze, it expanded in the confined space along the side of the pontoon, squeezing a little more each day until the fiberglass gave way.

If this refreeze-squeeze hypothesis is correct, then hollow floats won't work, and perhaps even foam-filled floats won't either. But several marinas in the Great Lakes use hollow black-polyethylene rectangular tubs, and these are not crushed--apparently because their sides are so long that they flex and deform to accept whatever pressures develop.

It is of no consolation to know that the U.S. ice breaker Polar Star was designed for beset ("to attack on all sides") squeeze ice pressure of 600 psi, a value thought realistic for an ice breaker trapped in ice, or to recall that the USSR had number of Siberian vessels beset in ice in the fall of 1983. But Figure 9.2 may give some comfort--and some puzzlement.

Figure 9.2 shows a pail of frozen water containing three glass objects--an ordinary round light bulb, a tubular light bulb and an 8-ounce sample jar, each weighted to float vertically and partly submerged. The pail was set outside in below-freezing weather. Restrained by the sides of the pail, the ice bulged upward. Under this biaxial state of stress, the ice did not damage the glass objects. Other tests in this series of backyard experiments (unpublished) included other pontoon-like shapes--round thin metal jello molds, round plastic margarine tubs, clear plastic rectangular butter dish tops, and rectangular tinfoil baking pans, some of which were painted black on the outside. All were partly filled with sand to represent the load on a pontoon.



FIGURE 9.2: Glass Objects Subjected to Ice Pressure Squeeze

The ice did not rupture any of the objects, but it did deform the tinfoil tubs (both the silver and the black). Their sides were dented in and their bottoms pushed up. All of the objects migrated part way across the pail due to the ice melting on one side.

These tests did not produce any quantitative results, but qualitatively got me hooked on ice. Some years later, a somewhat similar series of tests were run in a Lake Superior harbor with real pontoon units. The results have since been published (Wortley 1981) and are summarized in the next section.

DOCK FLOATS SUBJECTED TO ICE

Full-scale field testing and observation of floating docks placed year round in the Bayfield, Wis., harbor were conducted during 1977-81. I invited floating dock manufacturers to furnish and erect a standard dock module (or a variation of their standard) for my observation during the test period.

Four manufacturers participated: Bero Corporation; Mecco Marinas, Inc.; Rotocast Flotation Products, Inc., and United McGill Flotation Systems. Figure 9.3 shows some of the docks used in the tests.

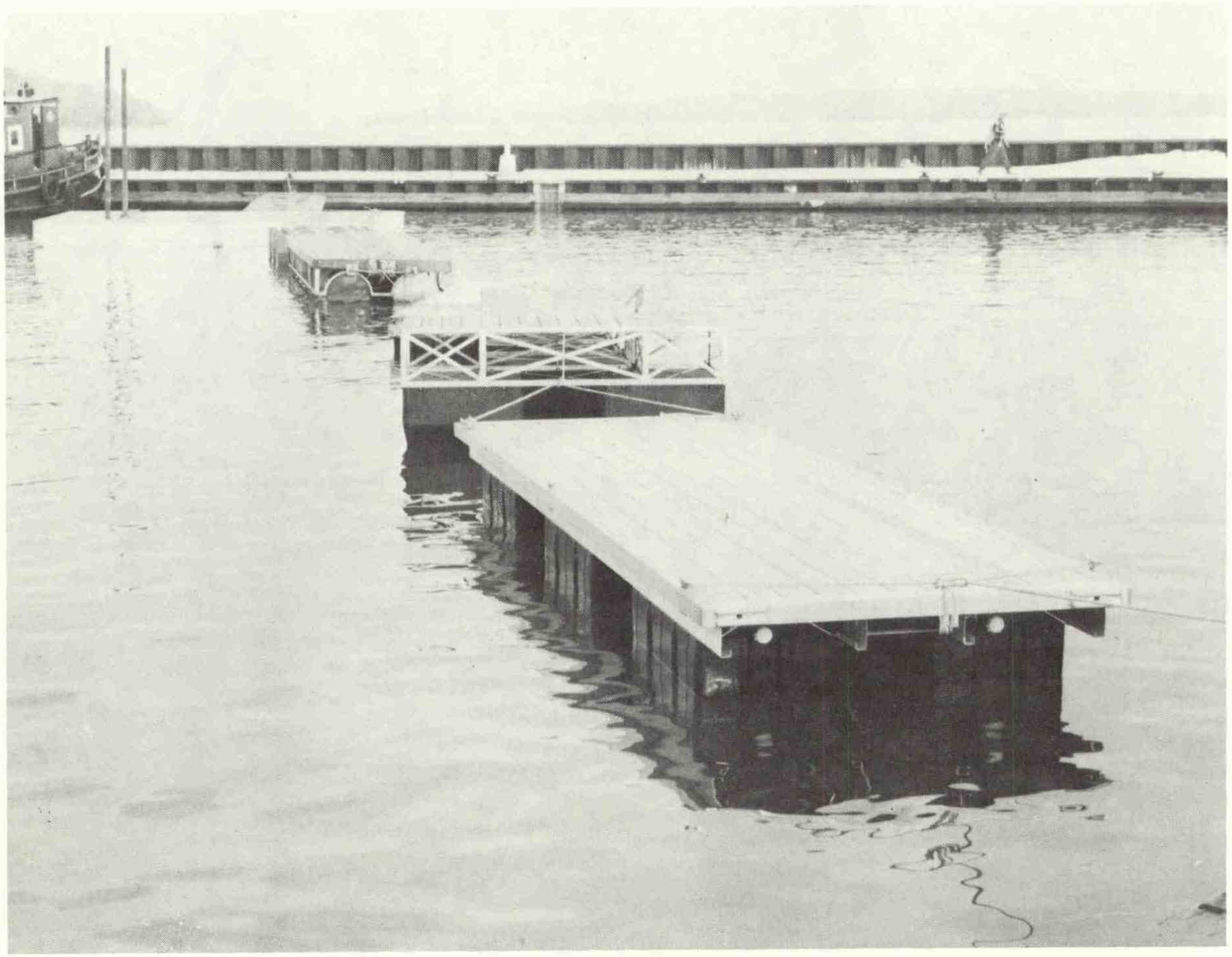


FIGURE 9.3: Floating Docks in Bayfield, Wis., Harbor Test

The floating docks were placed in a public harbor protected by breakwaters and not subject to ice floes or movements. Actual movements of the stable ice cover averaged 2 inches or less in any lateral direction, and a drop of 8 inches during the course of the winter. The water around the docks was 12 to 14 feet deep and varied in temperature from 32°F to 33°F, averaging 32 1/4°F. The test area was unshaded and exposed to wind and snow.

The 4-year test period included both an unusually severe and a mild winter. The maximum winter ice thickness measured was 30 inches, the norm about 20 inches. The site was frequently snow-covered, the average depth about 6 inches. At times the site was free of snow; at other times the docks were completely buried with only a trace outline visible.

Workers assembled the docks and placed them in the water the first autumn. They were free of restraints and located away from the cracking zone around the city pier. The flotation immersion depths ranged from 4 to 10 inches, and the deck freeboard heights above the water ranged from 18 to 36 inches. The tests did not replicate a complete marina dock system with many pier modules joined together. (However, other observations of complete floating dock systems in Great Lakes harbors have addressed this important point, and I comment on it later.) The docks at Bayfield were observed and inspected as separate, free-floating elements.

With a barge-mounted crane, the docks were plucked from the lake and carefully inspected both before freeze-up and then again after spring melt-out. In the summer, the docks were towed to other areas around Bayfield where they were used by boaters and swimmers. In the fall, they were towed back to the test site.

All the dock modules tested were either foam-filled pontoons with polyethylene or metal encasements, or hollow-core expanded polystyrene, covered with a 15-mil coating of polyurethane and reinforced with metal truss work. Other products requested but not furnished for the tests included hollow pontoons made from concrete, metal and polyethylene.

The wall thicknesses of the test products were 3/64-inch, 3/16-inch and 1/4-inch polyethylene; 20-gauge (0.039 inches minimum) galvanized steel with 1/4-inch-high ribs on 2-inch centers; 0.04-inch aluminum with 1/4-inch-high ribs on 2-inch centers; and 4-inch expanded polystyrene.

No major damage or failure of any of the pontoons occurred. Some minor damages or deformations were observed. Ice dimpled and pinched polyethylene pontoon corners where some of the pontoons were not completely filled with foam and were deficient in wall thickness, and ice slightly creased the polystyrene at the water line.

Melting of the ice next to the pontoons sometimes was observed on sunny winter days when the docks were not covered with snow. This melting occurred next to colored pontoons as well as next to the aluminum, galvanized steel and white expanded-polystyrene pontoons. The melted area was about a half-inch wide by 1-inch deep.

Some pontoons were drawn down about 4 inches into the ice sheet. This was occasional; it was not observed on all docks or during all winters. Round as well as side-tapered pontoons went down; they did not pop up and ride out of the ice.

These limited tests indicated that the encasement material and its color, the size and shape of the pontoon, and the immersion depth into the water (and ice) appear unimportant. The validity of these points is supported by other observations of Great Lakes floating dock systems.

DESIGN RECOMMENDATIONS FOR FLOATING DOCKS

The most dangerous condition for floating docks is shifting and moving ice. Floating docks placed in harbors that experience shifting and moving ice should be moved to safety during the winter, or lifted out of the ice, or deiced in place--or not used at all.

Thermal ice pressures are another concern. There are no measurements of these pressures on pontoons, but it appears that foam-filled pontoons and thick-walled pontoons adequately resist thermal pressure squeeze. In addition to squeeze, thermal stresses can exert lateral shoves on dock floats if nonuniform conditions exist--for example, if there is thin ice or open water on one side of a dock. Thermal stresses will force the dock towards this weakened zone.

Icing and spray can attack a dock in the water, before it freezes in for the winter. The ice build-up can twist, submerge and deflect a floating structure. When it becomes frozen in, it will not be floating level as you may envision, but rather contorted and possibly vulnerable to ice pressures and damages. Very few simple design solutions exist for the problems.

Materials and Types of Docks

The use of floating docks in Great Lakes harbors is increasing. This is quite appropriate, because floating docks solve lake level fluctuation problems. Many good manufacturers and builders of floating docks with relevant construction experience are available. Be wary, however, of claims that assert the dock system performs satisfactorily in ice conditions, unless the referenced ice conditions are representative of your site.

Ice conditions in the Northwest or Northeast U.S. seaboards, in Alaska, in the Scandinavian Baltic Sea and in reservoirs may or may not be what is faced in the Great Lakes. My guess is that they probably won't be the same, so proceed with caution--but do proceed, as performance data on new products is always needed. Floating dock manufacturers and builders, working with designers and owners, can continue the progress now being made in Great Lakes dock building.

Many materials and types of docks now are being used successfully in ice. They include styrofoam, polystyrene or urethane billets, sometimes encased in wood; heavy-wall metal pipe; foam-filled corrugated galvanized iron, aluminum and polyethylene pipe shells; foam-filled fiberglass and polyethylene tubs, and foamed discarded rubber tires. This list is probably incomplete.

Other unfilled floating docks that have been used in ice (sometimes removed or deiced) with inconclusive results are polyethylene pontoons, oil drums, concrete pontoons and olive oil casks. (Olive oil casks? Yes--one marina got a deal on used Greek olive oil casks. With a jig and press, and without cracking the plastic material, the roly-poly casks were squashed into a more stable rectangular billet shape. They were then pushed into the underside of a timber dock frame and used for flotation.)

Concrete pontoons--some hollow, but frequently poured around a styrofoam block as an inner form--are used extensively in Scandinavia, apparently with good results. Only a few floating concrete docks exist at this time in the Great Lakes.

One harbor has 150 concrete docks--at the bottom of the harbor. They were made of hollow post-tensioned precast concrete and sank due to water entering inspection holes and shell cracks. At another facility, the thin concrete walls cracked away from the styrofoam core of foam-filled concrete docks. Such egg-shell thin concrete encapsulation will probably prove unsuccessful. More experience with well-designed and built concrete pontoons is needed. They offer an attractive and durable alternative to the present kinds of floating systems being used.

Polyethylene encasements experience some damage, especially if they are not fully supported by the foam inside. Material less than 3/16 inch may tear, and it appears that 3/16 to 1/4 inch should be the minimum thickness specified. Ice squeeze has popped staples attaching polyethylene lids to tubs.

Lateral Restraint of Docks

Just as there are a variety of floating docks, there are a variety of ways to laterally restrain the docks. The restraint methods customarily are selected and designed for service loads of wind, current and boat impact. These restraints are not designed for ice conditions and should not be. Floating docks left in the ice should be left unrestrained.

The usual types of dock restraints are cables and anchors, shore struts, and pilings and spuds. Cables, chains and anchors hold the dock in position by attachment to the bottom or shoreline; shore struts are stiff and hold the dock away from the shore while keeping it properly positioned; dock pilings are driven into the bottom and develop lateral resistance from the soil, and spuds are usually dropped to the bottom and penetrate only a few feet.

The pilings can be sized and spaced so that a penetration sufficient to prevent uplift is achieved while meeting the penetration requirement to resist large lateral forces. A dock restrained in this manner must have sufficient strength itself to transmit the forces to the widely spaced pilings. Smaller, more closely spaced piles are used more frequently.

The spud system drops a metal pipe spud through a retainer piece of pipe attached to the side or end of the dock. With a number of spuds, a kind of "raking" resistance along the bottom is produced that keeps the docks in place.

Plenty can go wrong with these lateral restraint systems if the docks are not free to move independently with the ice with respect to the fixed restraint. Figure 9.4 shows a new floating dock in ice. The restraint system consists of anchors and cables with winches on the piers. The dock is well sheltered by an inner harbor structure. There should be no problems with this unit, provided the cables have been properly adjusted.

In the spring and occasionally during the winter, harbor water and ice will rise. If restraint cables are taut, they will snap or pull loose from the dock or the anchor, or the anchor will be displaced. In one marina, some of the cables snapped and pulled loose. This went unnoticed, and later in the spring, the pier was damaged in a storm because it was no longer adequately restrained.

In Sweden, heavy anchor chains connecting concrete docks to concrete shore anchors have been pulled out of the docks by the force of ice expanding between the shore anchor block and the end of the first float. Shifting ice has cut dock anchor lines. Hollow docks have sunk and others have been damaged as spring ice levels rose and anchoring cables did not fail. The docks were pulled down (held down) into the ice and water. Docks strutted to shore have caused shore anchor blocks to rotate and adjacent sidewalks to buckle.

Piles and spuds are not without their problems, too. Figures 9.5 and 9.6 show damages that have occurred with these types of restraint.

The dock shown in Figure 9.5 was damaged because the wood piling was lifting and generating pieces of rubble broken from the ice sheet. Made of exposed styrofoam under a wood deck frame, the dock was trying to ride out the winter, moving up and down with the ice. But the differential motion between the two



FIGURE 9.4: Cable-Anchored Floating Dock in Protected Harbor

caused the problem shown. The timber deck stringers split, and in other locations along the dock (not shown), the styrofoam billets split apart or pulled out of their timber frames. The piling shown is not even attached to the dock; it simply is nearby. Hoops and retaining rings around pilings cause similar if not worse problems. The corrective action taken for the dock shown in Figure 9.5 was to move it 5 feet away from its restraining pilings. This can easily be done if the piles are placed only on one side of the dock.

The spuds holding the dock in Figure 9.6 were to be lifted in the fall so the new floating dock would be unrestrained during the winter. Note that two of the three spuds on the finger pier in the background have been lifted while the others have not. The lifted spuds were either pulled up by the marina attendants as required, or lifted by the ice.

The damage to this dock resulted from the ice jacking the spuds, which are bound up in their spud well retainers at the end of the finger piers. The fingers have lifted up with respect to the head pier. The result was a

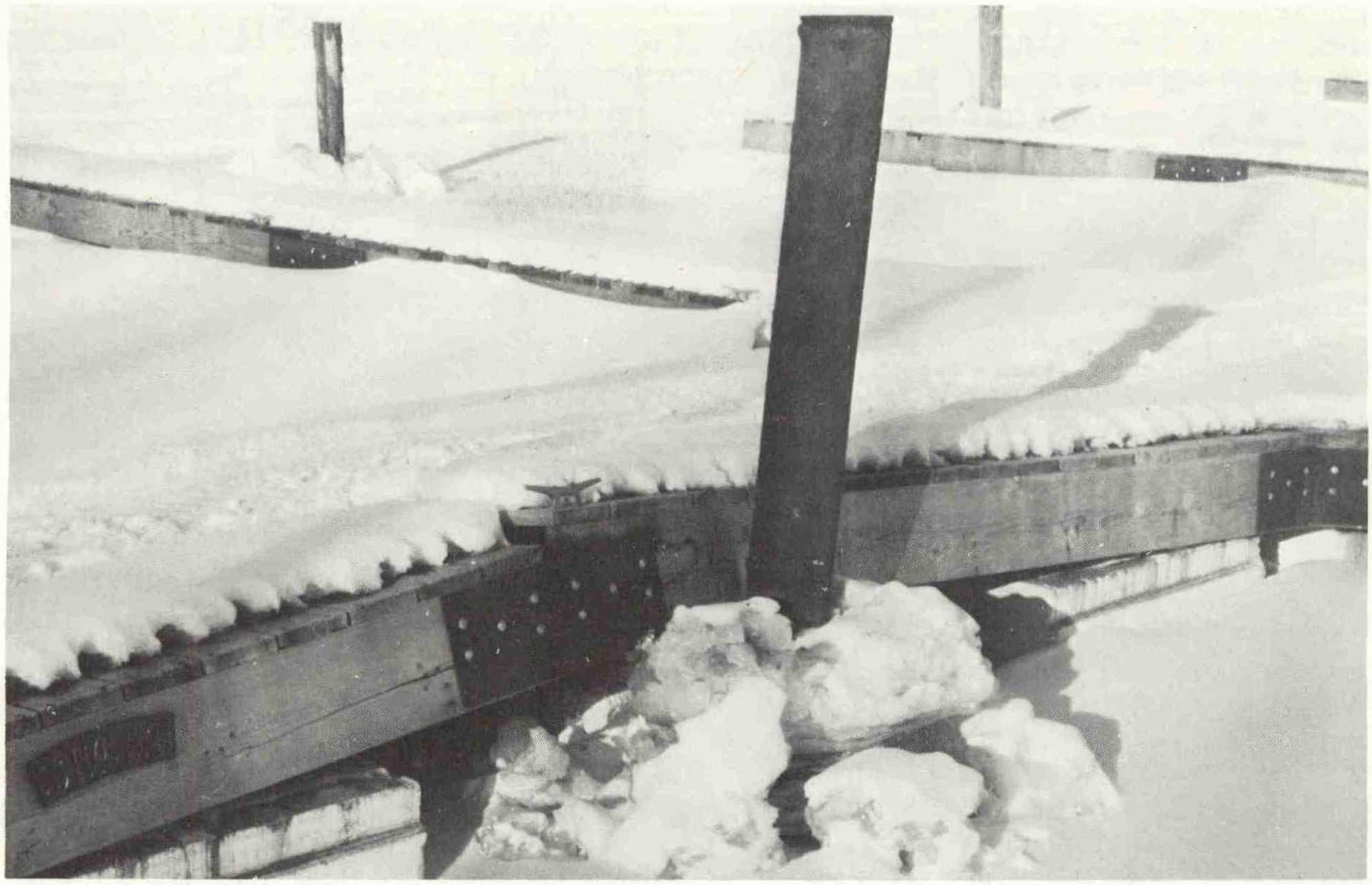


FIGURE 9.5: Floating Dock Damaged by Lifting Pile



FIGURE 9.6: Floating Dock Damaged by Lifting Spuds

breaking of the connection between the two, splitting of the finger stringers and ripping of the galvanized sheet metal bottom protection for the wood-encased styrofoam billets. All in all, a bad outcome for a new pier.

Joints and Articulation of Docks

If a dock has been successfully "freed" of restraints and dock components that won't rupture are used, can anything else go wrong?

Yes, a floating dock in ice may try to contort, and differences in elevation along the dock may vary by a foot, sometimes by 2 feet. Completely rigid piers are not functional and are not used. Articulated connecting dock modules are used, and they may kink.

Docks are constructed with "through-fingers" or "through-head piers." Which of these is best for ice is not apparent, but it probably doesn't matter.

Figure 9.7 shows an older type, kinky floating dock that has too many joints. Figure 9.8 shows a new level dock with finger joints and occasional head pier joints.



FIGURE 9.7: Floating Dock Contorted by Ice

Floating Tire Breakwaters (FTBs)

Bishop (1980) notes that the question arises as to whether or not a floating tire breakwater (FTB) has to be removed from the water during the winter. Dynamic ice forces can be very large and could easily exceed the restraining capacity of the FTB mooring system designed for wave forces. At sites with ice floes or wind-driven ice, therefore, it is recommended that the FTB be moved to a sheltered location during the winter or removed completely from the water. Storing an FTB in an exposed location in shallow water or at a beach for winter safekeeping is to be avoided, however. In such locations, the tires could fill with sediment if subjected to wave attack. FTBs that are designed correctly for wave forces can withstand thermal ice forces. Figure 9.9 shows an FTB wintering in ice.

Mooring Arms vs. Finger Piers

Some floating dock ice problems could be eliminated by a change in design concepts--by replacing floating finger piers with mooring arms for each boat slip. Figure 9.10 shows a floating concrete head pier with one type of mooring arm.

Many types of mooring arms are used in Scandinavia. Some cantilever from the head pier (usually they are a pole or member through the head pier acting as an arm for each side). Some connect to the head pier with decked-over

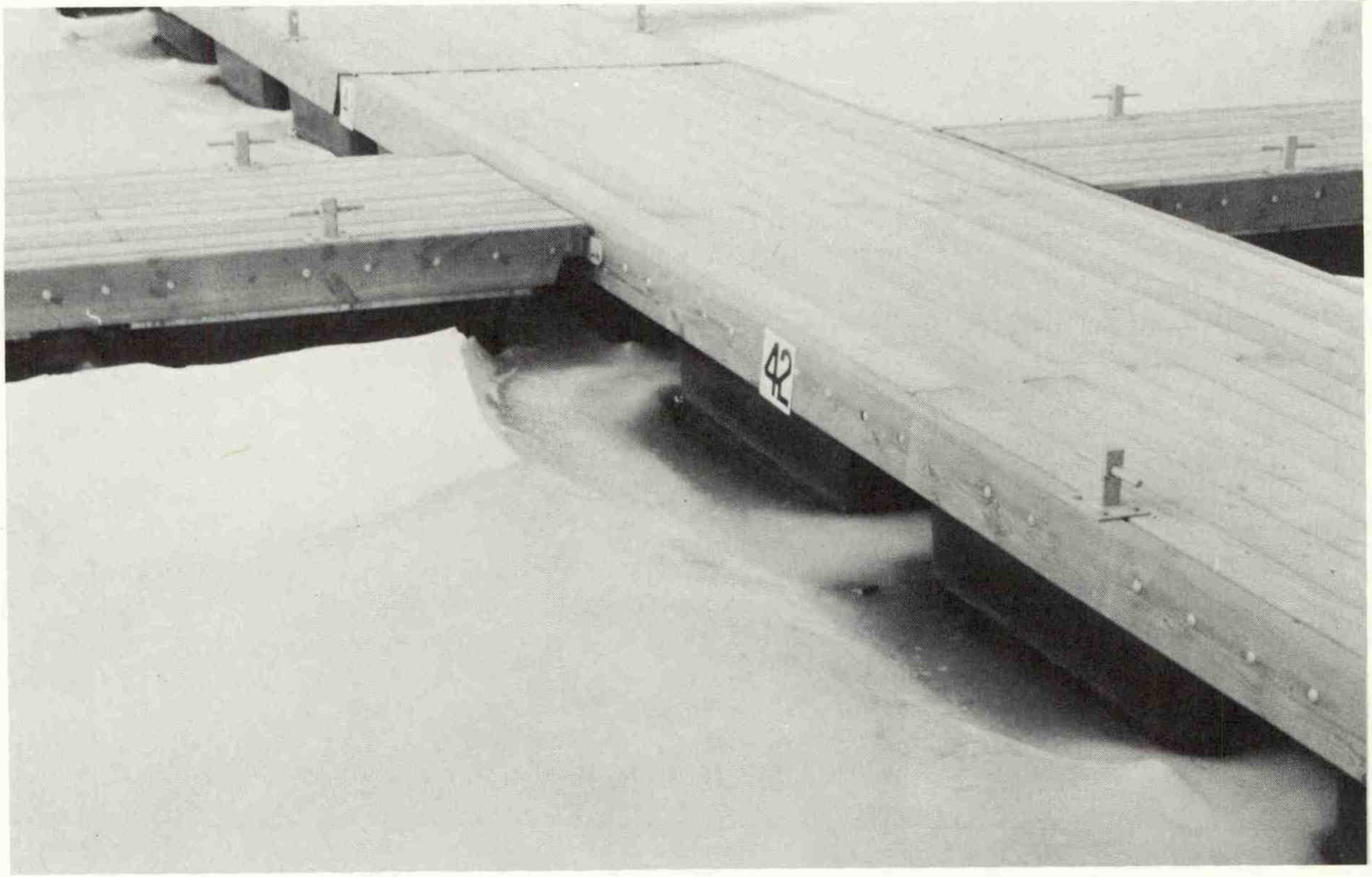


FIGURE 9.8: Floating Dock Level in Ice



FIGURE 9.9: Floating Tire Breakwater (FTB) Wintering in Ice

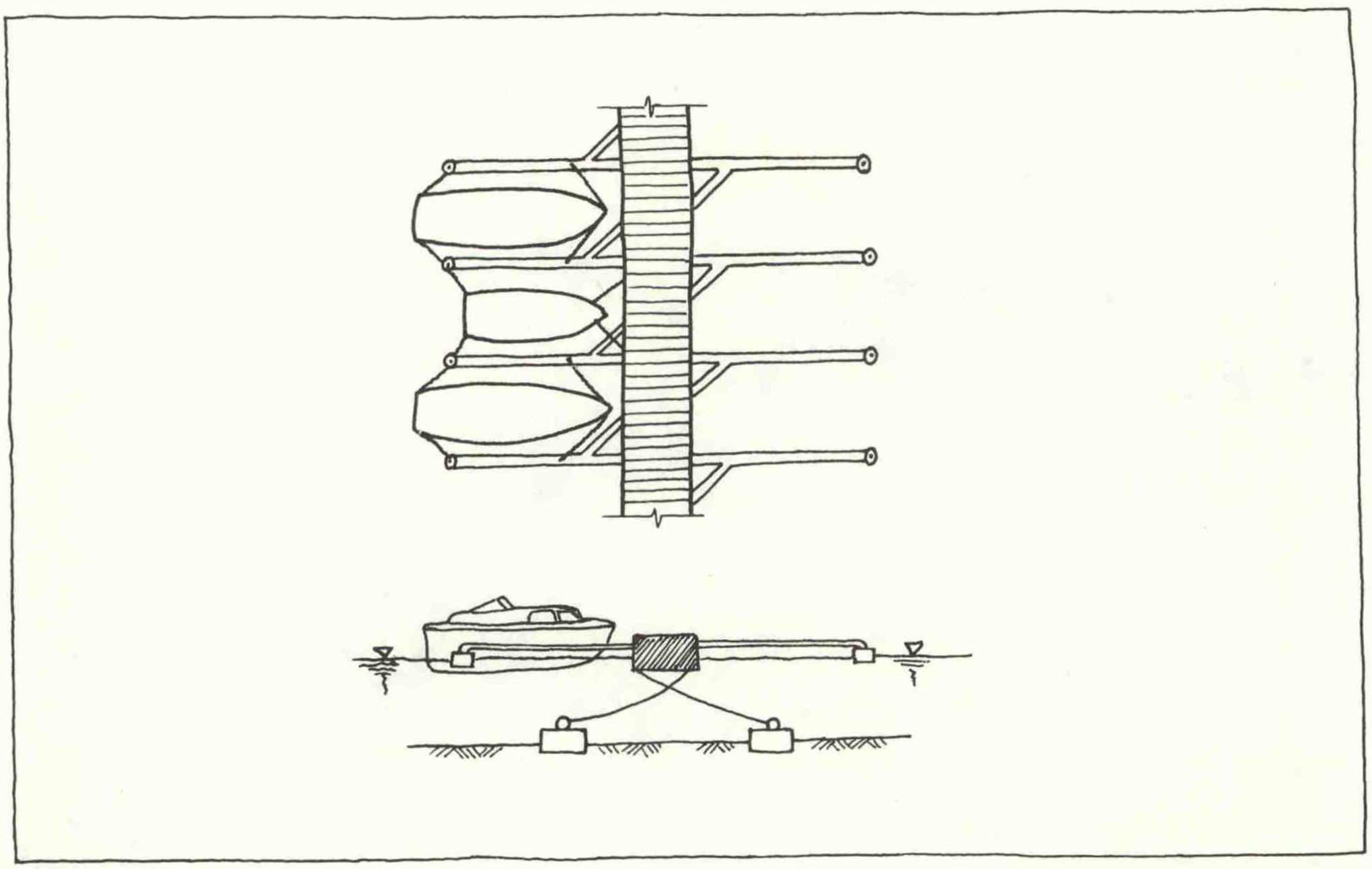


FIGURE 9.10: Floating Dock with Mooring Arms (after the Swedish guide Home Harbors for Leisure Boats by W. Altrock)

strut braces on each side of the arm (like conventional deck fillets at finger-to-head pier connections). Some can be moved laterally along the length of the head pier to provide varying slip widths. Some are braced with a single shock absorber strut (Figure 9.11). Most are supported in the water (and ice) at their far ends by one or two flotation canisters.

Also, the use of mooring arms eliminates the need for mooring or spring piles in double slips. Not only are mooring piles expensive, but in many harbors they are nearly an impossibility because of ice forces, water depths, soil conditions, etc. Mooring arms can also be used with fixed piers.

This and the preceding chapter examined some design methods for suppressing and controlling ice and for floating structures in ice. The next chapters examine some design methods for fixed structures, both shallow and deeply founded. The design is not precise by any means, because not only are there questions about the ice forces, but the soil response is very complex.

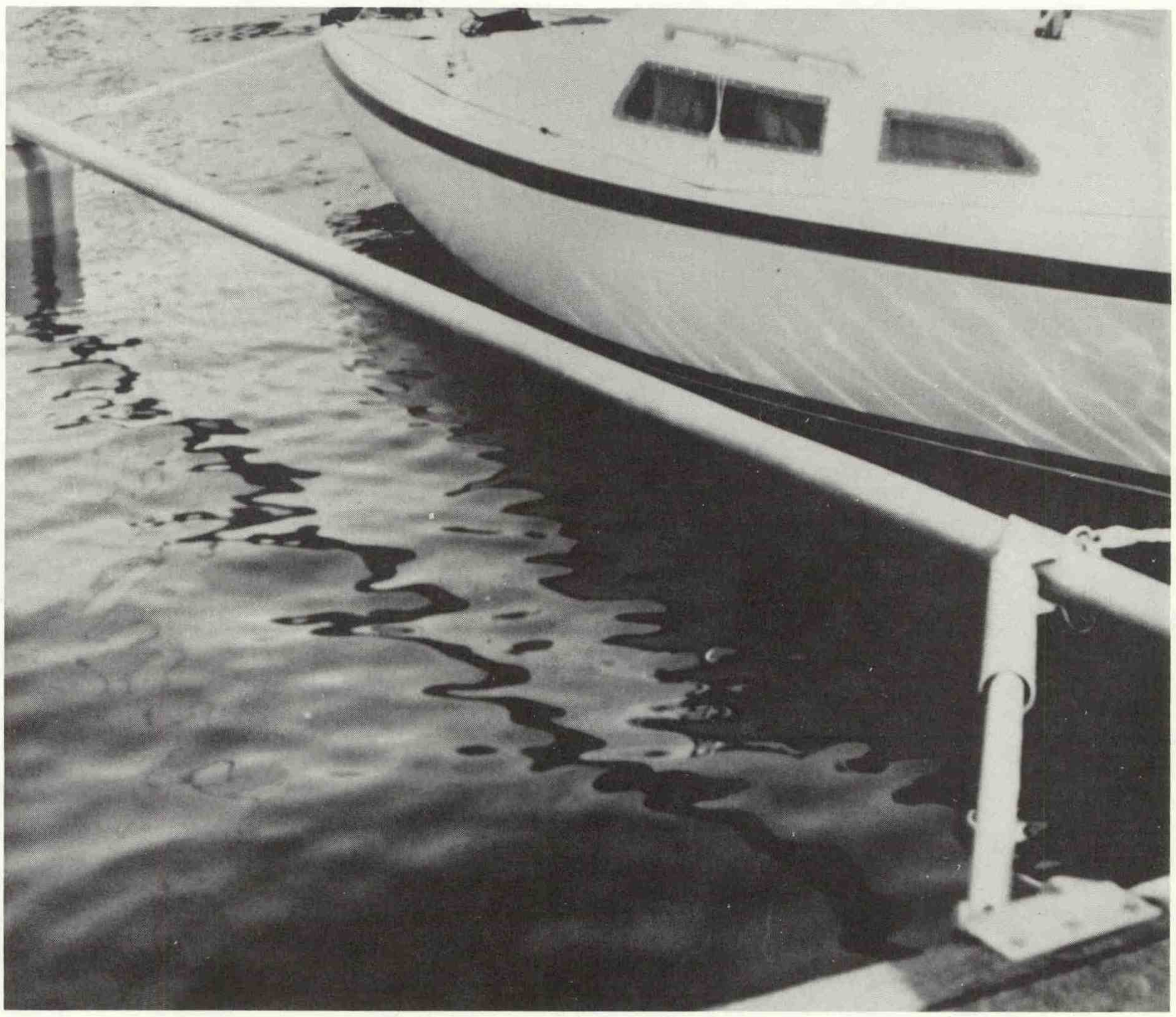


FIGURE 9.11: "Shock Absorber" Mooring Arm

10. Design Methods for Shallow Foundations for Structures

This and the next two chapters review and make recommendations for the design of bottom-fixed structures, both shallow and deeply founded. An extensive literature search turned up no simple design solutions. The design of foundations for small-craft harbors is largely a procedure based on experience and judgment, tempered by qualitative concepts and observations of their behavior. Stationary ice forces are the primary concern here; ice floes and jams against structures will not be analyzed.

ICE FORCES ON WALLS AND CRIBS

The stationary ice forces on walls and cribs are both horizontal and vertical. The horizontal forces are principally the result of thermal stresses in the ice cover, and the vertical forces are the result of the ice cover moving with the water surface of the lake. The vertical forces act both up and down.

Downward Ice Loads

Floating lake ice weighs about 57 pounds per cubic foot (except the spring ice that settles to the bottom--it must weigh more!). The ice frozen to harbor structures are assumed to lose the water's bouyant support as the water recedes until the ice becomes a hanging dead weight. If the ice is on a superstructure above the water, it behaves as a regular live load of 57 pounds per cubic foot. In short, marina piers subject to icings or having an ice cover dropped on them should be designed for this imposed load.

In computing hanging weight on cribs, walls and piled structures, you must estimate how far the ice will span before it breaks or sags to the lower water level. Observations of ice cracking patterns in marinas and around structures suggest a span of at least 30 feet in 20- to 30-inch ice sheets. Water levels may fall several feet during extreme seiches and wind events, suspending the greater part of the ice above water level.

Hanging thick ice underneath and a loss of water support are believed to be responsible for driving the piles supporting a Lake Superior marina pier 3 to 6 inches further into a stiff clay bottom--and after 5 years of satisfactory service. The pilings were nominal 15-ton bearing piles.

What tributary area of ice could cause this? Assume the pile was 15-ton and had an ultimate factor of safety of 6 and that the ice was 3 feet thick; then the tributary area would be:

$$\text{Tributary Area} = \frac{15 \times 2{,}000 \times 6}{3 \times 57} = \frac{180{,}000}{171} = 1{,}050 \text{ square feet}$$

If the area were circular, the radius would be 18 feet. The ice can easily span 18 feet. Had the dock support been a crib or a column on a spread

footing rather than a piling in this case, the load would have been the same--namely, 180 kips onto the shallow foundaton.

Upward Ice Loads

Ice lifting does little damage to walls and long cribs. Normally, the ice cracks parallel to these structures, and this reduces the uplift force. Occasionally, however, the top of a crib will be pulled off because it was inadequately attached to the lower part of the crib. The uplift estimates given in Chapter 3 can be used for analysis.

For example, estimate the possible uplift of a rock-filled timber crib that weighs 750 pounds per square foot and is 8 feet wide and 10 feet long. Its total weight then is 60 kips. The ice has not lifted this crib in the past. An equivalent circular area would be 10 feet in diameter, and assuming a 24-inch ice cover (Table 3.1), the minimum uplift load is 60 kips. This is based on an elastic first-crack analysis.

As indicated earlier, the ultimate uplift load could be 3 to 5 times the minimum--but apparently it isn't in this case, because the crib has not lifted. The crib probably doesn't see an infinite tributary area, and many of the other assumptions used in the theoretical analyses also probably are not representative of the real conditions.

It does seem that the first-crack analysis does give reasonable estimates for uplift--regardless of what actually is the correct analysis. There are very few actual force data, so you must rely on observations of what has worked in the past.

Table 3.2 lists estimated lifting forces per foot of wall. In the above example, the 24-inch ice sheet would lift 800 pounds per foot, or for the 8- by 10-foot area's perimeter, a total of about 30 kips uplift. As stated previously, the design of the crib must allow for this lifting force--whether 30 kips, 60 kips or whatever. The sides must not pull apart, and the bottom must not drop out.

Lateral Ice Loads

Gold (1978) observed that there still are not sufficient field observations on the interaction between ice and structures to establish unequivocally the correct design methods for calculating ice pressures under given conditions.

With regard to static ice pressure, Gold noted that ice covers exert a force on a structure only if they move relative to it. These forces may be associated with an ice cover that is essentially fixed or moving very slowly, such as land-fast ice in coastal regions and the covers on many lakes. Ice pressures are considered static if the inertial term can be neglected; if it cannot be neglected, they are considered dynamic. (Dynamic ice pressures won't be discussed here.)

Forces may be induced in or imposed on a stationary ice cover in a number of ways. The extent to which these forces are exerted on a structure depends on several factors, including the size of the cover, the degree to which it is

restrained at its edges, the size and shape of the structure, and the amount of movement to cause the forces to be reduced to a negligible value. In some situations (e.g., at the head of a long, narrow channel or in similar protected situations), the forces imposed by an ice cover always may be insignificant due to the restraining influence of the shores, or simply because the forces cannot develop to any significant degree. For others, the structure may be exposed to the full force of which the cover is capable (e.g., on an extended shoreline of a large lake).

It is possible, in theory, to establish from appropriate site investigations the extent to which a structure will be subject to ice action. This capability is still not adequately developed, however, because of the lack of knowledge about the forces in ice covers, the factors upon which they depend and the interaction between the cover and its surroundings.

Both wind and moving water exert a shear stress on an ice cover by a transfer of momentum to the surface. In a lake, wind stress can be assumed to be a factor, while a strong current will not. Gold estimates the wind drag on an ice cover and assumes some rather extreme values for both the drag coefficient and the wind speed.

Assuming a drag coefficient of 0.003 and a wind of 35 mph (measured at an elevation of 30 feet and converted to ice level), it would take an ice fetch of 10 miles to develop an ice pressure of 1 kip per foot. The pressure varies as the square of the wind speed and directly with the fetch. With a 70 mph wind, it would take an ice fetch of 2.5 miles to develop an ice pressure of 1 kip per foot. Such wind thrusts will not reach protected harbor structures, and they are less than those of thermal origin.

Chapter 3 discussed ice pressures of thermal origin. Gravity structures will experience lateral shoving and must be designed to withstand the thermal forces. Figure 10.1 shows a rock-filled crib gravity structure.

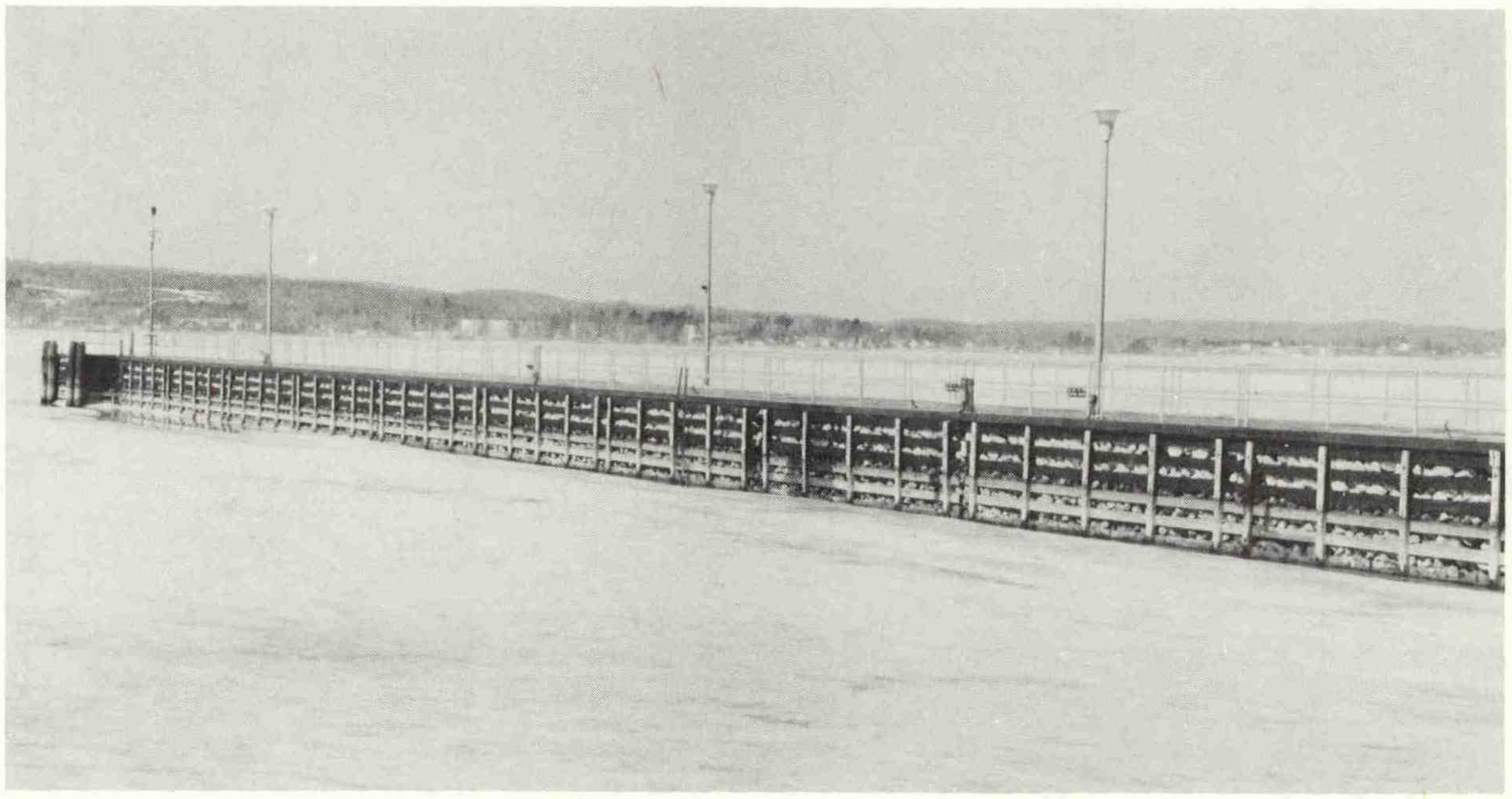


FIGURE 10.1: Rock-Filled Crib Gravity Structure

Table 3.5 presented the results of Drouin and Michel's (1974) laboratory work. Because of cracks, faults and discontinuities, natural ice will have a net sheet strength weaker than laboratory ice. Also, any snow on the ice will reduce the thermal responsiveness of the sheet. Thin ice is not capable of exerting significant thrusts; it buckles first. Thick ice tends to be self-insulating (i.e., the effects of a sustained temperature rise are attenuated with depth in the sheet). The thickness of the ice is not a critical factor in estimating thermal forces. For these reasons and using Table 3.5, some estimates of values to be used for design can be made. The table shows that, for temperature rises between -4°F and 32°F for durations of 5 to 20 hours, the thermal thrust for ice 20- to 30-inches thick ranges from 8 to 20 kips per foot. If the ice were warmed from 14°F instead of -4°F, as might be the case if it had some snow cover, the range would be 5 to 9 kips per foot.

I recommend using a design value of about 10 kips per foot (150 kN/m) for most Great Lakes structures; use values half this amount in areas of large snowfalls or weak, unsound ice; and use values twice this amount for clear ice in confined boat harbors (without sloping banks) and under unusually warm periods following very cold weather (20 kips per foot may be representative of biaxial restraint conditions). Of course, the importance of the structure to the overall project also will be a factor in selecting the ice design value and safety factors. Based on my observations of ice in the Great Lakes and on cribs that are still standing, I believe that 5 to 10 kips per foot (75 to 150 kN/m) are reasonable ice thermal thrust values for this region.

Figure 10.2 applies these estimates to a crib like that shown in Figure 10.1. It assumes water on one side and warming ice on the other. This is a severe

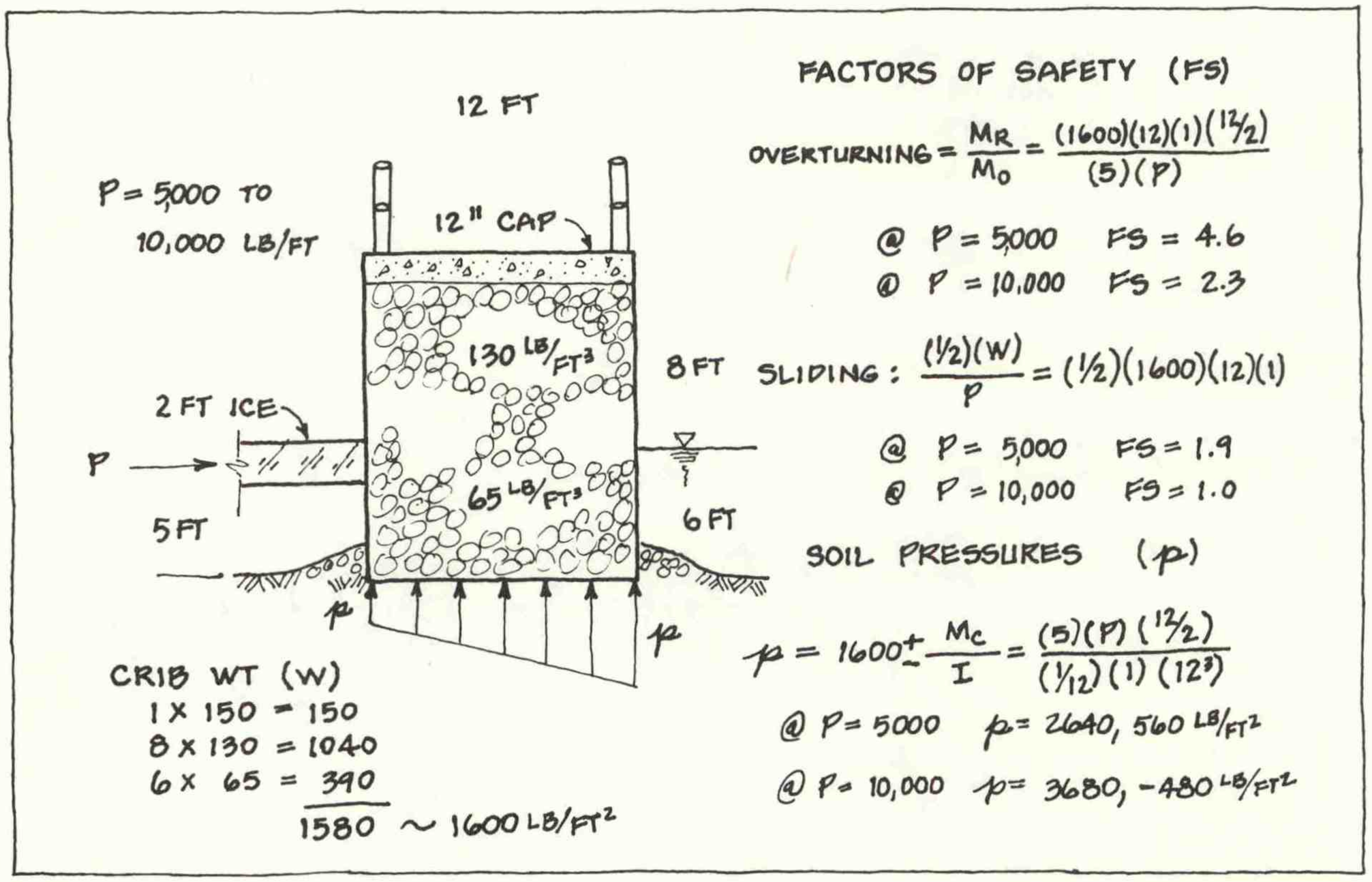


FIGURE 10.2 Analysis of Crib with Ice Thrust

condition, but conceivable if extensive deicing were being used in a nearby marina dockage or, if to relieve pressure elsewhere, someone had cut a trench in the ice. A coefficient of sliding friction of one-half is assumed.

The crib is stable with regard to overturning and marginal for sliding, though no passive resistance through embedment was assumed. The maximum soil pressures are reasonable and would be allowable on a medium sand or stiff clay (allowable pressures are discussed in the next section). A negative heel pressure (at the back of the wall) of 480 pounds per square foot results. Assume a heel pressure of zero and a toe pressure (at the front of the wall) of about 4,000 pounds per square foot.

With the ice forces estimated, the allowable bearing pressures for sands and clays, and the resulting settlement need to be determined next.

SOIL BEARING CAPACITY

In general, the bearing capacity of shallow foundations in small-craft harbor construction presents no unusual problems. A footing would not be put on a soft clay or compressible material; rather, a deep foundation would be used, the in-situ soil condition would be improved, or the unsatisfactory soils would be excavated and replaced with suitable materials. In a marina, settlements of an inch or two are tolerable, provided the settlement is uniform and not isolated in places. Immediate and long-term settlements would be in this range under typical loadings.

The allowable soil bearing pressure can be estimated from the bearing capacity equation (Equation 5.1) reduced with a factor of safety. Equation 5.1 can be further simplified if the cohesion term (N_c) is dropped and the depth term (N_q) in the case of a footing placed on a sandy harbor bottom. With the inclusion of a safety factor of 3, Equation 5.1 can be conservatively written:

$$q_a = \frac{1}{6} \gamma' B N_\gamma \qquad (10.1)$$

where: q_a = allowable bearing pressure on sand on harbor bottom (with factor of safety equal to 3)

γ' = submerged unit weight of sand

B = width of footing

N_γ = bearing capacity factor

Equation 10.1 shows that the allowable bearing pressure is primarily a function of the width of the footing (B). The submerged unit weight of sand (γ') varies some but not a lot; typical values are listed in Table 5.3 Values for N_γ are shown in Table 5.1 The approximate allowable bearing capacities in Table 10.1 were constructed with Equation 10.1 and Tables 5.1 and 5.3.

Now refer back to the previous example and Figure 10.2. An actual toe pressure of 4,000 pounds per square foot was calculated. According to Table 10.1, this figure is alright for a 12-foot-wide footing on medium sand. The kind of sand present--loose, medium or dense--should have been characterized by the

TABLE 10.1: Approximate Allowable Bearing Capacity (q_a) for Footing on Sand Harbor Bottom (in pounds per square foot)

Relative Density, D_r	Width of Footing (feet)				
	6	8	10	12	14
Loose	1,100	1,500	1,800	2,200	2,600
Medium	2,200	2,900	3,600	4,300	5,100
Dense	4,600	6,200	7,800	8,000[a]	8,000[a]

[a]Maximum recommended.

soil engineer in a geotechnical engineering report, based on field and laboratory procedures, and quite a bit of professional judgment.

As likely as not, the soil engineer will have used an empirical correlation of the relative density of the sand, with the standard penetration test (SPT) N-values or blow counts recorded in the first few feet of the test borings. Averaged values for N within a depth from zero to about half the width of footing will have been used, with care taken to notice any very soft and unsatisfactory zones.

Standard Penetration Test (SPT) and N-Values

Remember from Chapter 4 that the SPT is a very important procedure in sampling and penetration testing. The test consists of driving a split spoon sampler with a 140-pound weight freely falling 30 inches. The N-value is the number of blows required to drive the sampler 12 inches, which is referred to as the standard penetration resistance or, simply, the "blows."

The SPT is often criticized for its crudeness and imprecision. It is known that variations in test procedures and insufficient attention to regulating the fall of the hammer can produce significantly different results in the same soil.

N-values are used to correlate with many soil parameters and soil properties. In fact, most would agree that N-values are used beyond their inherent capabilities for prediction, but they are used nevertheless.

Vesic (1977) indicated it is increasingly accepted today that the primary purpose of the SPT test should be to correlate experiences of a single organization in familiar geotechnical profiles. I agree--that is why I said that the soil engineer would characterize the sand. However, since the SPT and N-values are so prevalent, it is worthwhile to review a little of what has transpired since the test's origin.

The SPT was originally developed for cohesionless soils so that samples would not have to be taken. (Since clay holds together, it is easier to sample.)

TABLE 10.2: Relative Density of Sand (D_r), and Consistency and Unconfined Compressive Strength (q_u) of Clay (after Terzaghi and Peck 1948)

Sand		Clay		
N-Value	Relative Density, D_r	N-Value	Unconfined Compressive Strength, q_u (kips/ft.^{2})	Consistency
0-4	Very loose	0-2	0-0.5	Very soft
4-10	Loose	2-4	0.5-1.0	Soft
10-30	Medium	4-8	1.0-2.0	Medium
30-50	Dense	8-15	2.0-4.0	Stiff
Over 50	Very dense	15-30	4.0-8.0	Very stiff
		Over 30	Over 8.0	Hard

The test has evolved to the current practice of routinely determining N for all soils (sands, silts and clays), supplemented with determinations of the unconfined compressive strengths for cohesive strata.

Terzaghi and Peck (1948) published two tables relating the relative density of sand and the consistency of clay (unconfined compressive strength) with the number of blows (N). I have combined their tables here as Table 10.2.

In their book, Terzaghi and Peck caution about silty soils, water table locations, scattering of data, the need for compression tests on clays and other matters. Since 1948, many developments regarding N-values have occurred. I have listed a few here so that you may have a little deeper understanding of soil borings and testing.

Research Concerning SPT and N-Values

Meyerhof (1956) made comparisons between the SPT and the CPT penetration resistance tests on a number of sites. CPT is short for cone penetration test, in which a 60° cone is pushed into clay or sand and the point resistance and the friction on a friction sleeve attached above the cone are measured. This test is not widely used in the United States. Meyerhof obtained a simple approximate correlation between both types of penetration tests, and relative density (D_r) and soil angle of friction (ϕ). Meyerhof used Terzaghi and Peck's N vs. D_r tables to correlate the static cone resistance to D_r and ϕ.

Gibbs and Holtz (1957) reported laboratory research on the determination of the density of sands by spoon penetration testing (the SPT). They were concerned that the empirical rules then available for interpreting SPT data did not provide an evaluation for the length and weight of drill rod and the

effects of moisture and overburden pressure. They concluded that penetration resistance increases with an increase either in relative density, or in overburden pressure. Since the principal object of the SPT in cohesionless sands is to evaluate density, the effect of overburden pressure at the depth of the test must be taken into account (rod length, by comparison, is insignificant).

Gibbs and Holtz had reservations about the results of saturation tests in their relatively small laboratory tank. Their laboratory results showed lower penetration resistances than field conditions and so may not be representative of the water table.

Gibbs and Holtz's results followed closely Terzaghi and Peck's N vs. D_r correlations for higher overburden pressures of 40 psi (5,760 pounds per square foot). So, it could be said that Terzaghi and Peck's correlations are on the conservative side for the shallow footing work for which they were intended (5,760 pounds per square foot is about 50 feet into the earth.)

Bowles (1982) reported that Bazaraa (1967) made an analysis of a large number of borings by others and proposed correlations to the actual blow count (N) based on overburden pressure. Bazaraa found that the widely used Gibbs and Holtz values were too conservative.

DeMello (1971) prepared a state-of-the-art report on the SPT. DeMello found that, in the case of sands, penetration tests--and in particular the SPT--had acquired a place of exaggerated importance. This was because of the difficulty, expense and frequent unreliability of the special procedures required for extracting and testing undisturbed samples, particularly from greater depths and from below the groundwater table. Thus, the SPT was intuitively implied as a test for granular materials, in which, moreover, it has been assumed that dynamic resistances should not differ much from static ones.

DeMello stated that it has been satisfactorily confirmed that submergence and saturation have no noticeable effect on the SPT values of fine to coarse sands and gravels. The SPT seems to be most profitably interpreted as related to in-situ undrained strength parameters of soil. Apparently, the SPT is least appropriate for shallow depths, perhaps particularly so for sands at shallow depths.

It appears to be convincingly established that in saturated insensitive clays, SPT values are a direct measure of the undrained shear strength and may in first order approximations be connected through conservation of energy principles analogous to those of judicious dynamic pile formulas. The sensitivity of clays should have a significant influence on the correlation of SPT with the unconfined compressive strength.

Marcuson and Bieganousky (1977) studied the SPT and relative density in coarse sands. The spread of data derived from testing four sands under optimum laboratory conditions suggests that a simplified family of curves correlating N values, relative density and overburden pressure for all cohesionless soils under all conditons is invalid. Correlation factors (Peck et al. 1974) are not solely a function of effective overburden pressure but are also functions of effective overburden pressure, relative density, soil type and other parameters that were not defined or evaluated in their study--such as soil structure, cementation, etc. Marcuson and Bieganousky's tests don't correlate well with Gibbs and Holtz.

Now, if you've read well these past few pages, I hope you will agree with Vesic's (1977) observation that the primary purpose of the SPT should be to correlate experiences of a single organization in familiar geotechnical profiles--and agree with me that your soil engineer should characterize the soil deposits.

Allowable Bearing Capacity of Clay Soils

Chapter 5 indicated that unconfined compressive strength of clay bottoms was approximately equal to the bearing capacity of a cohesive soil. The unconfined strength can be determined with a laboratory test on a recovered sample, with a pocket penetrometer reading on a sample, or by blow counts:

$$q_a = q_u \qquad (10.2)$$

where: q_a = allowable bearing pressure on clay (with factor of safety equal to approximately 3)

q_u = unconfined compressive strength

SPREAD FOOTINGS AND ISOLATED SHALLOW FOUNDATIONS

This chapter began with an example of 180 kips of hanging dead-weight ice on a marina piling and indicated that it could have been a load to a shallow foundation.

A marina in the Great Lakes was excavated in a very stiff clay deposit. The excavation area was dammed off from the lake so the work was done in the dry. Docks were suppported on 12-inch concrete columns on 5' x 5' x 1.5' concrete pads placed on the clay bottom. A very stiff clay has an unconfined compressive strength, or allowable bearing pressure, of 4 to 8 kips per square foot. A load of 180 kips on this footing pad would result in a contact pressure of 7.2 kips per square foot. This is alright--even if it exceeded 8, nothing would happen, as there is a factor of safety of about 3.

Theoretically, however, the ice could lift this 12-inch concrete column and footing. At a 12-inch radius of load distribution and 24-inch-thick ice, the minimum ice uplift force is 33 kips (from Table 3.1). The weight (submerged) of the footing and column is only 4 kips. Even with some deck load, that is way short of 33 kips. What is actually happening here is not known--the ice may be slipping on the concrete; the harbor ice may be cracking on nearby structures into small size pieces, precluding large uplift forces; or perhaps the column and footing are riding up and down with the ice.

One solution to uplift on small piers is to let the shallow foundation and pier post(s) rise and fall with the ice. One site where this has been done uses a detail with two 3-inch pipe columns for supports at the ends of finger piers. The pipe columns rest on mud sills or some type of bottom plates. Provision is made for dock height adjustment near the tops of the pipes, because the up and down movements may lead to uneveness.

A similar arrangement uses a truncated pyramidal-shaped metal box filled with rocks to support the finger end (yes, the larger end of the truncation rests

on the bottom). The originator of this scheme thought the ice would slip past the pyramid because of its tapered shape. It doesn't--it lifts the box of rocks up and down. This method of support worked well until other piers were to be constructed. The new piers were longer and needed an intermediate support. A piling was used for this support. Ice jacked this piling and broke the pier stringers, which wanted only to oscillate with the attached box of rocks.

11. Design for Deep Foundations for Structures —Part I

As in the last chapter, this and the following chapter review and make recommendations for bottom-fixed structures, both shallow and deeply founded. After a review of the information presented up to now, this chapter discusses ways to mitigate ice actions and forces on pilings and structures, the mechanisms of uplift and related topics. Chapter 12 addresses designing pilings for vertical and horizontal forces.

REVIEW AND SUMMARY OF CHAPTERS 1-10

Chapter 1: Introduction

A small-craft harbor in northern climates presents a complex situation of ice-water-soil-structure interaction that needs to be understood and provided for in the design of marinas for ice conditions.

Chapter 2: Ice and Ice Covers--An Introduction

Natural ice usually exists at a temperature near its melting point. Lake ice has a bottom surface temperature equal to its melting point and an upper surface temperature that varies with ambient conditions. Impurities in ice accumulate and form pores and cavities. Two principal forms of ice are columnar ice, which grows down into the water, and snow ice, which forms on top of the ice from snow and other sources of water. The most common impurity in ice is air, which gives ice its whitish color. Transparent "black" ice forms slowly, rejecting air. The principal impurity in sea ice is salt.

Fresh water reaches maximum density at 39°F (4°C); warmer or colder water will overlie this dense water. Water freezes at 32°F, and "warm" (unfrozen) water is found under an ice cover. An ice cover can thicken at rates exceeding an inch a day. Ice along the shoreline tends to melt first, leaving the main body of an ice sheet floating free. Lake ice is not equally dense nor uniform in thickness. Ice shoves from the stronger toward weaker ice or open water.

Chapter 3: Introduction to Ice Engineering

When ice is strained slowly, it behaves in a ductile manner; when strained rapidly, it is brittle. Under load, ice will creep over time. Static friction coefficient values range from 0.01 to 1.0. Ice adheres tenaciously to most construction materials. The adhesive strength for ice on construction materials is comparable to the shear strength of the ice itself, usually ranging from 60 to 150 psi (400 to 1,000 kPa). Low-adhesion coatings and other methods can be used to reduce ice's grip.

Changes in water levels produce vertical ice forces on pilings that frequently range from 25 to 75 kips (100 to 300 kN) and possibly higher. Vertical forces on walls may be on the order of one, possibly two, kips per foot (15-30 kN/m). Bearing capacity-safe loads on ice vary but are also in the range of 25 to 75 kips (100 to 300 kN).

The horizontal forces of ice are dynamic or static. Under dynamic loading, structures are impacted and also subject to vibration. Static thermal thrusts range from 5 and 20 kips per foot (75 and 300 kN/m) and are highest for small rates of temperature increase at the surface of the ice. The coefficient of thermal expansion of ice is 0.000030 at 32°F. Cracks in ice absorb a significant amount of movement. Field data on thermal thrust on individual pilings is not available.

Chapter 4: Review of Geotechnical Engineering: Soil Mechanics and Foundations

Two important soil relationships are the void ratio (the volume of the voids to the volume of the solids) and the water or moisture content (the weight of the water to the weight of the solids). The soil in harbor bottoms is saturated, and its weight is its bouyant unit weight (saturated weight minus 62.4 pounds per cubic foot).

Coarse-grained soils are gravels and sands, which are referred to as cohesionless soils; fine-grained soils are silts and clays, referred to as cohesive soils (though silts can also be cohesionless). The range of moisture over which soil is plastic is defined by the liquid limit minus the plastic limit (the Atterberg limits), referred to as the plasticity index of the soil.

The term "clay" refers to specific minerals, such as montmorillonite. Clay soil contains some clay minerals, which even in small percentages can sometimes markedly affect the properties of the soil. The structure of naturally occurring clay soil deposits is highly complex. Granular soils may be characterized by their relative density (a relation between void ratios in situ, and maximum and minimum void ratios).

Water in soils very strongly affects their engineering behavior. The total stress on soil is shared between the intergranular stress among soil grains (effective stress) and stress in the pore water (neutral pressure). The ratio of horizontal stress to vertical stress in a soil mass is called the earth pressure coefficient, and if the mass is stationary, the coefficient is the at-rest pressure coefficient.

Soils consolidate and settle as water is squeezed out of pore spaces. Soils have memory—they "remember" past loads and stresses and are said to be normally consolidated or overconsolidated from the effects of past loads. The shear strength of soil is related to its cohesion (c), its angle of internal friction (ϕ) and the overburden pressure. Soil strength parameters c and ϕ may be determined from laboratory tests, where drainage and consolidation conditions may be controlled to replicate field design conditions. Soil strength also may be estimated by field tests like the standard penetration test (SPT) using blow counts (N).

Geotechnical engineering problems may be analyzed from two approaches—"total stress" and "effective stress."

Chapter 5: Introduction to Foundation Design

Observations of the behavior of foundations subjected to loads show that bearing capacity failure usually occurs as a shear failure of the soil supporting the footing. The shear failure can be of a local, general or

punching type. Which type occurs in a particular case depends on factors only partially explored so far.

An estimate of ultimate bearing capacity (and of allowable bearing capacity by applying a safety factor) can be made with a general equation relating the soil strength parameters, unit weight of the soil, and the size and depth of the footing. The general equation uses bearing capacity factors which are functions of the angle ϕ. The bearing capacity of shallow footings on clay is approximately equal to the soil's unconfined compressive strength.

The coefficient of friction of concrete to sandy gravel in submerged conditions is approximately one-half. Settlements of footings on sands in a harbor, designed in accordance with allowable bearing capacities, should be less than an inch or two and not troublesome.

In addition to at-rest earth pressure on walls, there are active earth pressures when walls move under earth load, and passive earth presures when walls are pressed into an earth mass. These earth pressures are characterized with active and passive pressure coefficients, which are tabulated.

Earth tiebacks appear too costly to resist ice uplift loads. Rock anchors may be appropriate under some circumstances.

Cribs and gravity structures may be built of timber, steel or precast concrete and filled with rocks and soil. If bottomless, such structures must be designed for internal shear and general base failure modes.

For vertical failure of a pile, the shear stress at the pile shaft-soil interface attains a limiting value (possibly varying with depth and soil type). For horizontal failure resulting from lateral load or movement, the normal stress at the interface attains a limiting value (again, possibly varying with depth).

The installation of piles affects the soil mass by disturbing its structure and inducing stresses in both the soil and the water. Generally, the capacity of piles increases with time. A closed-end pipe or a full-section rectangular or square pile causes a much larger increase in lateral stress than an open-ended pile or a steel H-pile (a nondisplacement pile). The effects of driving piles in sands and clays are many, are not well quantified and are difficult to deal with specifically in a design-sense.

The ultimate uplift resistance of a pile is equal to the length of the pile times the soil contact surface area times the unit skin resistance (f_s), which is assumed to consist of two parts: adhesion (c_a) and friction.

Friction is assumed to be a function of the soil lateral stress on the pile shaft. For a granular deposit, c_a is normally small, but for a cohesive deposit, f_s will be related to the soil cohesion (c), which empirically is related to c_a.

Skin friction somehow varies with cohesion, adhesion, effective vertical stress, coefficients of friction for soil and other materials, undrained and drained parameters, lateral earth pressure coefficients, and pile characteristics such as method of placement, shape and length. The analytical methods are complicated.

Dynamic methods, using driving formulas, may be used to estimate the ultimate load capacity of piles. A driving formula is used to establish a safe working load, or to determine the driving requirements for a required working load.

The design of piles for lateral loads considers an adequate factor of safety against ultimate failure and an acceptable definition of working loads. In many cases, the ultimate load will be reached at very large deflections.

Chapter 6: Small-Craft Harbor Site Characterization

Probably the most variable soil deposits are those associated with glaciation, and the basins of the Great Lakes are the result of glacial erosion. Subsurface exploration must be adequate to disclose the essential character of the soil and, particularly, variations from point to point. Shelby tubes are used to sample cohesive deposits; SPT N values are appropriate for granular deposits.

No matter how complete the program of soil exploration and testing may be, there always remains a large margin of uncertainty concerning the exact nature of the subsurface conditions at a given site.

The water levels of the Great Lakes vary throughout the year and irregularly from year to year, so harbor designers must consider the effects of both high and low ice conditions during the winter. The Great Lakes are affected by wind setups, which can cause sudden changes in water level ranging from a few inches to many feet.

The deformation of lake water surfaces from winds and storms gives rise to water oscillations, a sloshing called "seiches." A seiche may last only a few minutes in a bay and up to 10 hours in a Great Lake. Uplift forces from seiches can occur in any Great Lakes harbor. A tributary can also experience large water level rises due to the damming effect of a seiche at its mouth, particularly when ice is present.

Harbor water temperatures are isothermal and frequently very near the ice melting point. Typical ice thickneses in the Great Lakes range from 2 to 3 feet. Ice attachments to pilings may be less than full sheet thicknesses. Sums of Freezing Degree Days are available for many locations and are a measure of winter's coldness. They can be used to estimate ice thickness.

General site features also include site accessibility, power supply and reliability, snowfall, spray icing potential, basin shape and orientation, and slope of basin sides.

Chapter 7: Preliminary Design Considerations

Structures planned for a marina may be fixed or adjustable, floating, movable or removable, or planned for deicing. The design must consider the environmental site characterizations and such other factors as safety, economy, use, appearance, service life and local preferences and needs.

Economy is a very important factor. Life cycle costing and value engineering-value analysis procedures can aid in economic analysis. "Cost of repairs" is a factor that should be planned (i.e., realistically addressed) in the project economic analysis.

Floating docks can be left in quiet, nonshifting ice conditions, or they can be removed if necessary. Fixed docks can be deiced or designed to withstand or at least tolerate ice forces and actions.

Chapter 8: Design Methods to Suppress and Control Ice

Compressed air bubbler deicing is an effective ice suppression and control measure. Air bubbles released at the harbor bottom entrain "warm" water, and the rising plume impinges the ice cover and flows laterally, melting the cover. Three to 6 cfm of air per 100 feet (0.0045 to 0.0090 m^3/min/100m) of bubbler hose is usually sufficient. Compressors should produce high volumes of air at low pressures--in the range of 10 to 15 psi (70 to 100 kPa). Rotary vane compressors and straight-lobe positive displacement blowers frequently are used on larger deicing layouts.

Condensate in compressed air will freeze if uninsulated distribution manifolds are laid above a lake ice cover. Air bubbler diffuser lines are laid uphill from deeper water to shallower water. A compressed air deicing system needs careful installation, maintenance and monitoring during the operating season.

Compressed air systems work in freshwater, saltwater and brackish water. They do not work well in rivers.

Flow developers and water agitators move surface waters and prevent freezing especially well with warmer waters, and they can also be used in saltwater and brackish water conditions.

Ice control measures, which control rather than suppress or melt ice, are effective. Two of these measures are high-flow air screens and ice booms.

Chapter 9: Design Methods for Floating Structures

Floating docks are an effective design choice when they are properly constructed and used in suitable harbor locations. Field experiments and observations indicate that flotation encasement material and its color, the size and shape of the pontoon, and the immersion depth are unimportant. Minimum encasement thicknesses are necessary for different materials. Floating docks not previously used in Great Lakes ice should be used with caution until satisfactory performance is demonstrated. Concrete pontoons should be tried, however.

Floating docks remaining in the ice for the winter must be free of restraints and placed in locations with nonmoving, quiet ice; otherwise, they are likely to be damaged. Docks must be flexible but not overjointed; otherwise, modules will become kinked. One- to 2-foot contortions along a dock should be anticipated. Floating tire breakwaters in calm ice apparently do all right.

Substituting mooring arms for finger piers can reduce some ice problems for floating docks. Mooring arms with floating concrete head piers have been used successfully in Scandinavian harbors.

Chapter 10: Design Methods for Shallow Foundations for Structures

Ice forces on walls and cribs can be vertical (up or down) due to water level changes, or lateral due to thermal effects. Downward loadings result from

hanging ice left suspended when lake levels fall sharply. Hanging ice may span 30 feet or more. Upward loads result from water level rises, and their magnitudes are described above. Thermal processes cause lateral shoves, ranging from 5 to 20 kips per foot (75 and 300 kN/m). Static wind-drag force is minor in protected harbors. Gravity crib structures can be analyzed for overturning, sliding and soil bearing capacity under the action of thermal ice pressures.

The allowable bearing pressures on medium relative density sand bottoms vary from 2,000 to 5,000 pounds per square foot (100 to 250 kPa) for structures ranging from 6 to 14 feet in width.

Empirical correlation of N-values with bearing capacities for sands and clays should be made by experienced soil engineers.

Marina piers supported on shallow footings and mud sill plates may ride up and down with ice movements.

MECHANISM OF PILE UPLIFT

Fluctuating water levels generate vertical ice forces. When the ice rises, either the pile is pulled from the bottom, or the ice slips on the pile or fails near the pile. If the piling is lifted, the soil at the tip sloughs into the void created. When the lake level recedes, the piling cannot return to its former depth. The ice eventually breaks away from the piling, drops and refreezes lower on the "jacked" pile; thus piles may be jacked completely out of the bottom (e.g., see Fig. 7.2.)

Tryde (1983) reports on vertical ice-lifting in Danish marinas during the winter of 1978-79. Denmark has a tidal variation of about a half-foot and wind-shear water level changes of 3 feet. In a survey of 93 marinas, Tryde found that 4,000 pilings (out of a total of about 31,000 pilings) were lifted partly or totally out of the ground and about 230 were broken. Many of the marinas surveyed had soils that were either sand, or clay (the predominant type by a factor of two to one). About 25 percent of the piles in sand (average bottom penetration 12 feet) were lifted, while only 9 percent of the piles in clay (average bottom penetration 9 feet) were lifted. The average diameter for the wooden piles was 10 inches, and the average ice thickness was 14 inches.

Tryde described the lifting mechanism as follows: The deformation of the ice sheet relative to the water surface seldom exceeds about 4 inches, which in most cases will produce a lifting force large enough to pull the pile out of the ground during rising water levels. In nearly all cases of piles driven in sand, the ice will lift the piles without even cracking the ice. When cracks are formed, it becomes much more complicated. If the water continues to rise slowly and if the air temperature is below freezing, the cracks tend to freeze up as they fill with water. The lifting force will gradually increase until the pile is pulled out of the ground a distance corresponding to the water level rise. This may be repeated until the pile is finally pulled completely out of the ground. If the air temperature is just above freezing, the water may penetrate through the cracks to the surface of the ice, and the lifting force will gradually decrease.

Muschell and Lawrence (1980) describe a "pinching" effect on lifting piles. Under natural ice sheet flexing from water level rise, the ice grip has a pinching effect on the pile because of compressive flexural stresses in the top of the sheet. Presumably, this effect aids in lifting.

I have observed a "prying" effect of ice on the end piles of piers. The ice pries or lifts the pile at the end of the exposed finger pier with respect to the piles under the head pier. I earlier mentioned Kerr's (1978) solution for a row of piles wherein the outer or more exposed piles receive more uplift load than those more protected nearer the shore. Figure 11.1 illustrates this point.

Very limited data exist on uplift forces from ice acting on marina pilings. Hodek and Doud (1975) did a field study in a Lake Superior marina for the Cold Regions Research and Engineering Laboratory (CRREL). They measured vertical loads of 18 kips in compression and 11 kips in tension on a 15-inch instrumented sleeve around a steel pile. The thickness of the ice was about 16 inches. From pilings that have lifted, it is known that uplift forces can be much larger than these recorded values. Using other instrumented pilings, CRREL is continuing its research on actual lifting forces with associated water level changes.



FIGURE 11.1: Steel Harbor Piles Uplifted by Ice

MECHANISM OF ICE RUBBLE GENERATION

Ice rubble is generated as a side effect of pile uplifting and resistance to lifting. This rubble is made up of pieces of ice "formed" around pilings. These pieces can accumulate under piers and transmit forces from rising ice sheets to horizontal structural members and utilities suspended under piers.

Rubble is an indication of fluctuating water levels. However, rubble won't form if the ice is slipping on a piling; with slipping, ice shavings and thin pieces of frozen water film will surround the piling.

If the ice is lifting or jacking a piling, there will be little rubble and pieces of ice, or ice rings, attached along the lifted length of piling. Rubble forms around pilings that are resisting uplift forces and that aren't being jacked from the bottom. The rubble pieces come from ice collars broken off when water levels rise (e.g., see Fig. 6.7).

Rubble is also generated when the ice sheet falls. When this happens, a piece of ice splits out of the top of the sheet. This occurs from diagonal tensile stresses produced between the top portion of the sheet frozen to the piling and the balance of the sheet going down.

When the water refreezes and the sheet again rises, these pieces get pushed up around the piling. Occasionally, very thin pieces of ice, called "blisters," will be formed when a sheet falls and these pieces split out.

Rubble formation is the result of the actions just described and combinations of them that occur throughout the course of the winter. Figure 9.5, for example, shows rubble around an unyielding wood piling; by comparison, Figure 7.2 shows lifted pilings and no rubble (there are also no ice rings along the lifted pile lengths—they perhaps melted off). Figure 11.1 shows small rubble on lifted piles (the rubble perhaps formed prior to lifting).

However formed, rubble can be a problem around docks. Pieces 3 or 4 feet in size have been observed. Details like X-bracing between pairs of piles and "sleeved" piles (discussed in the next section) in particular generate a lot of rubble with falling water levels.

The formation of rubble can be reduced if the ice can be made to slip on the piling.

Another type of ice formation around pilings is the "collar." Figure 11.2 shows a large collar on a piling. This type of collar is seen in harbors where the ice plate is frequently broken and perhaps eventually blown out into the open lake. What remains is skim ice—except around pilings, where the up and down water motion coats the structure.

Such collars or accumulations from such "candle-dipping" action can cause considerable dead load if horizontal dock members are within the water flux. Some hypothesize that, should a firm ice sheet form below a collar, the uplift forces could be very large. The sheet would pry against the enlarged collar area rather than a foot or two of piling.

Theoretically this could happen, but I'm not aware of any actual occurrences. The ice collar probably snaps off under uplift force.



FIGURE 11.2: Large Ice Collar on Steel Harbor Piling

RESISTING UPLIFT WITH SLEEVED PILINGS

Fifteen years ago (besides designing floating docks that sank), my firm designed a sleeved pile system (U.S. Patent No. 3,543,523) that worked and is still working (Wortley 1972). Figure 11.3 shows a sleeved mooring pile, a sleeved dock pile and a sleeved drop spud.

Sleeved piles are effective against ice uplifting. A piece of pipe pile with an internal bearing plate at mid-length is sleeved over a smaller driven pipe pile whose top is about 2 feet below the bottom of the ice sheet. The sleeve moves up and down with ice sheet without pulling on the pile (analogous to a stationary piston in a moving cylinder).

However, provision must be made for irregular vertical displacements throughout the length of a dock framed on top of sleeved piles. An assumption of a variance of one foot between any two piles has proven to be adequate.

Pairs of sleeved piles have been used under head piers with the assumption that they would lift and fall together. However, X-braces connected them, and the X-braces were a source of rubble. They should have been used at a point

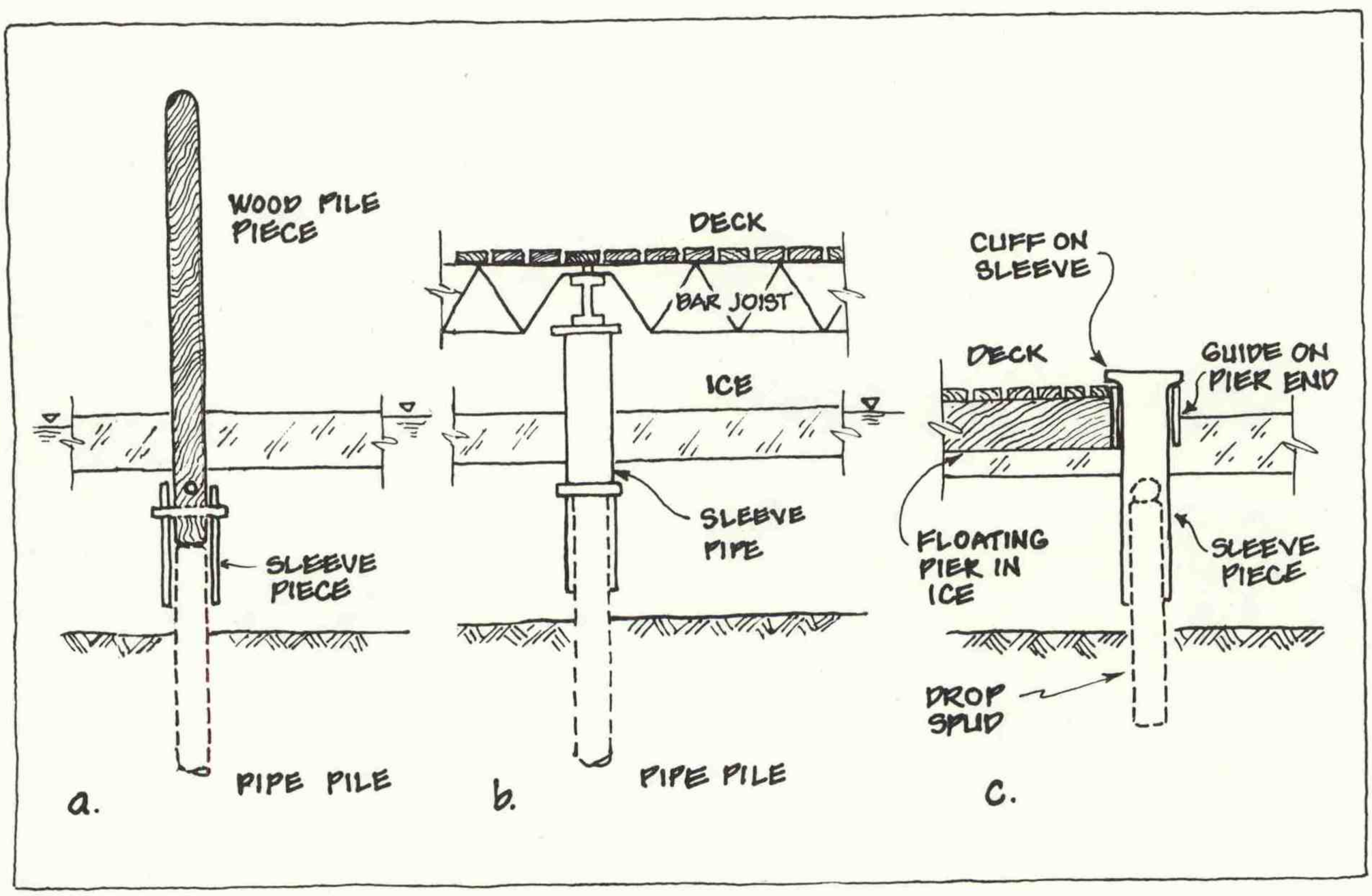


FIGURE 11.3: (a) Sleeved Mooring Pile (b) Sleeved Dock Pile (c) Sleeved Drop Spud

higher than the top of the ice. I have seen pairs of small sleeve piles used at ends of finger piers. They did not move together, and the finger pier was racked and damaged as a result.

The sleeved pile dock system is more expensive to frame structurally and to equip with utilities. Also, it does not address lateral shoving. Lateral shoving has caused sleeves to bind on the pipe piles, and several sleeved piles at finger ends have lifted. In addition, transverse beams at ends of finger piers, which suppport pairs of steel bar joists, have been severely twisted. This results from the pilings, under lateral loads, being pushed towards the stronger and stiffer head pier, with the movement effectively resisted by compression in the bar joists. Had these beams not yielded, the bar joists would have buckled horizontally.

The use of sleeves and wood "stick-ups" for mooring piles is simple to frame and is economical. Should a sleeve lift off, it can be reset. Sleeved drop spuds are now being used to restrain floating piers.

I have made time-lapse films of the dock and mooring piles of sleeved design. The films clearly show that the sleeves ride up and down with the ice. Occasionally, a sleeve is seen to bind up, and the ice then fails or slips on the sleeve.

RESISTING UPLIFT WITH COMPLAINT MATERIALS, WRAPPINGS, COATINGS, ETC.

Piles have been surrounded with metal drums and other retainers, forming annular spaces backfilled with substances like grease, half-inch polystyrene balls, low shear strength compliant materials and fuel oil (which floats on water but is unacceptable for environmental reasons). The result is that the surrounded pile experiences little or no uplift. However, it is very difficult to work out details so that this type of uplift-reducing measure lasts--it usually comes apart or fails.

A compressible 3/8-inch closed-cell seamless polyethylene foam jacket (dock bumper material) has been used with limited success. But ice has torn apart piles wrapped with styrofoam and duct tape. Piles wrapped with polyethylene sheets have also proven unsuccessful, as the sheets become ripped and torn. Attaching stove pipe metal also doesn't work, as it is torn off the piling under constant ice action.

A pile wrapping jacket made of high-density polyethylene with carbon black and ultraviolet stabilizer has shown some success. It is 0.06 inches (1.5mm) thick and smooth. If the very difficult detail of attaching it to an existing timber pile can be successfully worked out, the wrapping may prove to be an effective defense. Figure 11.4 shows a wrapped pile, but the ice is working on the wrapper's bottom edge--the wrapper was not long enough. It seems that the slightest protrusion, like a nail head or some other minor imperfection, can initiate the undoing of a wrapper. Wrappers with blind rivets instead of nails are now being tried.

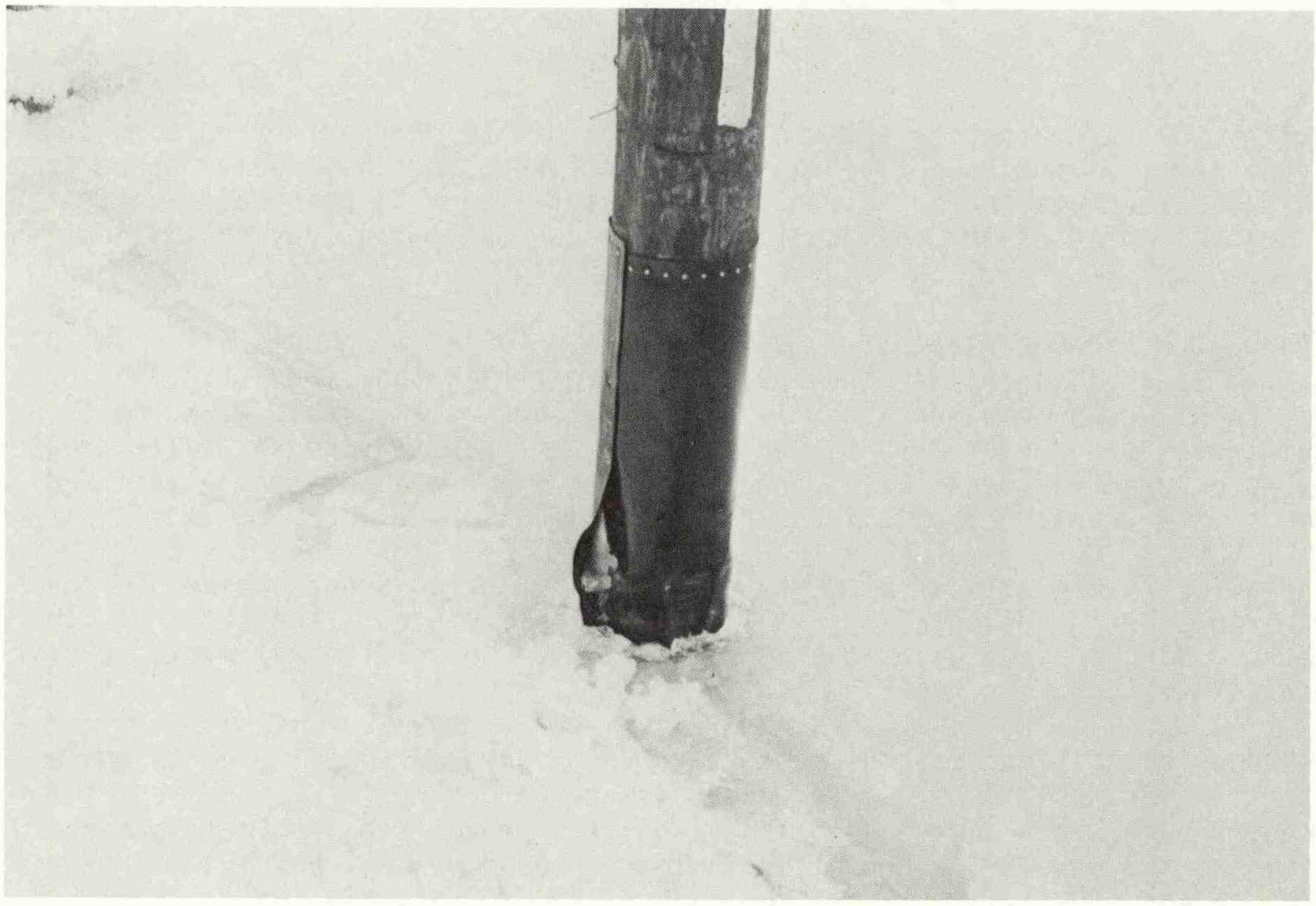


FIGURE 11.4: Black Polyethylene Pile Wrapper

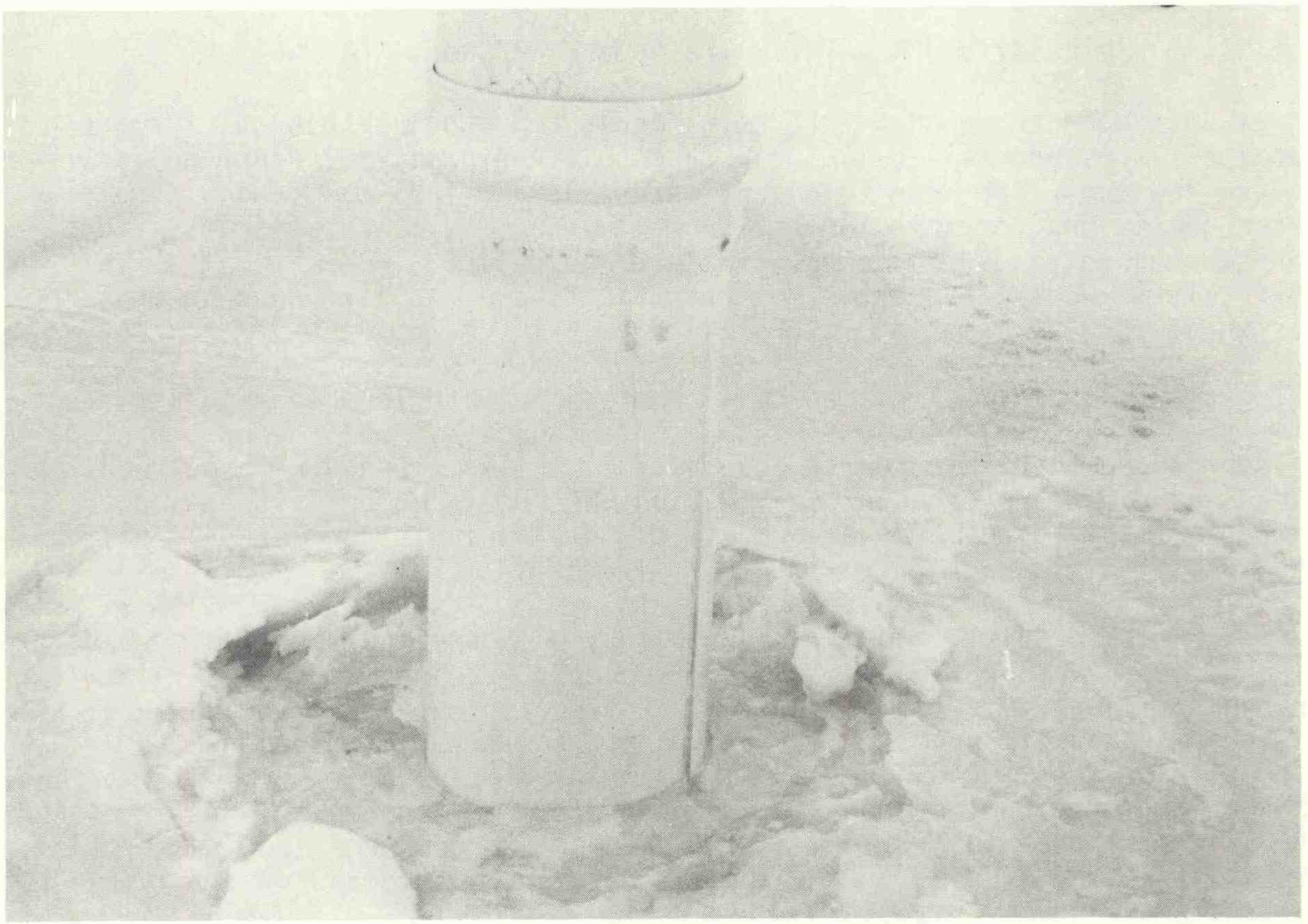


FIGURE 11.5: Concrete-Filled Polyethylene Pipe Pile

Similar to the wrapper is the container that can be dropped around a piling. In one test, a length of fiberglass pipe was used, and the annular space from the water level to the bottom of the ice was filled with pea gravel concrete. No rubble formed on this piling (indicating ice slippage), and no lifting occurred.

Figure 11.5 shows a piece of polyethylene pipe filled with concrete. The pipe is used as a piling rather than being a container around an existing piling. The ice slips on this surface. In contrast, look at Figure 11.6, which is an unprotected (and untreated) timber pile being ripped to shreds. Piles treated with pressure treatments like pentachlorophenol and creosote may also shred over time. Greasing a pile isn't a lasting solution, either.

Pipe piles filled with vermiculite or other insulation appear to reduce ice adhesion some in pullout tests of individual pile pieces extracted from ice sheets. Marine piles using geothermal heat-pipe principles may work, but the results have not been reported.

Low-adhesion coatings, discussed in Chapter 3, have not been tried to any extent in marinas. I believe progress can be made here. It should be remembered, though, that low-adhesion coatings will reduce the ice force but not eliminate it; however, a reduction in force should be most helpful. Epoxy paint is not a satisfactory coating--ice will ruin this type of coating in one season.

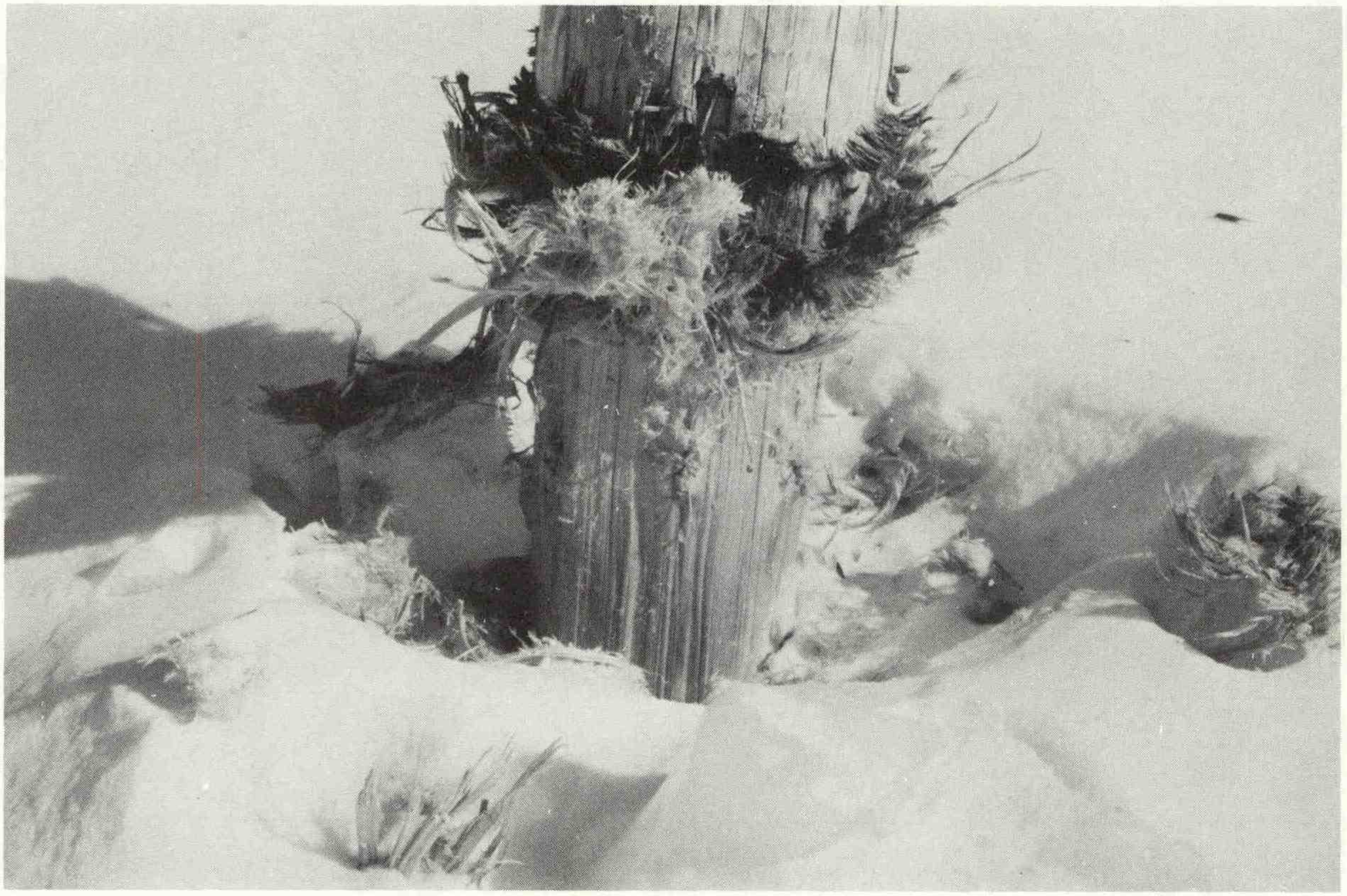


FIGURE 11.6: Untreated Timber Pile Being Shredded by Ice

RESISTING UPLIFT THROUGH GROUP ACTION

Another design strategy for reducing ice uplift is to fail the ice sheet; that is, cause it to crack predictably. In a general way, what can be expected in a given pier configuration and location in a marina basin is known--but exactly how to estimate the forces resulting from a forced cracking pattern is not known. Often, if not always, the crack pattern is an outcome of the dockage layout, which was based on some other criteria, such as number of slips, functional relationships and staged construction.

Figure 11.7 shows a simple long head pier in a U-shaped basin. The pier is supported on pairs of wood pilings. There are no finger piers, only mooring piles for every slip. Each year, a circumscribing crack connects the mooring piles and, in effect, perforates the sheet. It appears that this crack, produced by group action, protects or shields the head pier from large uplift forces. There also may be other cracks in the basin, such as the aisle crack shown.

Once such a group-action crack has been formed, it will likely exist all winter and will continue to serve as a plane of weakness in the ice sheet. Even rubble pieces overlying such a crack and partially refrozen together will show reflection cracking of the main sheet. The consolidated rubble mass will crack through from the sheet to the top of the rubble. These perforating mooring piles probably should not be designed for large uplift forces, especially if they are spaced 10 or 12 feet on centers. The ice can

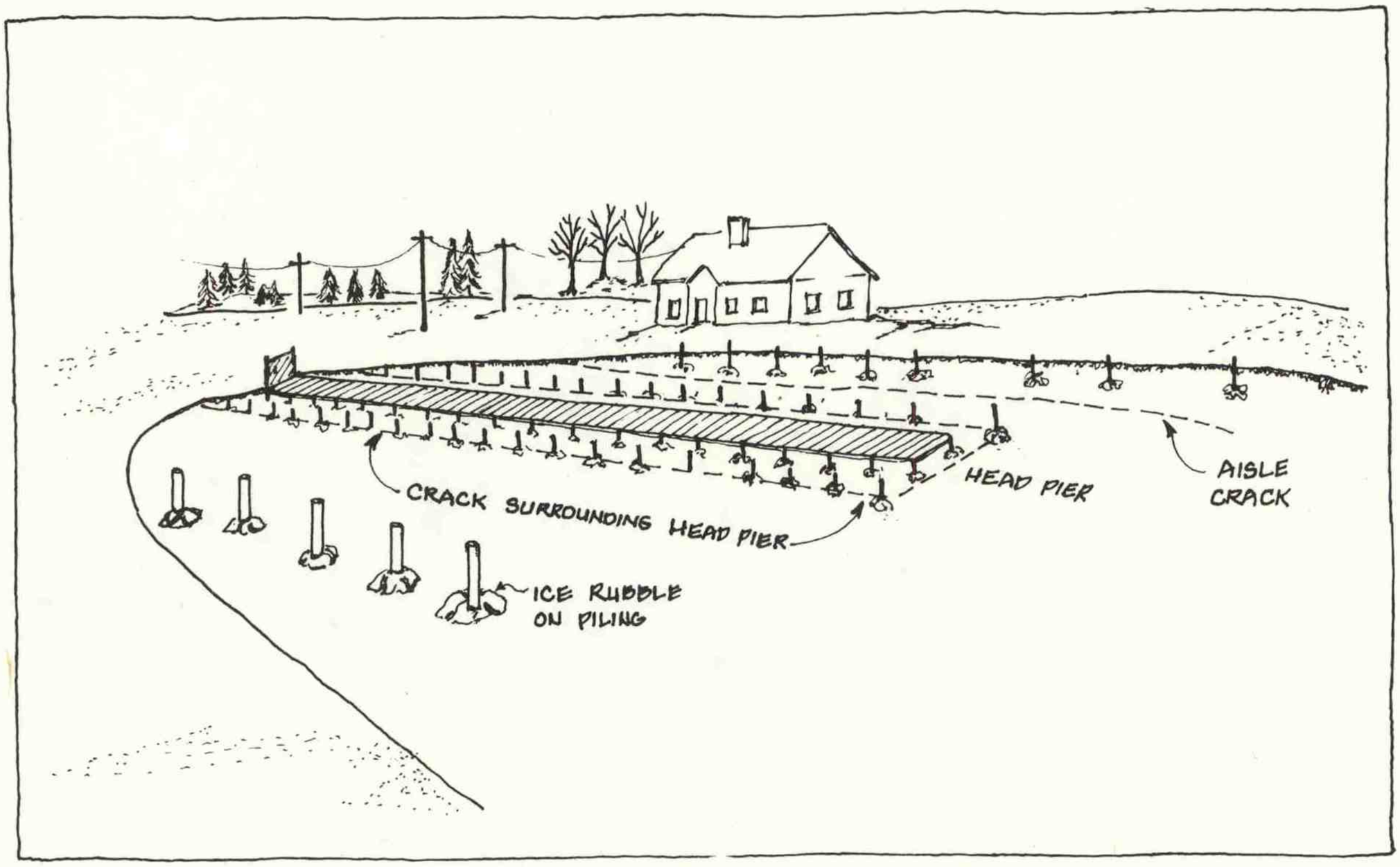


FIGURE 11.7: Group Action Cracking of Ice Sheet Around Head Pier

crack between pilings more than 30 feet apart. This cracking distance has been characterized by some to be about 10 times the sheet thickness. Perhaps so. Intuitively, it is known that the further apart the piles, the higher the uplift load. If the piles are too spread out, each pile would see its own "infinite" tributary area of ice.

Perhaps the penetration lengths of the piles under the head pier could be shortened. However, the monies saved by doing so may not warrant such a design decision when possible damage consequences are projected.

Figure 11.8 shows the end of a 12-foot-wide L-shaped head pier supported on steel H-piles. This pier end had major encircling cracks that connected all the pilings and also some smaller cracks that may not have completely penetrated the full ice sheet thickness. At the extremities, one 45° crack or two 90° cracks radiate out to the rest of the sheet. These corner cracks are quite common.

To observe and measure the cracks shown in Figure 11.8, it was necessary to shovel a lot of snow that had drifted into the dock area. This snow insulates the sheet and helps preserve the relieving crack. The same cracking pattern occurs annually at this site.

It should be obvious that the piles at the end of this L-shaped head pier are subject to much more loading than those along the pier and nearer to shore. Piles that can develop larger resistances would be required to cause the sheet to crack around the end. You do not want to break the bond between the ice and peripheral piles such as these.

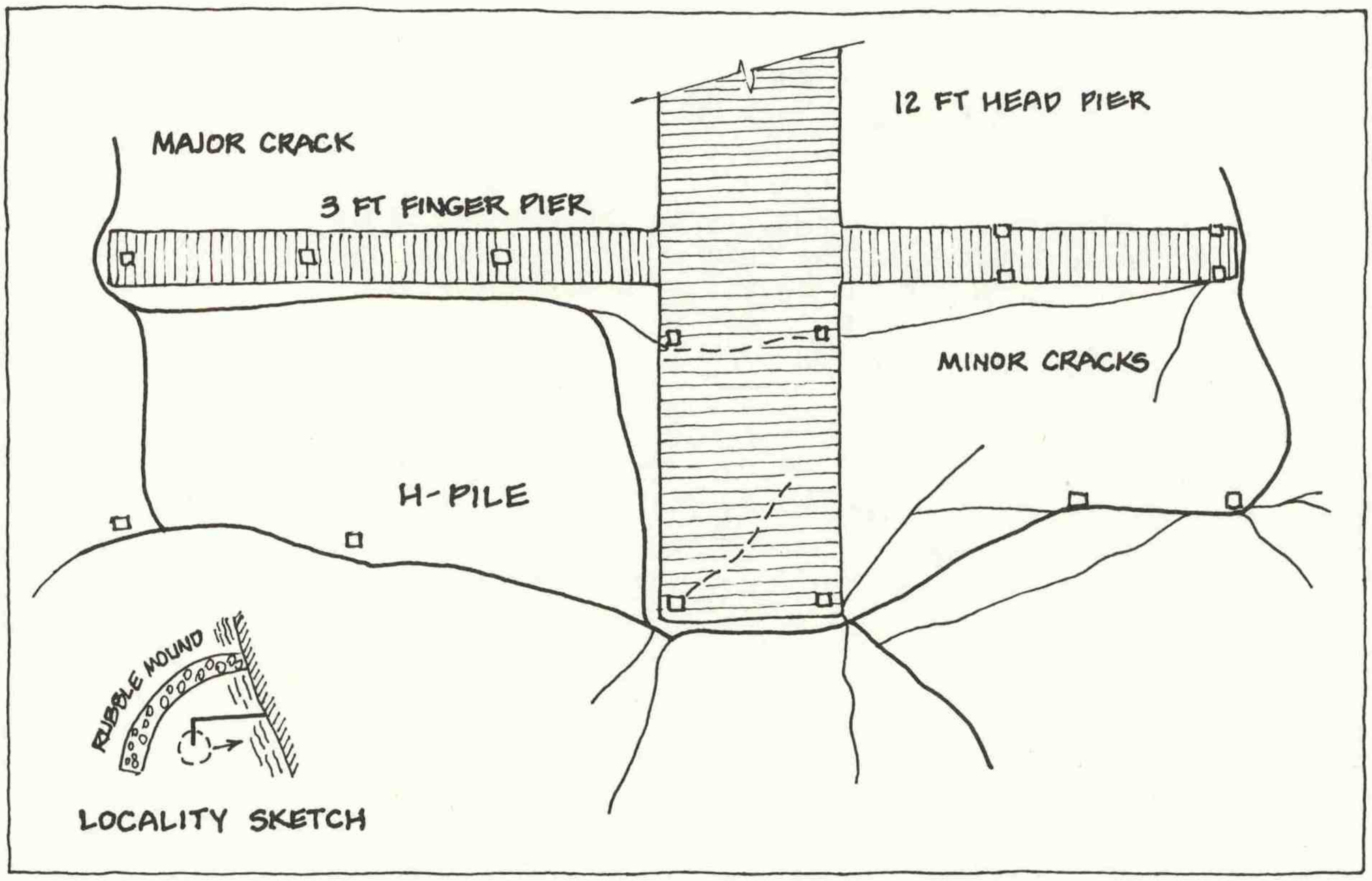


FIGURE 11.8: Group Action at End of L-shaped Pier

Unfortunately, how to quantify the group action is not known. The group action would be complex to analyze, and actual field force measurements or laboratory model studies have not been performed.

Some circumstances can exist where the outer piles will not see the largest forces, at least initially. An example would be a river where the ice forms first at the river banks and then towards the center. Lifting could occur early in its formation.

The next section discusses a recent and very interesting research project on the jacking of marina piles by Great Lakes ice. Directed by Prof. Tuncer Edil of the University of Wisconsin-Madison, the research examined practically all available data on the problem in an effort to try to come up with some numbers and design approaches.

ICE JACKING OF MARINA PILES IN THE GREAT LAKES (ROBLEE 1983)

The objective of the study by Roblee (1983) was to arrive at a systematic method for predicting likely levels of ice jacking damage for a given pile design and set of environmental conditions. Roblee reported that Wortley (1982) used an upper limit of design pullout force of 75 kips based on a back-calculated soil resistance, using undrained shear strength as the pile-soil adhesion in cohesive soils and a third of the effective overburden

stress as the skin resistance developed in granular soils. Roblee's study looked at the evaluation of minimum pullout resistance by investigating and analyzing 33 case histories of Great Lakes marina designs.

This study first established a methodology for characterizing the ice uplift force and the ultimate pullout resistance through the use of indices. It then applied these indices to the case studies to establish relationships that delineated between various levels of uplift damage.

Roblee attempted to characterize three general parameters thought to be of critical importance. These parameters were an index of the ice uplift potential, an index of the soil-pile uplift resistance and an index of the potential for water level fluctuation. Chapter 6 noted that Roblee concluded that any Great Lake harbor has enough seiche activity to create ice uplift problems. The index of water level fluctuation was nondiscerning, so I will concentrate on an index of ice uplift potential and an index of soil-pile uplift resistance.

Ice Uplift Index

To characterize ice uplift, Roblee used the elastic first-crack analysis presented in Chapter 3. Table 3.1 lists the minimum uplift load for a given ice thickness and approximate radius of load distribution. Roblee generally assumed 12 inches for this radius and computed an ice thickness based on Equation 6.1 setting the locality factor (α) equal to unity.

This method yielded a simple ice uplift index, which was then modified with a geometry factor. This factor recognized that a marina pile is not alone in an infinite sheet, so the ice force it receives should be reduced because of geometry. The effect of neighboring piles is to hold the ice sheet down, thereby sharing some of the potential uplift force generated by the sheet.

Before presenting Roblee's geometry factor—which agrees well with Hodek and Doud's (1975) uplift force measurement—it is necessary to introduce the concept of characteristic length. Characteristic length is also called flexural rigidity length, or the action radius, the latter term being most appropriate here.

The action radius is principally a function of the ice thickness but also varies some with the modulus of elasticity and Poisson's ratio. Michel (1978) gives the following simple approximation for the action radius:

$$l = 3.2h^{3/4} \qquad (11.1)$$

where: l = action radius, or characteristic length (feet)

h = ice thickness (inches)

For example, a 30-inch sheet has an action radius of 41 feet ($3.2 \times 30^{3/4}$).

Roblee defined the "infinite" affecting ice plate area as the area of a circle whose radius is equal to the radius of action, whereas the "actual" affecting area was defined as equal to the average of four circular areas whose radii are the lesser of the action radius or half the distance to the nearest bottom

fixed object. These four radii are mutually perpendicular and aligned with the principal orientation of the local pier structure (Figure 11.9). An exception to the half-the-distance rule was that a radius in a direction parallel to and between two rows of piles should not be greater than half the distance between the two rows (Fig. 11.9).

By multiplying the simple ice uplift by the computed geometry factor, a modified ice uplift index was obtained. This modified ice uplift index was the independent variable to be used with the dependent variable, pile pullout resistance. The modified ice uplift index becomes very small for "interior" piles, and the sum of these small indices should be set equal to the areal uplift capacity of the ice sheet.

Pile Pullout Resistance Index

Pile pullout resistance was estimated with static methods and Equation 5.11 for unit skin resistance as a function of soil adhesion and the effective normal stress. Roblee reported that some researchers (Mansur and Hunter 1970) have implied that upward shaft resistance is less than downward. However, because of a lack of consistent documentation and methodology, upward and downward skin friction were assumed to be equal to one another.

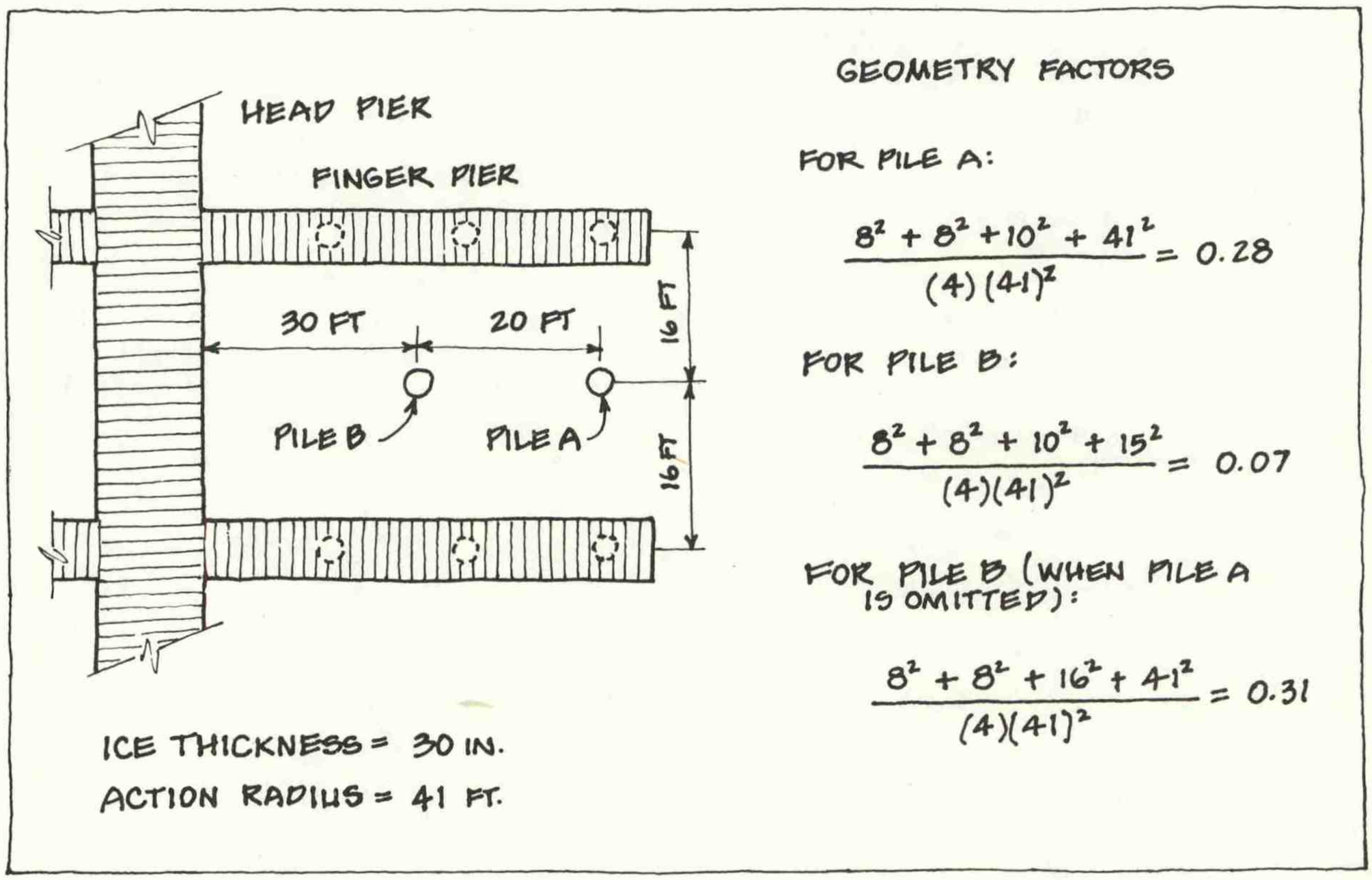


FIGURE 11.9: Example Calculations of Geometry Factors (after Roblee 1983)

The quality of geotechnical field information for the 33 studied sites varied from SPT N-values for some sites, to field descriptions only for most of the other sites. Soil strength parameters c and ϕ were inferred from N-value correlation tables from Bowles (1982) and corrected for depth with Peck et al. (1974) factors for sand.

Several analytical models were used to estimate the skin frictional resistance as functions of the strength parameters, the effects of overlying strata, and depth limitations on horizontal stress. Damages were characterized by the percentage of pilings that had lifted in a harbor and the amount they lifted.

Pullout Resistance vs. Ice Forces

Assuming unrestrained piles, Roblee found that the potential for damages could be estimated from the relation between the modified ice uplift index and the pile pullout resistance index. Damages could be expected if the pullout resistance was less than 1.8 to 2.3 times the uplift force, and damages would not be expected if the pullout resistance were 5.2 to 5.9 times the uplift force. For piles restrained by moment-resisting framing to other piles, Roblee found a somewhat reduced tendency for damages.

The range in these factors results from the several analytical models used. The models included methods of Tomlinson (1970), Vesic (1967) and others and involved combinations of either undrained cohesion or adhesion (ranging from 25 percent to 125 percent of undrained cohesion) for clays, and either uniformly increasing or limiting horizontal stresses for sands.

Roblee stated that the undrained value for cohesion is considered appropriate, as the ice sheet loads are not sustained. Also, due to the cyclic nature of the loading, either positive or negative pore pressures may develop in the fine grained granular soils. These pressures would cause uplift resistance values either more or less than those predicted by static methods. Cyclic work softening of the cohesive materials may also cause a reduced uplift resistance. (These analytic models and parameters will be examined in more detail later.)

My recommendation for estimating the potential for damages is to expect damage to occur if pullout resistance is less than 2 times the modified uplift force, and expect no damage if pullout resistance is more than 6 times the modified uplift force. For example, for pile "A" in Figure 11.9, assuming 30 inches of ice, and from Table 3.1, a radius of load distribution of 12 inches:

Total uplift force = 52 kips

Modified uplift force = (0.28)(52) = 14.5 kips

Damage if pullout resistance = (2)(14.5) = 29 kips or less

No damage if pullout resistance = (6)(14.5) = 87 kips or more

Obviously, these are only estimates or indications of what might happen, but the geometry factor approach and analysis of field data by Roblee (1983) indicate they are reasonable. Roblee suggests that 85 to 90 kips, rather than 75 kips as stated at the beginning of this section, would be a better first approximation of a limiting uplift force.

RESISTING UPLIFT WITH ANCHORAGES, ETC.

Attempts and suggestions on how to improve the holding capacity of a deep foundation are many. One marina increased the 8-inch pile size to 14 inches with a bolted connection below the bottom of the ice. A 14-inch shaft in the soil will obviously develop more uplift resistance than an 8-inch shaft, and to this extent this technique can be thought of as gaining additional anchorage. It could also be thought of in terms of reducing the surface on which ice can place its grip. Whichever, the use of variable-sized deep foundations is a step in the right direction.

Except for anchorages to rock, the other methods discussed here are not proven. They may or may not be worth the effort.

Rock Anchorages

One marina uses 12-inch pipe piles grouted 5 feet into rock as supports for 60-foot-long double-T precast docks. The designers believe the pullout resistance would be equal to the strength of the grout. According to Table 5.7, this grout strength would range between 200 and 400 psi. This would give a pullout resistance of 450 to 900 kips. (I think this should be adequate, don't you$_{\mu}$) However, if the resistance is estimated as suggested by Poulos and Davis (1980), you would have 5 percent of the 28-day concrete compressive strength, or 5 percent of the uniaxial compressive strength of the rock, whichever is smaller. The 5 percent concrete will usually control. Also, for tension, 70 percent of the value would be used. Therefore, if the concrete were 3,000-pound concrete, the pullout resistance would be about 100 psi (0.05 x 3,000 x 0.7), for a total resistance of about 225 kips.

Another marina successfully resists uplift with 7-foot rock sockets into which are set 8-inch pipe piles. The sockets develop lateral resistance for the piles. Uplift is developed with a 12-inch-long, 1.5-inch wedge anchors into the rock below the bottom of the socket.

Barb-Like Piles

Mushell (1970) has reported on "barbed" piles used in a Lake Huron marina. For the mooring piles, 1-inch steel plate uplift resister barbs were attached to the H-pile webs. During construction, two such barbed mooring piles--one penetrating 5 feet into site's medium fissured hard limestone and the other penetrating 14 feet--were subjected to a 50 kips test pull and did not move. During the first winter, which was harsh, some of the barbed mooring piles did come up. They penetrated only 5 to 6 feet into a nonfissured hard limestone instead of the desired 10 to 12 feet. During the ensuing years, pilings in this marina have lifted. This lifting is probably indicative of varying ice conditions over the years and the relative abilities of driving the barbed piles deeply.

Pieces of steel channel were welded to flanges of H-piles driven into a sand and clay Lake Superior harbor bottom. On a straight-shaft basis, these piles are estimated to have at least 50 kips uplift resistance. Of seven such piles, which are free-standing mooring piles, two have come up. The "barb" effect of the attached channel pieces is questionable.

Another type of barb is the so-called "friction ring." Rings measuring 1 inch by 2 inches have been welded on pipe shafts at 5- to 10-foot spacings. I have no data to indicate whether they help or not.

Freas and Anderson (1983) report on "friction fins" on pilings for an Alaskan dock. These steel fins were attached to the bottoms of pilings to increase the surface area, thus increasing the pullout resistance. This was done on two rows of batter pilings with 16-inch and 18-inch diameters. The fins were three half-inch by 6-inch plates attached to the pipe at 120 degrees for a length of 20 feet.

Freas and Anderson concluded that the use of radial fins on pipe pilings is an efficient method of increasing resistance of pipe piling driven into non-cohesive soils. Analysis of the pile driving records indicated that the increase in static resistance was directly proportional to the increase in surface area provided by the fins. Both sides of the fins were used in computing the surface areas. The use of these fins did not create any driving problems; rather, it is believed that the fins prevented the pilings from twisting during driving and thus may actually have driven straighter than piling without fins under similar conditions. (These are not barbs as such, and though no uplift tests were performed, the use of fins seems like a way to increase pullout resistance.)

Figure 11.10 shows barbed "spiles." A spile is a pile with attached spiles (stakes or attachments for support). The spiles consist of pieces of steel angle welded to the pipe shaft and raked so as to engage the soil when the spile is pulled through the soil. Some spiles have lifted, some have not.



FIGURE 11.10: Barbed Spiles

Vesic (1971) notes that very deep anchors do not fail in general shear, such as a cylinder or wedge of soil being pulled out of the ground. Experiments indicate that embedded objects can be moved vertically for considerable distances by producing a failure pattern similar to punching shear failure in deep foundations. Only after the embedded objects are pulled up to relatively shallow depths by a series of jerks (punching shears) can they eventually produce general shear failure and pull out a block of soil. I am skeptical about barbs on piles engaging much soil resistance, except in the immediate vicinity of the shaft to which they are attached.

Soil Anchorages

Chapter 5 noted that earth tiebacks are not a good choice for overcoming ice uplift forces because of costs and design loads. What about buried earth anchors?

Vesic (1971), in discussing breakout resistance of objects in the ocean bottom, emphasized the very complex nature of the phenomenon. No equation, no matter how elaborate, could be fully satisfactory for all varieties of soil conditions as well as methods of placement and types of objects to be pulled out.

For shallow anchors in loose sand or soft clay, the slip surface--though not clearly established--is close to being a vertical cylinder around the perimeter of the anchor. Thus, for objects imbedded in loose and compressible sediments, it is more reasonable to assume that the soil involved in breakout is essentially only soil immediately above the object. This may also prove to be a reasonable assumption in any case where the soil immediately surrounding the object is weakened by remolding. To get a torus or wedge shaped breakout, a relatively shallow anchor in dense sand or stiff silty clay is needed.

Meyerhof and Adams (1968) also discussed the breakout resistance problem. In the literature, uplift theories have generally been based on either a slip surface rising vertically from the edge of the footing, or a surface rising at 30 degrees from the vertical, forming a frustum. For the vertical surface theory, shear resistance along the sides of the plane or cylinder was calculated and added to the dead weight of the soil or concrete above the footing. For the 30-degree cone theory, only the dead weight within the frustum was usually considered.

Experience has shown that neither of these methods provides reliable uplift values. The cone method is usually conservative at shallow depth, but can be quite the opposite at large depth. It appears that the lack of agreement on uplift-capacity theory lies in the difficulty of predicting the geometry of the failure zone.

Andreadis et al. (1981) performed model tests of cylindrical, conical, plate and fluke sea anchors. They found that the mode of failure of an anchor subject to static loading is mainly controlled by the relative depth of embedment, soil relative density and anchor shape. Static breakout factors and relative anchor movements to failure increase sharply with the relative depth of embedment at shallow depths, tending to an approximate constant at greater depths. Uplift prediction for shallow anchors do not in general

yield good results, due mainly to the assumption of a unique failure surface shape for all soils. The complex soil-anchor interaction problem appears to be oversimplified by these methods.

Kulhawy et al. (1979) also performed uplift tests on model concrete shafts in sands. They, too, found that the truncated cone model does not predict measured capacities well. The same is true for a curved breakout surface. They found that the shear surfaces along which the shafts failed were cylindrical and on the order of a quarter-inch out from the soil-concrete interface in the soil. For one test in a dense saturated sand, the pullout was 4,000 pounds, which gave a computed skin friction value of 470 pounds per square foot.

To summarize, in computing the uplift resistance of buried objects in a harbor bottom, you will not know with any assurance what failure surface to assume. You will find the breakout loads are small for reasonably sized anchors. You will also find the resistances developed inadequate to handle ice uplift forces. Therefore, soil anchors as well as soil tiebacks seem inappropriate. You are left with deep piles developing skin frictional resistance--which is the subject of the next chapter.

12. Design for Deep Foundations for Structures —Part II (Pilings)

Pile foundation design was introduced in Chapter 5, which concentrated on single piles rather than groups. I noted that estimating skin friction capacity was complicated and opinions on it differed, and that lateral loads, though occasionally bothersome, are not paramount. This chapter first discusses friction piles for uplift resistance and then lateral loads.

I reviewed the literature on uplift of friction piles in an effort to glean whatever might be helpful to small-craft harbor piling design. Despite the large body of literature on the subject, no clear-cut, reliable methods for predicting the uplift of friction piles in sands and clays exists.

A good place to begin is with Meyerhof's (1976) Terzaghi Lecture. Meyerhof has worked with bearing capacity of ice and soil. The Terzaghi Lecture is a prestigious invited lecture sponsored by the American Society of Civil Engineers at their annual meeting.

BEARING CAPACITY (SKIN FRICTION) OF PILE FOUNDATIONS (MEYERHOF 1976)

The skin friction of piles in cohesionless soils varies widely, because it depends on the stress history of the soil and the shape and roughness of the piles, among other factors.

For piles in saturated clay and plastic silt, several months after the piles are installed, the shaft resistance is governed by the drained shear strength of the remolded soil. The skin friction of driven and bored piles can be estimated from skin fricton factors with respect to the average effective overburden pressure, provided the at-rest earth pressure coefficient of the deposit is known. In normally consolidated soft and medium clays and silts, the positive and negative skin friction factors of driven piles have a similar value, and they decrease with pile length due to progressive soil failure at the pile shaft.

For stiff overconsolidated clay, the skin fricton factor can vary widely with the degree of overconsolidation of the soil, pile shape, method of pile installation and other factors. If the value of the at-rest earth pressure coefficient of the clay is known, the estimated skin fricton will provide a lower limit for driven piles, and an upper limit for bored piles. In other cases, the skin friction factor can be only very roughly estimated from empirical correlations with the average undrained shear strength of the clay for driven and bored piles of various embedded lengths.

Friction Piles in Sand

The average ultimate unit skin friction (f_s) in homogeneous sand may be expressed by:

$$f_s = K_s \sigma'_v \tan \delta \leq f_1 \qquad (12.1)$$

where: f_s = the average ultimate unit skin friction

K_s = the average coefficient of earth pressure on pile shaft

σ'_v = the average effective overburden pressure along the shaft

δ = angle of skin friction

f_l = the limiting value of average unit skin friction for critical depth and beyond

Below the critical depth, the average skin friction remains practically constant in a homogeneous sand deposit due to effects of soil compressibility, crushing, arching and other factors. Since no satisfactory method of analysis of pile behavior below the critical depth is available, an empirical approach is necessary at present.

It is difficult to estimate the skin friction and particularly the earth pressure coefficient (K_s) on the basis of the friction angle of the sand and the method of pile installation. Though a rough estimate of the limiting unit skin friction (f_l) can be obtained from the results of penetration tests, reliable values of K_s and f_l only can be deduced from load tests on piles at the given site.

An analysis of load tests on instrumented piles driven into sand shows that the local ultimate unit skin friction increases with depth only along the upper portion of the pile to a maximum, and then it decreases to a minimum at the pile point. The average value of the ultimate unit skin friction is denoted by f_s.

Accordingly, the corresponding local coefficient of earth pressure on the shaft decreases with depth along the pile from a maximum near the top--where the local coefficient may approach the passive earth pressure coefficient (K_p)--to a minimum near the pile point, where the local coefficient may be less than the initial earth pressure coefficient (K_o). The average ultimate value of the local coefficient is denoted by K_s.

An analysis of the few available results of load tests on short piles above the critical depth in generally homogeneous normally consolidated sand shows that the value of K_s for a given initial friction angle (ϕ) can scatter considerably. The value of K_s varies from a lower limit of roughly K_o for bored piles or piles jacked into loose sand, to about four times this value or more for piles driven into dense sand due to dilatancy effects and other factors.

Conventional shaft capacity theory in terms of K_s cannot be used for piles longer than about 15 to 20 pile diameters, because the corresponding value of f_s becomes practically independent of the average overburden pressure along the shaft and is given by f_l.

As would be expected, the ultimate unit skin friction increases with the volume of displaced soil; therefore, bored piles or driven piles with a small displacement, such as H-piles, have a smaller average skin friction than large displacement piles.

Equation 12.2 gives a suggested average ultimate skin friction f_s for driven displacement piles. Since the observed average ultimate unit skin friction of

piles generally exceeds that given by Equation 12.2, the equation will also provide reasonable estimates for H-piles, as no further reduction needs be made for such piles.

$$f_s = \frac{\bar{N}}{50} \tag{12.2}$$

where: f_s = the average ultimate skin friction (tons per square foot; 1.0 max)

$\bar{N}$ = the average SPT resistance within embedded length of pile

The value of f_s in Equation 12.2 represents the limiting value of f_1.

The ultimate skin friction of a driven cylindrical pile also can be estimated by the unit resistance of a local friciton sleeve of a static penetrometer. However, as mentioned in Chapter 4, the friction cone penetrometer is not often used in the United States.

When piles longer than 15 to 20 pile diameters penetrate through a weak stratum into a thick firm deposit of cohesionless soil, the average ultimate unit skin friction in each cohesionless stratum roughly can be estimated directly from the limiting values of f_1 using average properties of each layer.

Friction Piles in Clay

The average ultimate unit skin friction (f_s) or the equivalent ultimate shaft adhesion (c_a) in homogeneous saturated clay is usually expressed:

$$c_a = \alpha c_u \tag{12.3}$$

where: c_a = the ultimate shaft adhesion

α = the empirical adhesion coefficient

c_u = the undrained shear strength

The coefficient α (a different α from that in Equation 6.1) depends on the nature and strength of clay, dimensions and method of installation of pile, time effects and other factors. The values of α vary widely, and they decrease rapidly with increasing shear strength.

For driven piles, the values of α range on average roughly from unity for soft clay to one-half or less for stiff clay. This value of α, which represents a maximum shear adhesion (c_a) of roughly 1 ton per square foot indicates that the drained shear strength of the clay would usually govern shaft adhesion.

Thus, immediately after pile driving, the shaft adhesion is closely given by the undrained shear strength of remolded clay. However, at later stages and particularly at the end of construction, the shaft resistance of piles will be governed by the effective drained shear strength parameters c' and ϕ', of

remolded clay failing close to the shaft. The corresponding effective unit skin friction (f_s) in homogeneous clay may then be taken as:

$$f_s = c' + K_s\sigma_v' \tan \phi' \leq c_u \qquad (12.4)$$

where: f_s = the average ultimate unit skin friction

c', ϕ' = the effective drained shear strength parameters

K_s = the average coefficient of earth pressure on pile shaft

σ_v' = the average effective overburden pressure along the shaft

c_u = the undrained shear strength

Equation 12.4 assumes that the excess pore water pressure sometime after installation and loading of the pile is negligible compared with the effective overburden pressure. The initial excess pore pressure induced by pile loading to failure generally appears to be in the range of only about $0.2c_u$ to $0.5c_u$ at the shaft.

Therefore, it may be concluded that the ultimate skin friction of piles in saturated clay can be estimated approximately from the drained shear strength of remolded soil, for which cohesion usually may be taken as zero. On this basis, Equation 12.4 may be written:

$$f_s = \beta\sigma_v' \leq c_u \qquad (12.5)$$

where: β = the skin friction factor

which equals: $\beta = K_s \tan \phi'$ (12.6)

This approach is supported by an analysis of load tests on instrumented piles driven into clay where the local ultimate unit skin friction is found to increase roughly in direct proportion to depth or the effective overburden pressure along most of the shaft.

For piles driven into saturated soft clay, the ultimate coefficient of earth pressure on the shaft ((K_s, Eq. 12.6) may be expected to be close to that of the earth pressure at rest (K_o), as was found previously for loose sand.

For this condition, and for homogeneous normally consolidated clay where $K_o = 1 - \sin \phi$ (Table 5.6) approximately, the skin friction factor β may be represented by:

$$\beta = (1 - \sin \phi') \tan \phi' \qquad (12.7)$$

--which would vary theoretically from about 0.2 to 0.3 for a typical range of ϕ' for clay. Analysis of the skin friction of piles driven into soft and medium clays show that the factor β decreases with the length of piles from a range of about 0.25 to 0.5 for short piles to a range of about 0.1 to 0.25 for very long piles. This decrease of β with greater pile length may be explained by progressive mobilization of the maximum skin friciton due to compression of long piles, for which the effective friction angle of the clay at the shaft may approach the residual value of roughly one-half of the peak angle, ϕ'.

For preliminary estimates, the skin fricton factor (β) of piles driven into soft and medium clays may be taken as about 0.3, provided the depth of penetration is not greater than about 50 ft. The value of β should be reduced for longer piles to about 0.15 for a depth of penetration exceeding 200 feet.

The ultimate skin friction of piles installed in stiff saturated clay also can be estimated from Equations 12.5 and 12.6 on the basis of drained shear strength of remolded soil, provided that the coefficient K_s is known from previous pile load tests or the value of K_o can be estimted from field or laboratory tests, from which roughly:

$$K_o = (1 - \sin \phi)(OCR)^{1/2} \qquad (12.8)$$

where: OCR = the overconsolidation ratio

Thus--for stiff fissured overconsolidation London clay with a value of K_o ranging from about 3 at shallow depth to unity at great depth--analysis of pile load tests shows that for driven piles, the average value of K_s varies from roughly K_o to more than $2K_o$, corresponding to a range of β from roughly unity to over 2.

The analysis indicates that an average value of K_s equal to K_o tends to underestimate the skin friction of driven piles in London clay due to the corresponding change in the initial horizontal stress in the ground near the shaft by pile installation.

On the other hand, if the value of K_o of stiff clay is not known, the corresponding skin friction factor only can be estimated within wide limits. Approximate values of K_o lie between limits of about 0.5 governing the skin friction of soft normally consolidated clay and about 3 for short piles in London clay. On this basis the coefficient K_s for driven piles in stiff clay is roughly 1.5 times K_o.

The effective horizontal pressure on the shaft of piles driven into clay also can be estimated from semi-empirical relationships based on either the average undrained shear strength and effective overburden pressure along the shaft, or more simply from the undrained shear strength c_u only. Thus, the latter approach--and a correlation between the plasticity index and values of c_u/σ_v' and ϕ of normally consolidated clay, indicate that K_o of such clay varies roughly between 1.5 (c_u/σ_v') for highly plastic clay, to over 2 (c_u/σ_v') for clay of low plasticity. Taking the lower value, the unit skin friction of piles driven into saturated clay then conservatively may be expressed as:

$$f_s = 1.5\, c_u \tan \phi' \qquad (12.9)$$

in which ϕ' would be expected to decrease from the peak value for short piles to the residual value for very long piles, as mentioned previously. Equation 12.9 indicates that the ultimate effective horizontal stress on the shaft of a driven pile in clay may be considerably smaller than the initial horizontal stress after installation mentioned previously.

The expression, which is related to Equation 12.3, is found to be in reasonable agreement with some load tests in stiff clay. It may be used when the value of K_o of overconsolidated clay is not known.

Summary and Review: Friction Piles in Sand and in Clay

Before critiquing Meyerhof's "drained" approach, I'll summarize it in outline form first for homogeneous sands and then for homogeneous saturated clays:

Homogeneous sand

Depth less than 15 to 20 diameters:

$$f_s = K_s \sigma'_v \tan \delta \leq f_1 \tag{12.1}$$

where: K_s and f_1 are from site load tests,
or $K_s = K_o$ for loose sand,
or $K_s = 4K_o$ for dense sand,
or K_s = more than K_o and less than K_p,
and tan δ from Table 5.9 or other source,
and K_o from Table 5.6 or other source

Depth greater than 15 to 20 diameters:

$$f_s = \frac{\overline{N}}{50} \quad \text{ton per square foot, 1.0 maximum} \tag{12.2}$$

Homogeneous saturated clay:

$$f_s = \beta \sigma'_v \leq c_u \tag{12.6}$$

$$\beta = K_s \tan \phi'$$

Soft to medium clay and normally consolidated clay (where $K_o = 1 - \sin \phi$):

$\beta = 0.3$ depth less than 50 feet
$\beta = 0.15$ depth more than 200 feet

Stiff clay (if K_s is not known from load test):

$$K_o = (1 - \sin \phi)(OCR)^{1/2} = 1.0 \text{ to } 3.0 \tag{12.8}$$
$K_s = K_o$ to $2.0^+ K_o$ ($\beta = 1.0$ to 2.0^+)

Stiff clay (if K_o is not known):

K_o = 0.5 (soft normally consolidated) to 3.0 (short piles in London clay), and therefore:

$K_s = 1.5 K_o$

Conservative semi-empirical relationship:

$$f_s = 1.5 c_u \tan \phi' \text{ (for clays)} \tag{12.9}$$

DRAINED EFFECTIVE STRESS SKIN FRICTION

Meyerhof (1976) does not specifically deal with uplift skin friciton; I have only cited portions of the paper dealing with the frictional component of ultimate bearing capacity. Many others besides Meyerhof discuss and/or advocate the use of drained effective strength parameters. Some comments and data from other papers are presented in this section.

Vesic (1977)

Solid evidence shows that the skin resistance of piles is governed by the effective stress conditions around the shaft. The increase of bearing capacity of friction piles in clay is essentially a phenomenon of radial consolidation of clay.

Shaft friction of bored piles in stiff-to-hard clays, as well as the shaft friction of all piles in soft clays, can be determined from:

$$f_s = K_s \tan \phi' \sigma_v' \qquad \text{(Equations 12.5 and 12.1 with } \tan \delta = \tan \phi')$$

For piles in normally consolidated clays inducing no appreciable change in lateral ground stress conditions, assume that $K_s = K_o = 1 - \sin \phi'$, or that $\beta = (1 - \sin \phi')\tan \phi'$ (Eq. 12.7), where ϕ' represents the angle of shearing resistance of remolded clay in drained conditions. When $15^\circ < \phi' < 30^\circ$, β should vary between 0.20 and 0.29. Other comparisons indicate that the equation or average value of 0.24 may be appropriate for tension piles or negative skin friction. There also may be a tendency for lower β-values for very long piles and higher β-values for shorter piles.

The behavior of piles in stiff clay is frictional in nature and fundamentally similar to that of piles in dense sand.

Measured values of β for driven piles in very dense sand are similar to those for piles in stiff clay, decreasing from about 2 for very short piles to about 0.4 for very long piles. In loose sands, β can be as low as 0.1 with no obvious decrease with increasing pile length. Available test data for piles in medium-to-dense sand seem to suggest that, after some penetration into the sand stratum, f_s reaches a quasiconstant limit value.

Poulos and Davis (1980)

For piles in stiff overconsolidated clays, the drained load capacity--rather than the undrained--may be the critical value, and some have advocated an effective stress approach in such cases. The analyses infer that the drained angle of friction between the pile and the soil can be taken as the drained effective stress friction angle of the clay.

Bowles (1982)

Some major problems not resolved with the β-method include what to use as the limiting value for f_s, since σ_v' can become quite large for long piles; also, there is some question as to whether one should use $\tan \phi'$ or $\tan \delta$ to describe friction. Some researchers have found that the actual fricton angle between the soil and pile material is on the order of 0.5 to 0.75 ϕ'.

Burland (1973)

Dangers exist in the purely theoretical approach to pile behavior, as exist in empiricism which takes no account of well-established fundamentals. The art of ground engineering lies in the ability to combine the established principles of soil mechanics with experience and judgment. (This often-cited Burland paper outlines an approach to the calculation of the shaft resistance of piles in clay using simple effective stress principles.)

It is customary to relate the average shaft adhesion (c_a) to the mean undrained strength down the shaft (c_u) by an empirical coefficient α equal to c_a/c_u and ranging from 0.3 to 1.5.

Whereas the use of the undrained strength for calculating the end bearing capacity of a pile ($q_{ult} = 9\ c_u$; Eq. 5.5c) appears justified, there seems to be little fundamental justification for relating shaft adhesion to undrained strength for the following reasons: (1) the major shear distortion is confined to a relatively thin zone around the pile shaft (drainage either to or from this narrow zone will therefore take place rapidly during loading); (2) the installation of a pile, whether driven or cast in place, inevitably disturbs and remolds the ground adjacent to the pile shaft; and (3) quite apart from the disturbance caused by the pile, there is no simple relationship between the undrained strength and the drained strength of the ground.

Empirical relationships between c_a and c_u are undoubtedly important in design—provided they are applied to the same pile type and similar ground conditions for which they were established. But there are dangers in extrapolating empirical relationships to new and untried situations.

Burland recommends a design value of $\beta = 0.3$. Among the load test data analyzed are a dozen concrete, steel and timber piles driven in several types of clay in which β ranged from 0.25 to 0.40, averaging about 0.32.

Burland notes that an effective stress approach to shaft friction in stiff clays is more complex than for soft clays. The central problem is to estimate the value of K_s. In the undisturbed state, the value of K_s ($= K_o$) for heavily overconsolidated clay varies with depth, and it can have values as high as 3 near the surface, decreasing to less than 1 at great depth.

Without attempting to explain the detailed behavior of driven piles in stiff clay, Burland developed a lower-bound envelope of average shaft friction in the first 30 feet of penetration depth. Burland used test data reported in the literature. The slope of the bound is approximately 60 pounds per square foot per foot. For example, at a penetration of 15 feet, the average shaft friction is 900 pounds per square foot for the full depth of penetration.

The simple approach outlined by Burland is not intended to replace the traditional empirical method of estimating shaft friction, particularly in the stiffer materials. However, it is useful for preliminary design.

Flaate and Selnes (1977)

Flaate and Selnes investigated a large amount of data from full-scale load tests on driven piles in soft to medium clays. Forty-four cases were used, involving mostly timber piles and a few concrete and steel piles. Penetration

depths ranged from 18 to 80 feet. Flaate and Selne concluded that formulas based on effective stress analysis give better estimates of average skin friction than estimates based on undrained shear strength, and that the magnitude of the side friction does not seem to depend on the pile material. A formula for f_s is given together with a simplification based upon relationships between the plasticity index and various soil properties:

$$f_s = u_1 \left[(0.3 - 0.001I_p) + (OCR)^{1/2}(\sigma_v') + (0.008I_p)(c_u)\right] \qquad (12.10a)$$

$$f_s = u_1 \, (0.4)(OCR)^{1/2}(\sigma_v') \qquad (12.10b)$$

where: f_s = the average ultimate unit skin friction
I_p = the plasticity index (PI)
OCR = the overconsolidation ratio
σ_v' = the average effective overburden pressure
c_u = the undrained shear strength

and: u_1 = a length reduction factor for mobilized side friction with depth
$= \frac{L + 66}{2L + 66}$

where: L = embedment length (feet)

Esrig and Kirby (1979)

Esrig and Kirby discuss advances in general effective stress methods for the prediction of axial capacity of driven piles in clay. They present preliminary design charts of $\beta(f_s/\sigma_v')$ vs. OCR. Their basic premises are that the critical failure surface is at or near to the pile-soil interface, and that the shearing resistance (f_s) is a function of the normal effective stress (σ_v') on, and the effective stress shear strength parameters appropriate for, the failure surface. They assume that the effective shear stress parameters can be measured.

Esrig and Kirby view the determination of the effective stress at the pile-soil interface as a problem of addition: to the initial state of stress in the ground prior to pile driving, add the change in stress due to pile driving, the change in stress due to reconsolidation after pile driving and the change in stress due to pile loading, and this yields the effective stress on the failure surface.

They found 13 reports of pile load tests in the engineering literature that included enough soils data to permit reasonable inferences to be drawn about soil properties and stress histories. Of these, nine were reports for piles driven into normally consolidated clays, and four were for piles driven into heavily overconsolidated clays.

A partial summary of their β-charts, which show some variation for the plasticity index and type of pile (displacement and partial displacement) is as follows: for normally consolidated clays (OCR = 1), β = 0.25 to 0.34; for OCR = 4, β = 0.58 to 0.72, and for OCR = 8, β = 0.90 to 1.22.

Blanchet et al. (1980)

Blanchet et al. conducted a detailed load test program for friction piles (timber, steel pipe with closed end, and precast) on the north shore of the St. Lawrence River valley. The test study showed that pore pressures influenced by pile driving are related to the preconsolidation of the clay and that they are much larger for tapered piles. It was demonstrated that the effective stress analysis method proposed by Meyerhof (1976) adequately determines the ultimate pile bearing capacity, but that the timber pile taper in effect doubles the skin friction.

In a discussion of the paper, Meyerhof suggests skin friction is increased maybe 1.3, not double. (Uplift extraction, not bearing capacity, is the application here, however, so the taper effect is not relevant.)

Blanchet et al. state that for straight-walled rough piles (timber or concrete) that the angle of skin friction (δ) can be taken as equal to the effective friction angle of the clay (ϕ'), and for steel piles δ is reduced to $\tan \delta = (0.75)\tan \phi'$. The test results suggest that skin friction is a phenomenon of the same nature in granular and in cohesive soils.

Tomlinson's (1957, 1970) total stress method based on undrained shear strength is not relevant to the determination of skin friction acting on piles in clay. The conclusion was that the skin friction on piles in soft and normally consolidated clay may be estimated from the effective stress analysis, using the same principles as those applied to sand. For straight piles:

$$f_s = (1-\sin \phi')(\tan \delta)(\sigma_v')$$

Kraft et al. (1981)

Kraft et al. examined the effect of length on frictional capacity in clays, using current technology and philosophy. The influence of the effect of length suggested by earlier case studies has significant economic impact on offshore structures that are suported on 3- to 12-foot diameter open-end pipe piles driven to penetrations of 200 to 500 feet. In their study, the α, β, and λ approaches were correlated to both pile length and pile-soil stiffness (the β approach is discussed later). The pile data studied had diameters of 6 to 30 inches, lengths of 8 to 333 feet and included pipe piles, concrete piles and timber piles.

The authors discuss their subjective estimates, which reflect their opinions of the present state of the art. There are relative uncertainty factors for pile capacity and soil conditions where they had previous experience and where the piles were installed with good construction practices. Considering these factors and the limitations of the analytic models, they said measured capacities in most cases would be from 0.7 to 1.3 times the predicted capacities. These factors include:

-- soil variability and disturbance of the soil surrounding the pile due to pile installation;

-- distribution and magnitude of ultimate skin friction values in heavily overconsolidated clays;

-- changes in the soil properties with time (consolidation, thixotropic and seasonal effects, and load history);

-- distribution and magnitude of residual stresses in the pile, and

-- pile installation details (displacement characteristics, time delays and sequence of installation).

Some methods may be more appropriate for certain soil conditions than others. The difficulty lies in establishing which method is the most appropriate for which conditions. The first step is to recognize the limitations of the available pile capacity methods. If a method is not appropriate, or the uncertainty is unacceptably large, load tests may be needed to guide and confirm design.

Though recent research on effective stress methods with cavity expansion and reconsolidation offer promise, they are not yet ready for use as a standard design tool. Nevertheless, these methods can be a useful analytic tool to gain insights useful in the decision-making process. Still, the need still exists for quality load tests on piles to improve design procedures and increase the reliability and cost-effectiveness of marine pile foundations.

Kraft et al. concluded that a length effect exists on the friction capacity of piles in clay. The magnitude of the length effect influence is affected by soil stress-strain behavior, pile-soil stiffness, lateral pile movements during installation, overconsolidation ratio and other factors considered to be of secondary importance.

Parameter analyses demonstrate that average friction for a pile is not likely to be related to a single parameter representing pile-soil stiffness and soil stress-strain characteristics. A statistical analysis of the available data show that the λ concept--either in terms of pile penetration or of pile-soil stiffness--is the most consistent and reliable single method for computing the axial capacity of piles in both normally consolidated and overconsolidated soils.

Other methods are also adequate. The pile lengths are longer than would be installed in a small-craft marina, and Kraft et al. note that the λ method is not intended for short piles in normally consolidated clays and that it over-predicts capacity for penetrations of less than 50 feet (the lengths of interest here).

Mayne and Kulhawy (1982)

Mayne and Kulhawy investigated the relationships between K_o and OCR for primary loading-unloading-reloading conditions. Virgin loading of a soil deposit is associated with sedimentation and normally consolidated conditions. During this process, the at-rest pressure coefficient K_o (ratio of horizontal to vertical effective stress: σ'_h/σ'_v) remains constant.

Any reduction in the effective overburden stress results in overconsolidation of the soil, which is expressed as the overconsolidation ratio (OCR--the ratio of the preconsolidated vertical effective stress at time past to existing vertical effective stress at time present: past σ'_v / present σ'_v). Reducing the overburden stress causes overconsolidation and results in an OCR of more

than 1. Mechanisms causing an overconsolidation effect include erosion, excavation, a rise in the groundwater table, removal of surcharge loads, etc. During unloading, the overconsolidation ratio has a pronounced effect on the value of K_o.

If loading is reapplied after the unloading rebound, a reload relationship results. Such reloading could be caused by seasonal factors like water table fluctuations.

Mayne and Kulhawy's study reviews more than 170 different soils in order to characterize the various relationships (during loading-unloading reloading) between K_o and OCR.

Their statistical analyses confimed Jaky's (1948) simple and widely used formula for normally consolidated soils, $K_o = 1 - \sin \phi'$. Other investigators have suggested that K_o may correlate with liquid limit, plastic limit, clay fraction, uniformity coefficient, void ratio or other index properties of soil. The data collected by Mayne and Kulhawy did not confirm any of these relationships, but did support the Jaky formula as valid for cohesive soils and moderately valid for cohesionless soils.

For overconsolidation from unloading, the data support the following suggested relationship:

$$K_o \text{ (overconsolidated soils)} = (1 - \sin \phi') \, OCR^{\sin \phi'} \qquad (12.11)$$

Mayne and Kulhawy noted that the maximum value of K_o may be assumed to be equal to the coefficient of passive earth pressure (K_p), and they also presented an equation for reloading based on what little published data there is for the behavior of K_o during this phase (see Mayne and Kulhawy 1982).

Discussion of the paper has noted that K_o for normally consolidated soils is moderately valid, as suggested by the authors, because of the difficulty in determining the relevant value of ϕ' due to the nonlinear strength envelopes of many sands and because of differences in dilatancy in sands vs. clays. It is also noted that the suggested relationships between K_o and OCR are invalid for fibrous materials like peats and paper mill wastes (which doesn't matter here, as no one would build a marina in a waste pond or swamp anyway).

Kraft (1982)

The development of effective stress approaches to compute shaft friction provides an improved understanding of the factors that effect shaft friction and should lead to improved techniques in the future. The subject of axial pile capacity--especially theoretical treatments in terms of effective stresses—lends itself to stimulating differences in professional opinions.

Kraft compared computed results for four effective stress models (ESMs) developed in the last five years and two conventional methods with values measured on 10 piles at two field and one laboratory test sites. Each of the effective stress methods attempts to track the effective stresses in the soil as they change from the initial free-field stress state to the states immediately after pile installation, during and after consolidation, and during pile loading.

These comparisons showed that the effective stress methods are on a par with conventional approaches. This is not at all surprising. The developing ESMs can be expressed in α and β with the introduction of linking equations and several new soil parameters in addition to undrained shear strength or effective overburden stress. These complicated equations won't be presented here, but note Kraft's comments that the additional parameters are elusive and influenced by other soil parameters, installation details and pile details (such as pile length and tip conditions). Further improvements in the theoretical ESMs for axial pile capacity will have to be directed toward these new parameters and improving the accuracy of the stress-strain model of the soil.

Effective stress methods cannot yet be used alone as design tools because the available pile load test data with well-documented data on soil properties are inadequate to test fully the validity of the ESM. In fact, the data are inadequate to test fully the validity of conventional approaches, especially for offshore pile and soil conditions.

Closure

These accounts of some of the ongoing research were intended to acquaint you with this important area of geotechnical engineering practice. Interestingly, current geotechnical engineering research and ice engineering research have a common practice link: both are receiving substantial support from the world's oil companies in connection with resource development in arctic and other offshore locations.

OTHER SKIN FRICTION ESTIMATE METHODS

This section presents two other methods for estimating skin friction. In general, I think the methods discussed so far are adequate and perhaps more correct for small-craft harbor pilings.

Tomlinson (1957, 1970)

Tomlinson (1957) analyzed loading test data on 56 piles driven into clay soils. The results of the analysis showed that the ratio of observed adhesion to undisturbed cohesion of the clay (i.e., α, the empirical adhesion coefficient from Equation 12.3) falls with increasing stiffness of the clay from about 1.25 in very soft clays to about 0.25 in very stiff clays.

The loss of adhesion is not related to loss of strength by remolding, but it is believed to be due to the presence of a partial gap between the pile and the soil. This gap may be formed by transverse vibrations during driving and by movement of the displaced soil upwards and away from the pile. Whereas the heaved-up soil will reconsolidate and close up the gap in soft clays--thus giving 100 percent or more adhesion--firm and stiff clays will only partially reconsolidate and a soft clay slurry can form around part of the pile shaft. However, in soft clays, the reconsolidation gives an ultimate adhesion greater than the original undrained cohesion due to consolidation strength gain.

The data from the 56 pile tests analyzed by Tomlinson show a lot of scatter. The theoretical adhesion was calculated (not measured) by deducting an assumed end bearing load (the product of the base area and 9 c_u; Eq. 5.5c) from the

ultimate load from the test. It also appears that cohesive strengths were estimated (not measured) in most cases.

Only one test was a tension test, and it's computed α equalled 0.98 for a stiff clay, which is 100 percent larger than Tomlinson's empirical α relationship. (The pile was 10 3/4-inch pipe driven into 20 feet of stiff clay with an undrained strength of 1,800 pounds per square foot, and the tension load was 119 kips 16 days after driving.)

I calculated α coefficients for 19 of the 56 tests. The 19 piles were round and square timber, square precast concrete, and steel pipe. The pile sizes ranged from 6.125 inches in diameter to 14 inches square. Pile embedment lengths ranged from 13 to 45 feet. Average coefficients (α) for undrained strengths (pounds per square foot) were, respectively, 0.9 at 250, 0.8 at 500, 0.6 at 1,000, 0.5 at 1,500, and 0.4 at 2,000.

Tomlinson (1970) stated that when piles are driven into stiff to very stiff cohesive deposits, that soil is carried down from the upper layers in the form of a skin which separates that surface of the pile from the surrounding undisturbed soil. Tomlinson studied nearly 100 load tests and made limited observations of this dragdown effect on bearing piles and sheet piles.

For sands or sandy gravels overlying stiff to very stiff cohesive soils, Tomlinson recommended using an α factor equal to 1.25. If the penetration ratio (defined as the depth of penetration into the stiff deposit divided by the pile diameter) exceeds 20, and if the undrained strength exceeds 1,500 pounds per square foot, then α should be decreased at a uniform rate, reaching a lowered value of 0.75 at 3,000 pounds per square foot.

This recommendation was based on load tests for about 30 piles reported at 10 sites. The scatter in α is large (0.40 to 1.91), and the clays involved ranged in strength from 1,500 to 3,200 pounds per square foot. In addition, Tomlinson inspected one 6 5/8-inch steel pipe pile. It was seen to have a sand dragdown of 1 foot 9 inches into a 3,000-pound-per-square-foot clay.

This is the only reported direct observation by Tomlinson, and though other researchers since have referenced Tomlinson (1970) and this recommendation, they do not endorse it. Neither do I. The basis for it seems inadequate, and it would be conservative to not count on sand dragdown.

Soft clays or silts overlying stiff to very stiff cohesive soils are another story, however. Here Tomlinson recommended $\alpha = 0.40$ for penetration ratios between 8 and 20, and $\alpha = 0.70$ for penetration ratios greater than 20. An α of only 0.10 was noted where a penetration was only 3 diameters. Tomlinson observed soft clay dragdowns to depths of 15 feet on timber piles. It also has been observed on troughs of steel sheet piles.

It seems appropriate to be careful about selecting an α where the pile could be "greasy" and have only shallow penetration.

Finally, Tomlinson considered just plain stiff to very stiff cohesive soils without overlying strata. The recommendation here was to use $\alpha = 0.40$ for penetration ratios between 8 and 20, and when penetration ratios exceed 20, α should range uniformly between a value of 1.0 at 2,000 pounds per square foot and a value of 0.5 at 3,000 pounds per square foot.

Tomlinson concluded that the skin friction effects on H-piles and cast-in-place piles are uncertain. None of the approximately 100 tests studies involved these types of piles.

Vijayvergiya and Focht (1972)

Vijayvergiya and Focht advocated a new way to predict pile capacities in clay. It is referred to as the λ method and assumes that unit skin friction is specifically a function of effective vertical stress and the undrained soil shear strength. The functional relationship is expressed by Equation 12.12:

$$f_s = \lambda \ (\sigma_v' + 2c_u) \qquad (12.12)$$

where: f_s = the average ultimate unit skin friction
σ_v' = the average effective overburden along the shaft
c_u = the average undrained shear strength along the shaft
λ = the dimensionless frictional capacity coefficient

Vijavergiya and Focht determined λ values for 43 previously reported load tests on steel pipe piles in clay and 4 additional tests. These tests included piles 8 to 333 feet long with capacities of 6 to 1,760 kips. The frictional capacity coefficient was found to have a close correlation with embedded pile length.

The values of λ obtained from their curve, fitted to the data analyzed, and the corresponding depths of pile penetration are: 0.5 at surface, 0.4 at 7 feet, 0.3 at 20 feet, 0.2 at 50 feet, 0.15 at 100 feet, and 0.12 at 200 feet.

Focht in Kraft et al. (1981) stated that the λ method was not intended for use with short piles in normally consolidated clays nor for use with penetrations of less than 50 feet, as it overpredicts capacity. Accordingly, the λ method seems inappropriate for Great Lakes small-craft marina design.

UPLIFT LOAD TESTS ON PILES

This section presents the results of several reported load test programs involving uplift pulls. I have included them to give you an idea of what actual uplift values have been measured.

Ireland (1957)

Ireland reports the results of pulling tests of cast-in-place Raymond step-taper piles installed in very uniform fine sands with the water table near the surface. N values for the sand were in the range of 10 to 20, generally less than 20.

Using their results, I characterize their data to indicate that piles in this submerged sand had a resistance of 60 kips for a 15-foot penetration, or about 4 kips per foot. For the average embedded surface area of 46 square feet, or about 3 square feet per foot, I calculate an equivalent uniform diameter of 24 inches and a skin friction value about 1,300 pounds per square foot.

Sowa (1970)

Sowa investigated the pulling capacity of cylindrical concrete piles cast in situ in bored holes in sandy and cohesive soils. Sowa concluded that the pulling capacity of piles in cohesive soils can be approximately estimated, while the pulling capacity of piles in sandy soil is considerably more difficult to estimate.

Three piles were pulled with the following results: 60 kips (about 275 pounds per square foot) on a 21-inch diameter pile 40 feet in a 11- to 15-blow sand with the ground water level 4 feet below the surface; 70 kips (about 450 pounds per square foot) on a 15-inch diameter pile 40 feet in the same sand, and 140 kips (about 650 pounds per square foot) on a 21-inch diameter pile 40 feet in a cohesive deposit with a cohesion of 2,300 pounds per square foot and the water table 8 feet below the surface.

Friels (1977)

A 45-foot and a 55-foot pile were test-loaded to capacities of 50 tons and 65 tons, respectively, and the piles--HP 10 x 42 steel H-sections--were then pulled. The pullout loads were 42 tons and 57 tons, respectively. The computed skin frictions, based on a bounded area of 4 times 10 inches, were 400 pounds per square foot for the first 30 feet of penetration, which was into a 5- to 15-blow fine-grained ML silt, and 900 pounds per square foot into the underlying 10- to 50-blow medium to medium-dense SP-SM silty sand. The piles pulled about three-quarters of an inch at failure, and about a tenth of an inch at a load of 10 tons.

Wagner and Lukas (1980)

Two concrete-filled 10 3/4-inch pipe piles were pulled at a site in Milwaukee. The site was 10 to 20 feet of 5- to 10-blow miscellaneous fill underlain by 10- to 30-blow clayey organic silts (cohesions of 500 to 1,000 pounds per square foot). The water table was at 10 feet, and the pipe pile had a surface area of 2.8 square feet per foot. One pile penetrated 45 feet and the other 85 feet. The respective uplifts were 45 tons (700 pounds per square foot) and 90 tons (750 pounds per square foot).

O'Neill et al. (1982a) and O'Neill et al. (1982b)

The papers by O'Neill et al. describe patterns of measured load transfer in a full-size, instrumented pile group; subgroups within the main group, and in single control piles loaded in compression. The effect of uplift loading on load transfer is described, but the picture is confused because some of the compression loadings were taken to failure several times, which degraded uplift resistances.

Eleven 10 3/4-inch (0.365-inch wall) steel pipe piles were driven closed end to a penetration of 43 feet into a layered overconsolidated clay. Eight-inch diameter, 10-foot-deep pilot holes were drilled prior to driving the piles. The group of piles was a 3 x 3 array with 3-diameter nominal spacing, and two separate piles were installed away from the group.

The soil was saturated below 2.5 feet and was characterized as follows: 0 to 8.5 feet--very stiff CH (OCR of 11 at 6 feet); 8.5 to 26 feet--stiff to very stiff slickensided CH; 26 to 30 feet—medium stiff silty CL; and 30 to 47 feet--stiff to very stiff sandy CL with sand partings (OCR of 4 at 47 feet).

Pore-water pressure dissipation rates were rapid. Within about four days after driving, the groups and reference piles had apparently reached their maximum side shear capacities (as indicated by the case method dynamic analyses during retap). Pore-water pressure changes of 1 psi were measured during uplift tests on six piles.

About 45 kips of load was released from the tips of each of the piles as they were tensioned, which was apparently equal to the residual tip load present at the conclusion of the last compression tests. If the tip load released in the uplift tests plus the indicated tip load at peak side load transfer in the last compression tests is subtracted from the mean reference pile capacity, a side shear load of 110 kips is obtained in the compression mode in the final compression tests on the reference piles. This was approximately the capacity of the piles in uplift, leading to the observation that overall side shear at failure in individual piles at the site was essentially independent of the direction of loading.

Peak unit side shear was seen to be uniformly low in the upper zone where predrilling was employed. (This suggests that driving in pilot holes will not help develop uplift resistance.)

Muschell (1982)

Muschell pulled a 12-inch (0.203-inch wall) pipe driven 32 feet into 1- to 10-blow materials adjacent to a marina. The pulling load was 55 kips (an average of 550 pounds per square foot). The soil was saturated below 4.5 feet and was characterized as follows: 0 to 8 feet--sand and gravel fill, soft black peat and soft silty marl with N values of 0 to 2; 8 to 12 feet--medium gray coarse sand with N values of 5 to 7; 12 to 17 feet--stiff brown clay with N values of 6 to 10 and q_u = 3,600 pounds per square foot; 17 to 23 feet--firm brown clay with N values of 6 and q_u = 2,100 pounds per square foot; 23 to 27 feet--very loose brown silty fine sand with N values of 1 to 2; and 27 to 32 feet--compact medium to coarse clayey gravel (no N values reported).

RELATIVE DEPTH PARAMETER FOR PILES IN SAND

Coyle and Castello (1981) summarized the results of a study made to improve state-of-the-art design of piles driven into cohesionless soils. Data from full-scale load tests reported in the literature were used to evaluate bearing capacity factors.

Coyle and Castello concluded that the best design correlations for piles in sands are the relative depth (depth-to-diameter ratio) and the sand friction angle. For unit side resistance (f_s), constant values were not indicated at penetrations of 30 diameters. The unit resistance showed a pronounced increase with an increase in relative depth for shallow penetration. However, the rate of increase in average unit resistance with relative depth becomes smaller and relatively constant at greater penetration (roughly between 10 and 30 pile diameters).

All discussion of this paper took issue with it on the basis that there is no need to assume "critical" depths if the possible variations of ϕ with overburden pressure are properly taken into account.

In discussing this state-of-the-art design paper, Meyerhof (1982) stated that Coyle and Castello's design correlations for piles driven into sand of a given friction angle indicate that both unit point resistance and unit skin friction increase considerably with relative penetration depth even much greater than the critical depth ratio. This conclusion contradicts the results of a detailed state-of-the-art review of the behavior of pile foundations by the writer (Meyerhof 1976) to which the authors did not refer in their paper. Meyerhof wrote:

> "The writer's analysis confirmed previous studies of the ultimate load of similar piles driven at a given site to various depths of penetration in fairly uniform sand that conventional bearing capacity theory for point resistance and skin friction is limited to short piles of less than about 15 to 20 pile diameters, and not for longer piles (as analyzed by the authors) for which the unit point resistance and unit skin friction remain roughly constant with depth at limiting values, which depend mainly on the geometry and roughness of the pile, the method of pile installation, the friction angle, compressibility and stress history of the soil, the ground water conditions and other factors. [A 113-word sentence!] Accordingly, preliminary estimates of pile capacity can, at present, only be made from the results of a site investigation using static or standard penetration tests for which semi-empirical methods were reviewed in the previously mentioned paper by the writer."

BEARING CAPACITY OF PILES IN LAYERED SOILS

Meyerhof and Valsangkar (1977) extended previous bearing capacity theory and semi-empirical methods of estimating ultimate pile loads in uniform soils to layered soils. Their analyses are compared with the results of model and field tests of piles in nonuniform soils of two and three layers.

For a weak layer overlying a firm stratum--for example, a soft remolded clay of low plasticity or a loose well-graded sand overlying a dense sand stratum--the skin friction (f_s) is given by Equation 12.1.

The maximum value of f_s increases with the strength of the upper layer for shallow depths of pile embedment in the bearing stratum. With increasing embedment depth, the maximum f_s and average earth pressure coefficient (K_s) approach the corresponding values of a very thick bearing stratum (deep appears to be 4 or 5 diameters into the lower layer).

For a firm layer overlying a weak stratum--for example, a dense sand over a compact sand, a dense sand over a loose sand and loose sand over clay--the f_s in the upper layer is practically unaffected for shallow pile penetration in the lower layer, and the maximum f_s and the average K_s approach those of the weak layer with increasing depth of embedment in the weak layer.

For a thin firm layer between weak strata--for example, a thin compact or dense sand layer between loose sand layers--the f_s of the piles in the thin dense sand layer were smaller than the values for a very thick bearing layer.

CYCLIC RESPONSE OF PILES

Poulos (1981) and Swane and Poulos (1982) discuss pile wiggling. These reports are referenced for general information and are not directly applicable to our analysis. Though exactly what "wiggle forces" might be eludes quantitative description, I believe that thermal thrusts and up-and-down action, among other factors, may be involved when a pile is extracted from the bottom. The action may be something akin to pulling and wiggling a nail out of a piece of wood.

Poulos (1981) described a series of model tests on piles in remolded clay which indicated that considerable loss of skin friction could occur if piles were subjected to a two-way cyclic loading (i.e., loading alternating between tension and compression with a zero or small mean value). Because of the possibility of two-way cyclic loading on pile foundations supporting offshore structures, it is deemed essential to develop a means for predicting the circumstances under which cyclic loading will affect pile performance.

Recent tests suggest that cyclic degradation of skin friction arises primarily from destruction of interparticle bonds and particle alignment, rather than the development of pore pressures.

Small-scale pile cycle tests of 1,000 cycles at a frequency interval of 2 1/2 seconds were conducted on 3/4-inch piles in remolded clay for skin friction only. Poulos concluded that the ultimate load capacity and cyclic stiffness decrease with an increasing number of cycles and increasing cycle load level.

The effect of the number of cycles becomes more significant when the cyclic load (half the peak-to-peak load) approaches 50 percent of the static ultimate load. Cyclic degradation begins at the top of the pile and progresses downward as the half peak-to-peak load and as the number of cycles increases, resulting in a gradual transfer of load to lower parts of the pile. (Cyclic action may have something to do with piles that come up after several years of satisfactory service--but probably not, as the piles "rest" all summer.)

Swane and Poulos' (1982) paper describes a method for examining the theoretical response of single vertical piles to cyclic lateral load. The soil-pile interaction model is bilinear-elastoplastic and based on subgrade reaction theory. The model allows the effects of soil yielding and pile-soil separation to be considered for any arbitrary loading program.

Material degradation is not considered in the analysis, and the material properties are assumed to remain constant during the cyclic loading. Attention is foused instead on the mechanical degradation of the soil-pile system caused by plastic deformation of the soil.

APPROXIMATE SOIL UPLIFT RESISTANCES

Table 12.1 lists approximate soil skin-friction uplift resistances, ranging from 10 to 125 kips for 6- to 16-inch pilings penetrating uniform harbor bottom deposits for 5 to 50 feet. This table is intended as a guide in determining the penetration depths necessary to achieve uplift resistances desired. The table is based on Meyerhof (1976).

In developing the table, I made use of Equations 12.1, 12.2, 12.5 and 12.6 and Tables 5.3, 5.4, 5.6, 5.9 and 10.2. I assumed circular shapes for computation; other shapes can be stated in approximate circular size through perimeter calculations. For example, a 10-inch pipe has a perimeter of 31.4 inches; an 8-inch square has a perimeter of 32 inches. For H-piles, I recommend use of a core-bounded perimeter equal to 4 times the size of the H-pile.

Table 12.1 is a guide and can be used directly to estimate pile lengths if you can characterize your deposit. Otherwise, and in the more usual cases, you will need to calculte an estimated length using the information presented in this chapter. This calculation will also result in only an estimated length—to determine the actual length for a given pullout resistance, a tension load test is required.

LATERAL CAPACITY OF FREE-HEAD PILES (AFTER BROMS 1972)

This discussion of the design of piles for lateral loads is taken from Broms' (1972) state-of-the-art paper. This paper incorporates some of the methods contained in earlier papers (Broms 1964a, 1964b, 1965). Summaries of Broms' methods can be found in Poulos and Davis (1980) and Winterkorn and Fang (1975).

I am confining this discussion to free-head piles. Piles developing moment resistance through connection with other piles and superstructure framing must be analyzed by other methods not discussed here. (These other methods are contained in the references just cited). Also, I don't calculate pile deflections for free-head (mooring) piles, because as long as the pile does not fail by breaking or yielding, and the soil does not fail in passive resistance, the functional concern here is the length of embedment and strength of the pile required, not deflection. Deflection analysis methods are briefly reviewed, however.

Behavior of Laterally Loaded Piles

The resistance of laterally loaded piles has been intensively studied in the past. Despite the considerable advances made, the detailed behavior and the mechanism of failure of soil around laterally loaded piles aren't completely understood, nor are the methods for calculating the lateral resistance at working loads or at failure entirely satisfactory.

The behavior of laterally loaded piles has been investigated by theoretical studies based primarily on the theory of elasticity, by model or full-scale tests under controlled conditions, and by observations of actual structures.

TABLE 12.1: Approximate Soil Skin Frictional Resistance for Uplift (kips) (after Meyerhof 1976)

Depth of Penetration (feet)	Sands			Clays		
	Loose to Medium	Medium to Dense	Dense to Very Dense	Soft to Medium	Medium to Stiff	Stiff to Very Stiff
	Nominal 6-inch Round (4 3/4-inch Square) Piling					
10	-	-	13	-	10	22
15	-	8	19	-	22	48
20	-	11	26	10	38	64
25	-	14	32	15	60	80
30	-	17	38	22	72	96
35	-	20	45	29	84	112
40	10	22	51	38	96	128
45	11	25	58	49	108	-
50	12	28	64	60	120	-
	Nominal 8-inch Round (6 1/4-inch Square) Piling					
10	-	-	17	-	13	29
15	-	15	33	-	28	63
20	-	20	45	13	50	84
25	10	24	56	20	79	105
30	13	29	67	28	95	126
35	15	34	78	39	110	-
40	17	39	89	50	126	-
45	19	44	100	64	-	-
50	21	49	111	79	-	-
	Nominal 10-inch Round (8-inch Square) Piling					
10	-	10	21	-	16	36
15	-	20	47	-	35	78
20	13	30	69	16	62	104
25	16	38	89	24	97	130
30	20	46	104	35	117	-
35	23	53	122	48	136	-
40	26	61	139	62	-	-
45	29	68	-	79	-	-
50	33	76	-	98	-	-

TABLE 12.1 (continued)

Depth of Penetration (feet)	Sands			Clays		
	Loose to Medium	Medium to Dense	Dense to Very Dense	Soft to Medium	Medium to Stiff	Stiff to Very Stiff
	Nominal 12-inch Round (9 1/2-inch Square) Piling					
5	-	-	-	-	-	11
10	-	11	25	-	19	43
15	11	24	56	-	42	93
20	19	43	99	18	74	124
25	23	54	124	29	116	155
30	28	65	149	42	139	-
35	33	76	-	57	-	-
40	37	87	-	74	-	-
45	42	98	-	94	-	-
50	47	109	-	116	-	-
	Nominal 14-inch Round (11-inch Square) Piling					
5	-	-	-	-	-	13
10	-	13	29	-	22	52
15	13	29	67	-	50	111
20	22	52	118	22	89	148
25	32	76	148	35	139	-
30	39	91	-	50	-	-
35	45	106	-	68	-	-
40	52	121	-	89	-	-
45	58	136	-	112	-	-
50	65	-	-	139	-	-
	Nominal 16-inch Round (12 1/2-inch Square) Piling					
5	-	-	10	-	-	15
10	-	15	33	-	25	59
15	14	33	75	10	56	126
20	25	59	134	25	100	-
25	39	92	-	39	157	-
30	50	117	-	57	-	-
35	59	137	-	77	-	-
40	67	-	-	100	-	-
45	76	-	-	128	-	-
50	84	-	-	-	-	-

Laterally loaded piles are usually classified as rigid, flexible or semirigid. A marina mooring pile probably behaves in a semirigid manner because of its embedment length relative to its size. A semirigid pile may rotate in the soil when loaded, and flexural deformations may contribute appreciably to the lateral deflections.

The behavior of such piles is affected by a change of the pile length. If the pile is unrestrained or free-headed, the bending moment in the pile at the ground surface is positive and acts in the same direction as the applied lateral load. The maximum moment will occur at some depth into the harbor bottom.

Deflections of Laterally Loaded Piles

The deflections of a laterally loaded pile increase approximately linearly with increasing applied load until the applied lateral load exceeds about a half to a third the ultimate lateral resistance of the pile. Above this load level, the lateral deflections increase faster than the load.

In cohesive soils, the soil separates from the pile at the back face as the loaded pile rotates in the soil. In cohesionless soils below the water table, the crack at the back face gradually fills up with loose soil as the pile rotates.

For piles in cohesionless soils, the larger part of the lateral deflection takes place at the time of loading. However, a substantial increase of the lateral deflection may be caused by repetitive (cyclic) loads or by vibrations, especially when the relative density of the surrounding soil is low.

The lateral deflection of piles in cohesive soils can increase appreciably over time due to consolidation and creep. At loads less than a half to a third of the ultimate, lateral deflection of the pile is generally calculated from the theory of elasticity (Mindlin's equation) or from a coefficient of subgrade reaction.

Under the theory of elasticity method, the soil is replaced by an ideal elastic and isotropic material with a constant modulus of elasticity and a constant Poisson's ratio. It has not been possible to solve the case when the modulus of elasticity increases with depth, which is often the case for sand, or when the modulus of elasticity at unloading is higher than at loading.

The coefficient of subgrade reaction method assumes that the lateral earth pressure on a pile increases linearly with increasing lateral deflection (the Winkler foundaton model). This coefficient relates the load to the deflection. The coefficient is not a material constant--it varies with the dimensions of the pile, the intensity of the applied lateral load and with the depth below the ground surface. The concept of a coefficient of subgrade reaction does not take into account the continuity of the soil mass.

Lateral Capacity of Free-Head Piles

The moment distribution for a laterally loaded pile is dependent on the restraint provided (e.g., a pile cap) and on the earth pressure distribution

along the pile. As the applied lateral load on the pile increases, the lateral resistance of the soil strata located progressively further down the pile is mobilized. The location of the maximum bending moment moves down the pile with increasing applied load.

The failure of a laterally loaded pile occurs either when the pile rotates as a unit in the soil (soil failure), or when the ultimate moment resistance of the pile section has been exceeded. The lateral resistance at soil failure is governed by the shear strength of the soil around the pile, while at pile failure it is also affected by the ultimate moment resistance of the pile section.

The maximum lateral earth pressure at the ground surface corresponds approximately to passive earth pressure for an infinitely long wall. The shape of the pile section has only a negligible effect on the ultimate lateral resistance and the lateral earth pressure.

Several authors have assumed that, for piles in cohesionless soils, the ultimate lateral earth pressure is related to passive earth pressure. One such author, Brinch Hansen (1961) has presented a method to calculate the lateral resistance of piles.

Brinch Hansen assumed that the ultimate lateral resistance at the ground surface corresponds to passive earth pressure on an infinitely long wall, that the increase of the lateral earth pressure at moderate depths corresponds to the forces acting along the two vertical triangular sides of a wedge with a width equal to the width of diameter of the pile, and that at large depths the soil at failure moves laterally around the pile. (This wedge constitutes a general bearing capacity shear failure like that shown in Figure 5.1a.)

Broms (1964b, 1965) concluded from an analysis of available test data that the ultimate lateral resistance for cohesionless soils can conservatively be calculated by assuming that the ultimate lateral resistance is equal to 3 times the passive earth pressure. Similarly, Broms (1964a, 1965) conservatively estimated that the ultimate lateral resistance for cohesive soils is equal to 9 times the undrained cohesive strength (c_u).

Broms neglected the resistance of the soil for the first 1.5 pile diameters down into the clay deposit. In this zone the clay bulges upward when loaded to failure.

Figure 12.1 shows Broms' analyses, and Table 12.2 was prepared with methods recommended by Broms and information from Tables 5.3, 5.4, 5.5 and 10.2.

Figure 12.2 shows free-standing mooring piles permanently deflected by ice in a confined harbor basin. The ice is generally 2 feet thick, and the water is 12 feet deep. The pilings penetrate about 20 feet into a very stiff to stiff brown gray clay that is somewhat silty and sandy with gravel and has a 15- to 20-blow count. The pipes are 10 3/4 inches and filled with pea gravel concrete. The wall thickness is unknown but assumed to be 0.188 inches, 0.219 inches, or 0.250 inches. (The corresponding section moduli and ultimate moment capacity, assuming Grade 36 steel, would be 16.4 inches cubed and 49 kip-feet, 18.8 inches cubed and 56 kip-feet, and 21.2 inches cubed and 64 kip-feet. Obviously, the soil has yielded, the pipes have yielded, or both. The question is, what might have been the ice load?

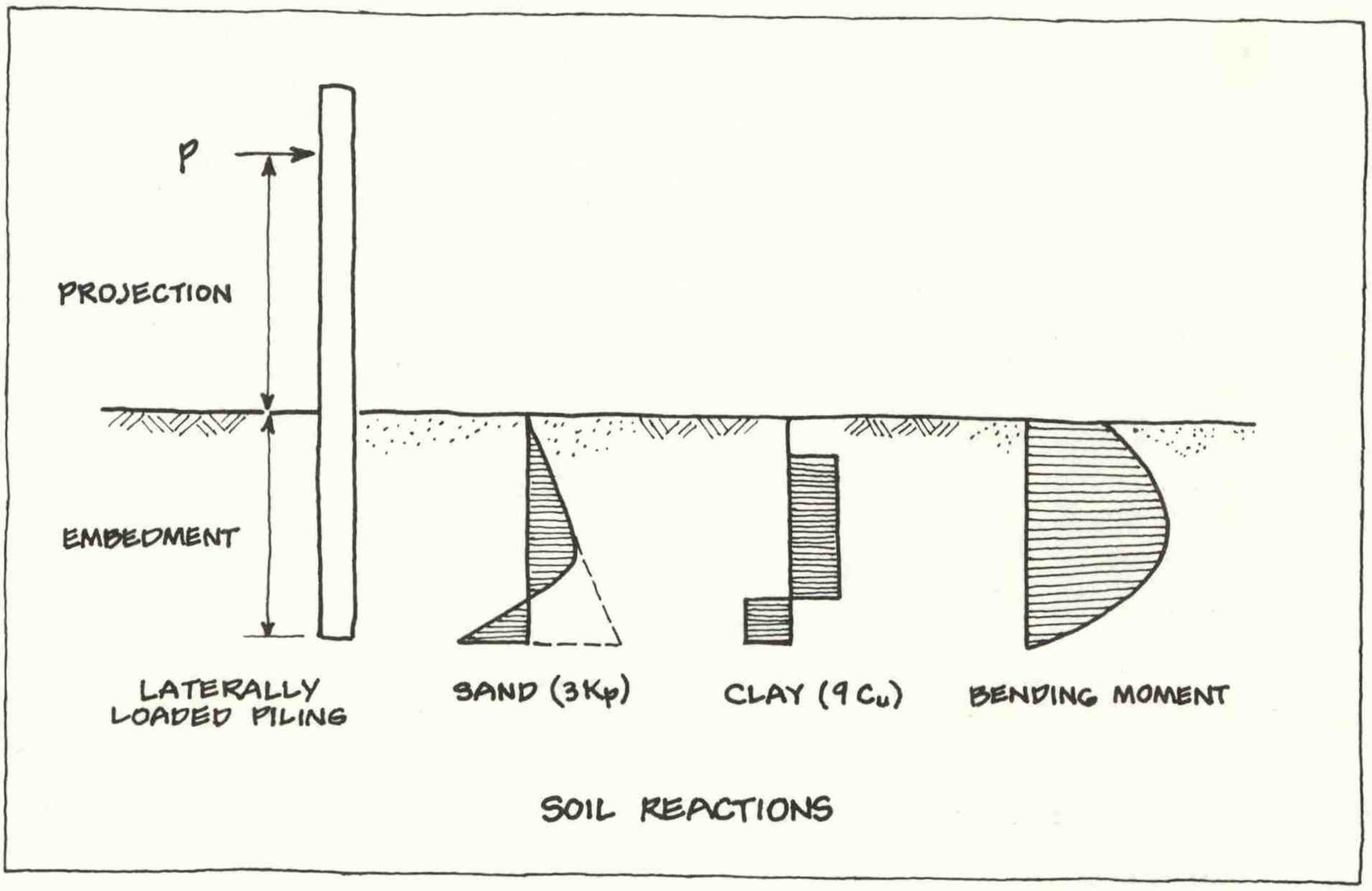


FIGURE 12.1: Laterally Loaded Piling, Soil Reactions and Bending Moments (after Broms)

For a 12-inch piling with a 12-foot projection, Table 12.2 shows that, with a bottom penetration of only 4 feet, a 2,000-pound ice force could possibly fail this soil, if the pile could stand 28 kip-feet of moment (i.e., the pile would not yield before the soil yielded). At 3,000 pounds force, the required penetration would be 5 feet and the corresponding moment 42 kip-feet. Any of the assumed wall thicknesses of 10 3/4-inch pipe can handle 42 kip-feet.

If the ice force were 4,000 pounds, the required penetration is still considerably less than the 20 feet of penetration in this case. So, if the 10 3/4-inch pipe can handle 57 kip-feet of moment, the load could go as high as 4,000 pounds. Theoretically, only the pipe with the thinnest assumed wall thickness (0.188 inches and 49 kip-feet) would be inadequate. Since these pipes are filled with concrete, they will have some indeterminate bending strength beyond their unfilled condition. Therefore, I estimate that the ice force might have been in the 5-kip range and that the pile and soil materials may have yielded.

From an ice engineering standpoint, it has been suggested that thin piers like pilings in thick ice produce larger pressures than wide piers. Presumably, this is because of some triaxial effects. This is analogous to the very high unit pressures in the ice caused by pushing a knife against it. Reducing widths of piers does not reduce ice forces proportionately because of this aspect ratio effect.

TABLE 12.2: Approximate Minimum Required Embedment Length (feet) and Average Bending Moment (kip-feet) for Free-Headed Pile Projecting above Harbor Bottom with Lateral Load (after Broms)

Piling Size Projection Lateral Load	Embedment Length Range[a]		Average Bending Moment[b]
	Sand	Clay	Sand and Clay
8-inch Round or Square Piling			
6-foot Projection			
1,000 pounds	5-6	3-4	8
2,000 pounds	6-8	3-4	16
3,000 pounds	7-9	4-6	25
10-foot Projection			
1,000 pounds	5-7	3-4	12
2,000 pounds	6-8	4-6	24
3,000 pounds	7-10	5-7	37
14-foot Projection			
1,000 pounds	6-7	3-4	16
2,000 pounds	7-9	5-7	32
3,000 pounds	8-10	5-8	48
12-inch Round or Square Piling			
8-foot Projection			
2,000 pounds	5-7	4-5	20
3,000 pounds	6-8	4-6	30
4,000 pounds	7-9	5-6	41
12-foot Projection			
2,000 pounds	5-7	4-6	28
3,000 pounds	6-8	5-7	42
4,000 pounds	7-10	5-8	57
16-foot Projection			
2,000 pounds	6-8	4-6	36
3,000 pounds	7-9	5-7	54
4,000 pounds	8-10	6-9	73

TABLE 12.2 (continued)

Piling Size Projection Lateral Load	Embedment Length Range[a]		Average Bending Moment[b]
	Sand	Clay	Sand and Clay
16-inch Round or Square Piling			
10-foot Projection			
3,000 pounds	6-7	5-6	36
4,000 pounds	7-9	5-7	48
5,000 pounds	7-9	5-8	62
14-foot Projection			
3,000 pounds	6-8	5-7	49
4,000 pounds	7-9	5-8	65
5,000 pounds	8-9	6-9	82
18-foot Projection			
3,000 pounds	7-9	5-8	60
4,000 pounds	7-9	6-8	80
5,000 pounds	8-10	6-9	102

a Lower value of range is for denser or stiffer materials; higher value is for looser or softer materials. Add one to two feet to minimum values shown for disturbed bottom conditions.

b Bending moments for sands and clays and for load projections vary less than 10 percent.

Muschell (1981) stated on the basis of personal experience that it appears the action of lateral thermal ice thrusts on individual piling is quite different than thrusts on solid or gravity types of crib structures and that these forces may be far greater under certain conditions than expected. So, as I noted before, our knowledge of lateral forces on pilings, both ice-wise and soil-wise, is incomplete and imperfect.

Broms (1972) noted that the rotational capacity of steel pipe piles--with the exception of thin-walled pipe piles and cast-in-place unreinforced concrete piles--is probably sufficient to cause a full moment redistribution (as yielding occurs) and to develop passive earth pressure in the soil before complete collapse.

However, local buckling of the pile walls may occur in thin-walled pipe piles before full moment redistribution has taken place. Such piles should be filled



FIGURE 12.2: Free-Standing Mooring Piles Permanently Deflected by Ice in a Confined Harbor Basin

with concrete to prevent local buckling. The rotational capacities of timber piles is limited, however, because of the relatively low ductility of wood in tension. (Nonetheless, wood piles will rotate without breaking. In one new marina, timber spring piles drove easily for 8-10 feet and then seized up during the last 3-5 feet of driving. They are leaning now; it is apparent that the penetration resistance is inadequate.)

Methods to Increase the Lateral Resistance of Piles

If the pile itself is strong enough, it may be worthwhile to increase the pile's lateral soil resistance. One way to do this in sand is by driving timber piles between laterally loaded piles. Reportedly, the coefficient of subgrade reaction can be increased two to three times by this method. An increase of the relative density by approximately 10 percent is required to increase the coefficient of subgrade reaction by 100 to 200 percent. The lateral resistance of granular soils probably also can be increased by densification by vibroflotation.

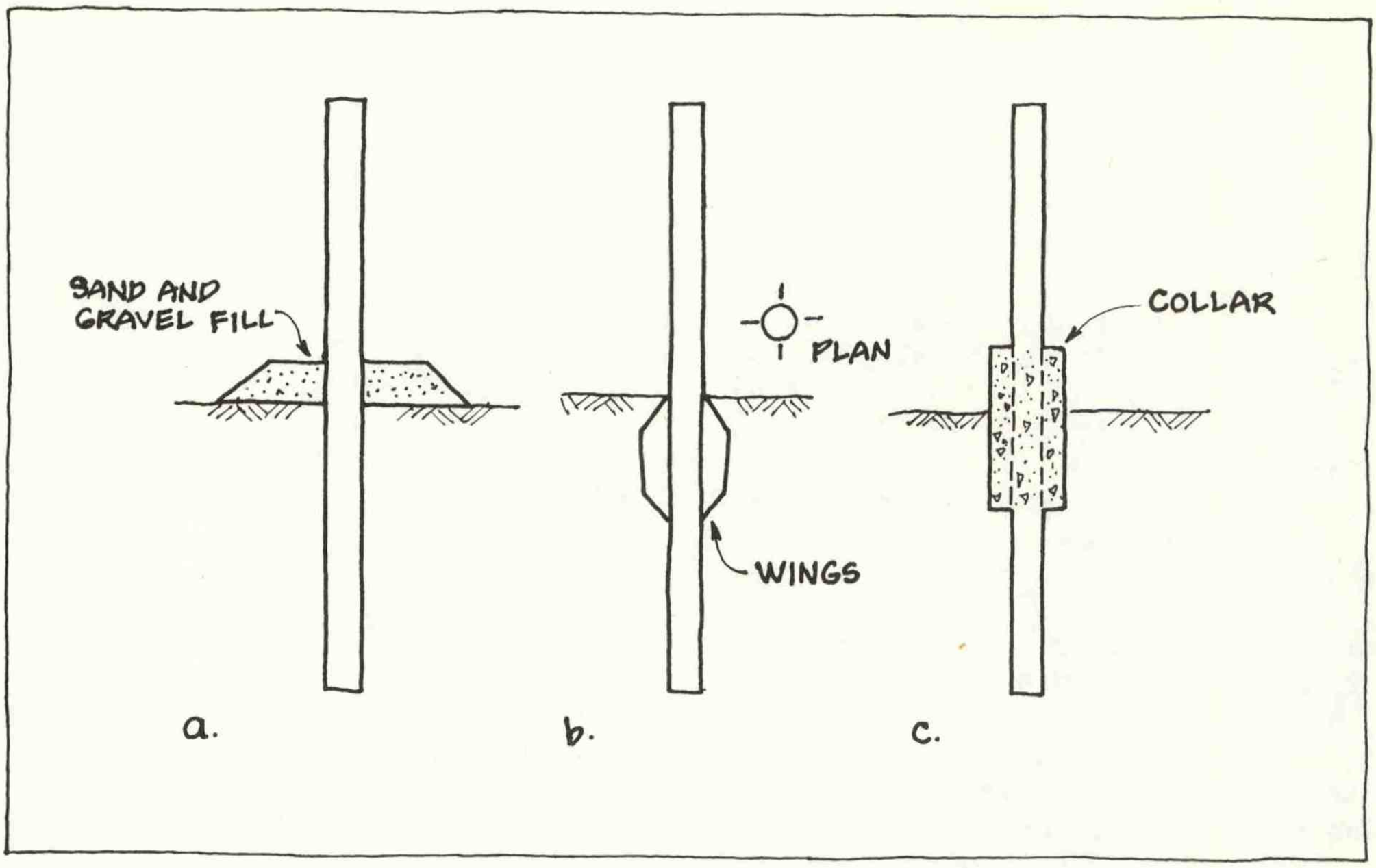


FIGURE 12.3: Methods Used to Increase the Lateral Resistance of Piles (after Broms 1972)

Figure 12.3 illustrates other methods for increasing the lateral resistance of piles. Sand and gravel fill (Figure 12.3a) is effective for very soft clays when the piles are subjected to cyclic loads. The granular fill gradually works itself down into the clay and increases the effective diameter of the piles. The height of the fill around the piles is limited by the bearing capacity of the underlying soil. This method seems appropriate for small-craft harbors, where ice and boat mooring forces could help work the gravel into the clay.

The lateral resistance can be effectively increased also by providing piles with wings just below the ground surface, as illustrated in Figure 12.3b. The wings are welded directly on the steel piles or attached to a loose collar, which is pushed or driven into the soil before placement of the pile. The space between the collar and the piles is often filled with mortar or sand.

Other methods can also be used to increase the dimension or stiffness, or both, of the pile near the ground line (e.g., a concrete collar, as shown in Figure 12.3c).

Davisson and Gill (1963) analytically investigated the engineering behavior of a laterally loaded pile in a two-layered soil system. Using a modulus of subgrade reaction to define soil stiffness, they found that the surface layer of soil has a strong influence on the behavior of the soil-pile system, even if it is only a few diameters thick.

LATERAL LOAD TESTS ON PILES

An oft-cited report on thrusts on piles is McNulty (1956). McNulty performed lateral load tests on concrete cast-in-place piles, heads not fixed, embedded in a medium-dense silty sand. In a second project, McNulty employed fixed-end timber piles embedded in a medium sandy clay.

Sixteen-inch concrete piles--embedded 15 to 30 feet into fine silty sand (blow count about 10) below the water table, with heads free to rotate--developed an ultimate thrust resistance of about 25 kips at a lateral deflection of about 1 inch. The yield point was estimated at 15 kips.

Twelve-inch timber piles--embedded 40 to 50 feet into medium clay (blow count about 10 or less) below the water table, with heads fixed against rotation--developed an ultimate thrust resistance of about 20 kips at 1 inch deflection with no indication of yield.

Both piles could take 10 kips or more, with a 0.25-inch deflection, and then rebound when unloaded.

Muschell (1982) reported on an investigation of pile-soil interaction for lateral loads and their displacement and vertical soil uplift resistance. (The soil profile for this site, which is near a marina, was discussed in the "Uplift Load Tests on Pilings" section). Three 12-inch pipe piles were driven. Two of the piles were fastened together with bolted steel channels, and the third pile was left free of lateral restraint. A series of lateral and vertical test loads were then placed on the fixed and free piles.

At full test-load, the deflection at the load point for the unsupported pile was 2.6 times the deflection of the supported piles (roughly 6 inches vs. 2 inches). Theoretically, the ratio should have been 4 to 1. The lateral load on the unsupported pile was calculated with Broms' methods. At yield it was 11 kips for cohesionless soil and 13 kips for cohesive soil, and at ultimate it was 16 kips for cohesionless and 20 kips for cohesive. The ultimate load measured was 20 kips.

Muschell said that there was good agreement with the test data for free-headed piles analyzed with Broms' methods, but that for fixed-headed piles considerable differences existed. Nevertheless, Broms' methods are the best format for single piles available in today's methodology.

Part Four:
CONSTRUCTION



13. Considerations During Construction

PILE LOAD TESTS

I have emphasized that field load tests are required to ascertain the uplift resistance of pilings. They also are necessary to determine the lateral capacity, though this generally is not of the same level of importance.

A load test or series of tests can be carried out during the design phase of a project, or sound engineering estimates of pile types and capacities can be made then and confirmed with field tests during the construction phase.

"Production piles" can be used for tests. For example, one or more piles on a head pier could be pulled. Subsequently, these piles can remain part of the finished dock construction. They will have only a slightly reduced capacity because of the tensioning.

The lifting force may be developed by reaction against other head pier piles; in fact, the layout of head pier piles and framing could be designed to accommodate an extraction testing program.

Piles in granular soils may be load-tested several days after driving. In cohesive soils, wait at least a month for the soil deposit to thixotropically gain strength. In other words, the test piles should be driven early if driven into clay. And waiting only a month may not be long enough--it may be several months (though an estimate could be made of the "untested" subsequent strength gain if the waiting period were only a month).

ASTM D3689-78, "Standard Method for Testing Individual Piles Under Static Axial Tensile Load," is the current ASTM specification designation. D3689 covers procedures for testing a vertical or batter pile to determine the response of the pile to a static tensile load applied axially to the pile. The method includes apparatus for applying the loads, apparatus for measuring movements, loading procedures, procedures for measurement of pile movement, safety requirements and test report.

Poulos and Davis (1980) stated that the usual method of carrying out lateral load tests is to install a pair of piles and jack their heads apart. A horizontal beam is inserted between the piles, and the jack reacts against one of the pile heads and the beam the other. Lateral deflection of the pile is usually measured with dial gages, and strain gages are sometimes installed along the embedded portion of the pile to measure flexural stresses, whereby the bending moments may be obtained. With steel piles, inclinometers also may be installed within the pile to measure the variation of lateral deflection with depth along the pile.

Three common compression load tests exist. They are maintained loading tests, constant rate-of-penetration tests (CRP), and method of equilibrium tests. A considerable number of arbitrary or empirical rules are used or contained in codes to serve as criteria for determining the allowable working load from load tests.

PILE DRIVING

Bowles (1982) stated that when the required driving resistance is encountered, driving should be stopped. These driving resistances may be arbitrarily taken as 4 to 5 blows per inch on timber piles, 6 to 8 on concrete piles, and 12 to 15 on steel piles.

Pile driving is an art as well as a science, and both the art and the science of it exceed my knowledge: Experienced contractors and construction engineers should be consulted for help in selecting the pile, the hammer and the installation procedures for your specific set of circumstances.

Jetting Piles

It is common belief that a "jetted-in" pile has less skin resistance than a driven pile. In the jetting process, a pump capable of delivering about 500 gallons of water per minute at a pressure of 150 to 200 psi is used to penetrate dense soil layers to within a few feet of the desired depth. The last few feet are driven to achieve end bearing.

Concrete piles have been installed by "jetting" through a small internally cast PVC tube along the pile axis. Since the piling is protected by a deicing system, the effect of this type of jetting on skin friction is uncertain.

It is safer to drive, not jet, piles if they are to be uplift-resisting piles, though I think that in time the difference in skin resistance would diminish. In some deposits, jetting may be necessary just to get the pile into the ground, in which case deep-jetting would provide a penetration that gives more total uplift resistance than a shallower driven piling.

Driving Piles with Closed vs. Open Ends

Should pipe piles be driven open-ended or closed-ended? I think closed end would be the choice in most deposits. If they are driven open-ended, they must be cleaned out before they are filled with concrete; if they are driven closed-ended, this is not required. There is little difference in driving resistance because the end of open-ended pipes become plugged with soil. Similar plugs form between the flanges of H-piles during driving. So, if the driving resistance is about the same, you might as well drive close-ended.

In any case, the driven pile should be "lamped" (i.e., examined by lowering a light into it) to determine the soundness of the pile shell after driving.

Driving with Flat or Conical Plates

Should the closure be a flat plate or a conical point? The answers to this question don't agree. Driving points are often essential to penetrate cobbles and small boulders and fissured rock. But if these are not a problem, would a cone or a plate be better?

Bowles (1982) said that the driving resistance for pipe piles with flat points differed little from those with conical points. The reason is that a wedge-shaped zone of soil develops in front of the flat point, somewhat like a zone of failure beneath a footing (Figure 5.1).

However, a 60-degree cast steel conical point distributes the shock load from obstructions or founding strata around the full periphery of the pipe, whereas a flat plate may concentrate load on only a segment of the pipe, permitting pipe failure to start when an obstruction is struck. End closures on pipe piles can also be made with die-shaped structural steel tapered fittings.

Nottingham and Christopherson (1982) noted that the converse of hard driving is the case where loose, saturated alluvial soils are encountered. Here, the open-ended pipe of H-piles tend to "run" and drive much deeper than actually required. They have found that pipe piles with conical tips will "take up" within reasonable distances, as they tend to densify soils and create more friction. In time, these piles often exhibit remarkably increased load resistance.

Peck et al. (1974) stated that ordinarily pipes are closed at their lower ends, usually with plates. More elaborate closures, such as conical points, rarely display any significant advantages. In a few soils, such as stiff plastic clays, the overhang of the plate (beyond the pile diameter) should be eliminated.

The use of plates and perhaps of conical points seems justified in developing skin friction resistance in pipe piles. Seek the judgment and experience of people familiar with the soils and past pile performances of the site with which you are working. Similarly, driving timber piles "butt-down" may subsequently provide greater uplift soil resistance. Again, consult experienced people and weigh site performance evidence.

PILE DEGRADATION

Figure 11.6 showed what ice can do to a timber piling. What about concrete and steel? Bowles (1982) stated that freezing and thawing can damage concrete piles in any exposed situation, but he was addressing the effects of cold weather rather than ice sheets. My own observations are that concrete piles do alright. Concrete does not appear to wear any worse than steel. I have not observed any spalling and flaking. A properly designed concrete mixture, with due recognition of the cold environment, should prove satisfactory.

Some concrete dock elements have had chipping and spalling trouble, but this problem resulted from inadequate space for movement between the concrete elements and improper cushioning. Neoprene pads, joint fillers, etc., are required if the pilings supporting concrete dock elements are moved by ice thrusts.

Occasionally, piles have joined upper and lower sections made of dissimilar materials, such as concrete and timber. The cost and difficulty of forming a suitable joint--which is also true of splicing timber piles for tension--have led to virtual abandonment of this type of construction. It is inappropriate in small-craft harbors.

Steel piles can corrode. The corrosion deterioration is usually insignificant if the entire pile is buried in a natural soil formation, but it may be severe in some fills due to trapped oxygen. Harbor piles would not be affected so except that they are in a fluctuating, wet-dry environment; consequently, they

will rust and corrode. Steel in sea water needs protection (e.g., by encasing it in concrete). In fresh water, the protection (e.g., by painting) is mostly cosmetic. If ice acts directly on the piling, the paint will soon be worn away; if the pile is deiced, the coating will remain intact much longer.

The painting and repainting of pilings can be done in early spring on warm days when the ice is low and serves as a working platform--which brings up the last subject of this manual:

ICE AS A WORKING PLATFORM

Figure 3.3 showed ice serving as a construction platform for a lightweight pile driving rig. Large timber skids were used to better distribute the rig's weight on the ice. To move the rig, workers jacked the skids up to place the weight on a pair of dual truck tires, and the rig was then pushed with a small bulldozer tractor. (Later in the season, the tractor fell through, but it was easily pulled out of the rotting-ice quagmire).

Driving pilings from the ice instead of a barge has an additional benefit: the ice can serve as a template. By boring holes at planned locations in the ice plate, the piles were accurately positioned and maintained during driving.

In Situ Ice Strengths and Conditions

Gerard (1983a) stated that design for ice forces should assume that the ice is competent. For operational purposes, however, it is desirable to know the in situ ice strength in a particular place at a given time, either before or during melt.

Beyond cutting out a sample and making a qualitative visual assessment, there is little useful field testing that can be done because of the brittle nature of the ice and the excessive difficulty in getting repeatable results. Indeed, a visual estimate by a reasonably experienced ice engineer of the strength of an ice sample is likely to be more accurate than the results of any practical field test.

Gerard cautioned about a circumstance that is often approximated by field situations, and this is the case where a load is imposed at the edge of a semi-infinite ice sheet.

For example, this would be the limiting situation as a load moved over a "wet" crack or as a crane operated near a slot cut in a ice sheet for pier construction. As you might imagine, the maximum stress for a load at the edge of a semi-infinite ice sheet is approximately double that for the same load on an infinite ice sheet. However, the maximum deflection of the semi-infinite sheet is nearly four times larger. This is important because once the surface of the ice goes below the water level, water will move onto the ice sheet and significantly reduce the additional load bearing capacity.

Dredging from Ice

Just how close can you operate construction equipment near a crack or opening in the ice cover? Figure 13.1 shows a crane dredging from an ice platform. The ice was nominally 2 feet thick, and the crane was positioned about 40 feet

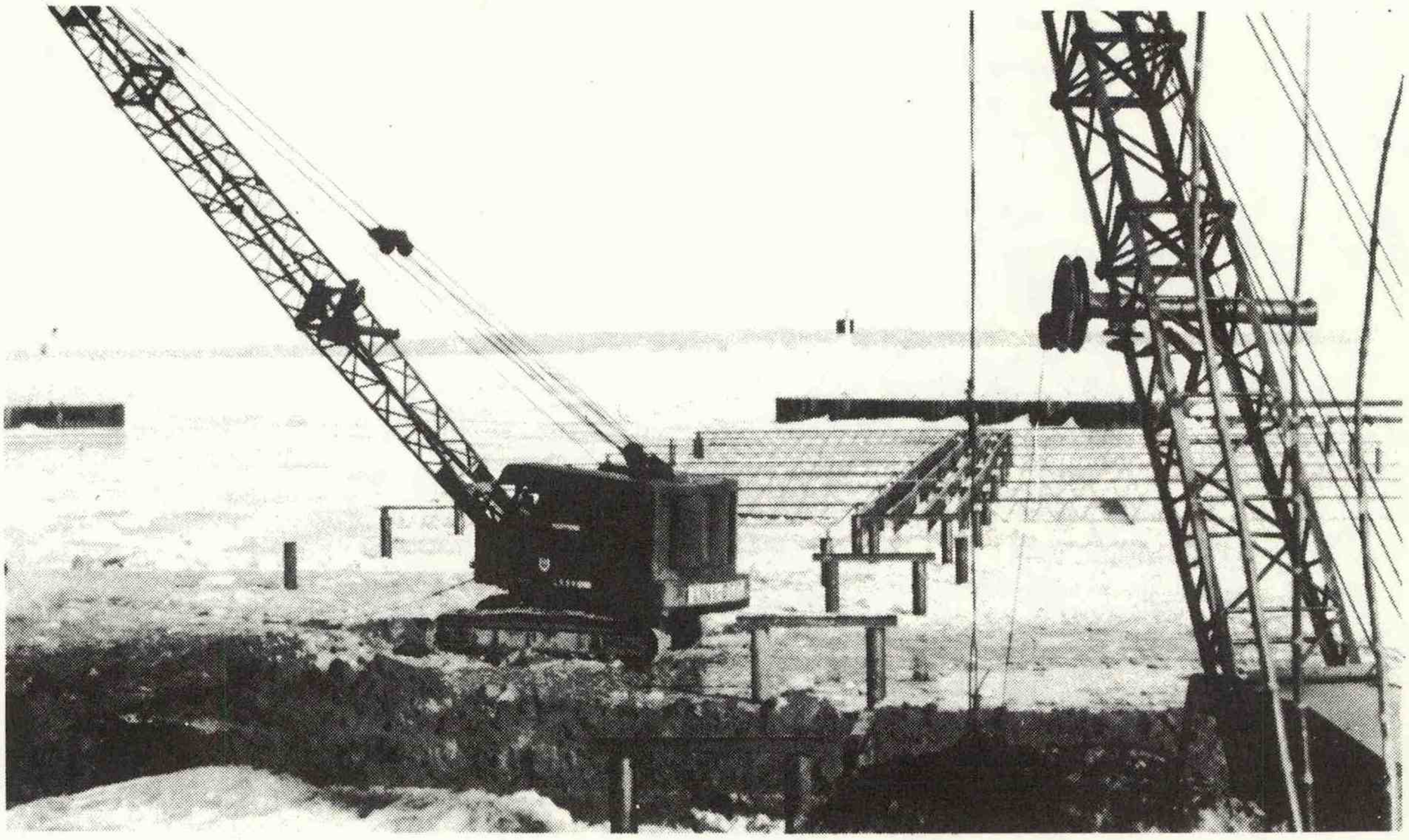


FIGURE 13.1: Crane Dredging from Ice Platform

from the hole broken in the cover. During this operation, the crane started offshore at the dredging limit and excavated (both ice and soil) its way back to the shore.

Another way to do winter dredging, if the ice can't be used and if the site is amenable, was described by Wortley (1972). Workmen drove temporary sheet piling to serve as a bulkhead between the lake and the harbor site to be dredged. Once in place, the site was sump-pumped and the dredging performed "in the dry" with conventional earthmoving equipment. Frozen and semifrozen excavated materials were easily handled while working a two-shift day. It is important to keep working and to not expose more areas than can be kept up with.

Furthermore, you don't have to accept the natural ice thickness if it is too thin to use as a work platform--you can intentionally thicken the ice at your site, provided you don't get a "warm" winter.

Ice Thickening and Reinforcing

Hoffman (1967) described surface flooding techniques for improving natural ice covers. The mean daily temperature should be 15°F or less, otherwise the freezing rates are slow and long waits are required between applications. The

depth of water applied to any point should not be greater than that which will freeze completely in 24 hours. At temperatures of 0°F to -10°F and with a little wind, this is about 4 inches of water. A cooling period equal to the freezing period should be allowed before the area is reflooded. Such cooling is necessary for restoration of ice temperature and recovery of ice strength. More water should not be applied until all areas of the previous flood have frozen solid. Premature reflooding of an unfrozen area is very undesirable, since the freezing time increases exponentially with depth. Any air bubbles that form in the flooded ice should be broken.

Duff (1958) described ice landing construction and maintenance for logging operations. Any snow cover on the ice sheet should be compacted by rolling before flooding. Slush usually forms after a snowfall as free water comes up through ice sheet cracks. Rolling immediately after a snowfall--or during the storm if it is a severe one--improves frost penetration and lessens the slush problem. (Can you imagine a city road crew rolling snow at the town harbor during a storm instead of plowing the streets?)

Rose and Silversides (1958) described the merits of surface flooding for increasing the speed at which ice thickens. They used the example of a 12-inch-thick ice sheet and 20°F weather. If 5 inches of water are added to the surface, it will take about 15 hours for it to freeze and give a total ice thickness of 17 inches. If the ice surface is bared and ice is to be formed on the underside of the 12-inches sheet, it will take 60 hours to add another 5 inches--or four times as long.

Ohstrom and DenHartog (1976) investigated the efficacy of adding reinforcement to an ice cover. The reinforcements tested included 1-inch diameter tree branches, 3/16-inch diameter wire rope and 9/16-inch half-round wood dowels. The reinforcing of ice consists of laying reinforcement material on the ice, then flooding the area and allowing the reinforcement to freeze into the sheet.

Ohstrom and DenHartog noted a definite advantage in using reinforcement, even if poorly placed. Reinforced ice carries load even after it cracks, so after the initial cracks there is still time to remove people and equipment before final breakthrough. The disadvantages to reinforcement are that the darker types of material absorb radiation, thereby weakening the ice cover. Also, in many cases, the time and effort required to reinforce the ice may exceed those required to achieve the same strength through simply thickening the ice sheet.

For optimum strength, reinforcement should be added to the side of the ice that carries the tensile forces. Initial cracking of the ice sheet is caused by tensile stresses near the bottom surface of the ice, but final breakthrough is caused by tensile stresses near the top surface. (I think that many of the geotextiles and soil reinforcements would be fully applicable to ice--except dark-colored materials).

Maintenance Work

An ice cover provides not only a suitable construction work platform, but also a good maintenance work platform. Early spring is a good time to schedule dock and facility repairs using the ice cover as a platform.

In closing, I have one example of spring repair work--not directly from the ice as such, but rather through the ice. How would you repair the decidedly



FIGURE 13.2: Timber Head Pier Damaged by Ice

damaged, deformed and defective Danish dock depicted in Figure 13.2? The mooring piles had to be removed and redriven, but what about the head pier piles?

That repair was accomplished rather easily with a vibratory earth compacter. With jetting equipment on the piles to loosen the sand, the vibratory roller was driven along the dock until the head pier was jiggled back into the soil. Simple!

POSTSCRIPT

Are you, too, now "hooked" on ice as I am? Perhaps not. Nevertheless, if you need any help in this regard, please contact me by writing to the Department of Engineering and Applied Science, University of Wisconsin-Extension, 432 N. Lake Street, Madison WI 53706, or by phoning (608) 262-0577. For help with related Great Lakes problems, contact Sea Grant Advisory Services, University of Wisconsin, 1800 University Avenue, Madison WI 53705, phone (608) 262-0645. Copies of this and other publications are available from the Sea Grant Communications Office at the same address.

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