CHAPTER 3

SUMMARY OF DESIGN LOADS

Sound engineering design of any structure must include a discussion of design loads. In the case of docks, piers, and wharves, the major environmental design loads are waves, wind, current, and ice. Loading conditions that may be considered man-made include boat impact, and dead and live loads. While it is not usually practical to design for catastrophic loads, it is important to discuss them, as well as precautionary measures that may help avoid damage.

It is also important to compute accurately the magnitude of each load category as the cost of the finished structure is directly proportional to the severity of the design loads. Loads should be calculated on the basis that all berths are occupied by the largest vessel that can be accommodated since wind, wave, and current forces are generally assumed to act on the boat hulls and have negligible effect on the structure alone. Hubbell and Kulhawy (1979b) discuss general environmental loads acting on coastal structures. The following section presents methods for estimating the loads to be used in design.

3.1 DESIGN WAVE AND WAVE FORCES

A harbor by definition should provide a place of refuge for boats with a protected entrance to allow for safe access. There exists some maximum acceptable wave height within a harbor under which a boat may be handled safely without undue hazard to either the boat underway or the surrounding harbor structures. Depending on the characteristics of the using craft, the normal criteria for acceptable maximum wave
height are 2 to 4 ft (0.61 to 1.22 m) in the entrance channel, and
1 to 1.5 ft (0.30 to 0.46 m) in the berthing area (Dunham and Finn, 1974). Since floating docks derive their support from the water
surface, they are obviously much more susceptible to wave action than
are fixed docks. A wave height of about 2 ft (0.61 m) is the maximum
allowable for any floating system for structural design reasons. This
value is generally used as a design wave for small craft harbors, and
protective structures must be provided to shelter inner harbor facilities
in locations subject to greater wave energy.

Wave energy within a harbor may have three sources, including wave
energy generated within the harbor by boats, wave energy generated within
the harbor by wind, and external wave energy passing through the harbor
entrance.

Boat-generated wave energy (wake) is a function of boat displacement,
speed, and distance from the sailing line. As displacement and distance
are not practical variables in minimizing boat wake, it is common to
post a five mile per hour (8 kph) speed limit on the busier waterways.
Speed limits must be strictly enforced if the harbor structures are
expected to reach their design lives without major maintenance. Boat
wake generation is discussed by Seymour (1977) and Das (1969).

Wave energy generated within the harbor is usually negligible for
small craft marinas. Local wind waves may become appreciable, however,
if long unrestricted overwater fetches within the berthing area are
aligned with the prevailing winds, or under hurricane conditions. The
result will be short period, steep sided waves that cause excessive
agitation in the berthing area. These local wind waves may present
a severe wave loading condition since their short periods may correspond to the natural period of oscillation of berthed small boats. Hubbell and Kulhawy (1979b) review techniques for predicting waves. If it is found that local wind waves may be a problem, alteration of the harbor geometry or construction of an inner-harbor protective structure is required.

External wave energy may only enter the harbor by way of an uninterrupted waterway or by overtopping the harbor boundaries. In the process of harbor planning, breakwaters and jetties are commonly provided to furnish protection for the harbor area, and create a safe, convenient navigable entrance. To obtain an acceptable wave environment with minimal harbor surge, the planner must design the entrance (using variable geometry and orientation) such that the external wave energy is properly attenuated. Obviously this process can only be started with a study of the wave input: wave height, period and direction. Next a refraction and diffraction diagram analysis should be performed to determine the optimum orientation of the protective structures (See Hubbell and Kulhawy, 1979b). Finally, model studies may be useful to check the actual performance of the layout and make adjustments if necessary to obtain maximum attenuation.

In a discussion of wave forces on docks and piers, a distinction must be made between breaking and non-breaking waves. Breaking waves create a state of dynamic loading in which air pressures and impact must be considered. Non-breaking wave forces are not as abrupt and are usually applied to the structure as a static load. According to Hubbell and Kulhawy (1979b), wave breaking is likely to occur for basin depths less than 1.5 times the incident wave height. The suggested 2 ft (0.61 m) design wave will not break in the berthing area of a marina since
the depth required for navigation is greater than 3 ft (0.9 m). Therefore, the assumption will be made that all wave forces are caused by non-breaking 2 ft (0.61 m) waves, and these loads will be applied as static loads. It should be noted, however, that waves are a cyclic phenomena and that fatigue of the structural connections may be a problem.

Dock structures are usually analyzed for wave loading applied in the principal directions, parallel and perpendicular to the axis of the main walkway. If the structure has adequate strength in these directions, it has been found that all other orientations will be satisfactory as well (Dunham and Finn, 1974).

Horizontal wave forces on a floating body may be determined using the Froude-Kriloff theory as described by Brater, McNown, and Stair (1958). This procedure is presented graphically in Figure 3.1 and is based on the assumption that the berthed craft can be approximated by a barge-like hull shape that is in contact with the pier (Winzler and Kelly, 1979). Given the characteristics of the berthed vessels and the approaching wave, the wave force \( F_{wl} \) on a floating object is taken from Figure 3.1 in terms of the object's displaced volume. Figure 3.2 may be used to estimate the displacement volume \( V_d \) of small craft as a function of their overall length. Figure 3.3 is then consulted to adjust the final wave force for the length of the floating body relative to the wave length. A sample calculation of wave force is demonstrated in Design Example 3.1

In the case of floating docks, two additional wave load situations must be considered. First, when the incident wave is travelling parallel to the structure face, buoyancy will not be uniform along the length
Figure 3.1  Horizontal Wave Force on a Floating Object (Winzler and Kelly, 1979, p. III-16, after Brater, McNown and Stair, 1958)
Figure 3.2 Small Craft Weight and Displacement Volume
(State of California, 1980, p. 16)
Figure 3.3 Wave Force Adjustment for Relative Body Length
(Winzler and Kelly, 1979, p. III-17)
Design Example 3.1

Given:

A deep draft recreational vessel with an overall length of 30 ft, width of 7.5 ft, berthed in a saltwater mooring, and moored broadside to the direction of wave attack.

Find:

Wave force \( F_{w1} \) to be resisted by each finger pier because of a wave of 20 ft length and 2 ft height.

\[
\text{Figure 3.1} \quad F_{w1} = 5.8 \frac{\text{lbs}}{\text{ft}^3 \text{ displacement}}
\]

\[
\text{Figure 3.2} \quad V_d = 157.4 \text{ ft}^3
\]

\[
\text{Figure 3.3} \quad 2/l_w = \frac{7.5}{20} = 0.375
\]

Body adjustment factor \( P = 0.92 \)

Therefore:

Wave force on each finger pier

\[
F_{w1} = 5.8 \frac{\text{lbs}}{\text{ft}^3} \times 157.4 \text{ ft}^3 \times 0.92
\]

\[
F_{w1} = 839.88 \text{ lbs}
\]

\[
= 0.84 \text{ kip}
\]
of the float system. The system should be analyzed for the location of
the incident wave where it produces maximum moment and shear forces.
Second, horizontal wave forces on the dock profile are usually considered
negligible where deep draft craft (2 to 8 ft : 0.61 to 2.44 m) are
moored at the berth. When shallow draft craft (less than 2 ft or 0.61 m)
are moored at the berth, however, the wave force on the face of the dock
should be estimated. The dock width is not to be added to the craft
width to obtain the body length used in Figure 3.3 since the connection
is not rigid. The displacement volume \( V_d \) of the dock may be found
using the dead and live load weights described later in this chapter
and Figure 3.3 of this section.

Attempts have been made to refine the wave force analysis to recog-
nize the fact that most boats are moored in such a way that the hull
is not in contact with the dock (See Section 6.6). Unfortunately,
the analysis becomes complex as the vessel is able to translate relative
to the dock. This movement is restrained by the mooring lines, but
the load must still be considered dynamic. For example, Raichlin (1968)
developed an analytical model in which the restoring forces of quasi-
elastic mooring lines respond in a nonlinear fashion to boat displace-
ment as a result of wave impulse loads. The restoring force predicted
with this method correlated well with measured values for a series of
field tests.

3.2 WIND LOADS

The maximum lateral load on a harbor structure is most often the
result of wind pressure. Strong, steady winds usually cause loads greater
than those produced by waves, current, or impact. The wind velocity,
shape of the exposed object, and the severity of gusts are factors influencing the design wind load which is usually expressed as a pressure acting on the above-water profile.

In many areas, the design wind velocity may be taken from local building codes. Where these specifications are not available, local wind records, or isotachs such as that shown in Figure 3.4 should be used. In the case of the local wind records, it is important to ascertain how and where the measurements were made. The wind velocity variation with elevation is generally assumed to be logarithmic with near-surface winds being much less severe than those measured at the standard altitude of 30 ft (9.1 m). Figure 3.5 is a dimensionless plot of altitude \( Z \) versus mean wind velocity \( \bar{V}_w \), and may be used to reduce wind velocity measured at standard elevation to a design wind velocity at the level of the overwater profile. Figure 3.6 is presented to permit reduction of wind velocity not measured at standard height. The relationship used to develop this profile is taken from Linsley, Kohler, and Paulhus (1975) and is expressed as follows:

\[
\frac{\bar{V}_w}{\bar{V}_{w1}} = \frac{\ln \left( \frac{Z}{z_o} + 1 \right)}{\ln \left( \frac{Z_1}{z_o} + 1 \right)}
\]  

(3.1)

in which: \( \bar{V}_w \) = unknown mean velocity at profile height \( Z \)  
\( \bar{V}_{w1} \) = measured mean velocity at altitude \( Z_1 \) (usually 30 ft or 9.1 m)

and \( z_o \) = roughness length (Table 3.1)

The roughness length \( z_o \) of Equation 3.1 is defined as the height above the surface at which the wind velocity is zero. Values of roughness length are presented in Table 3.1 for various terrains including a rough
Figure 3.4  Isotach of Maximum Wind Velocity (mph), 30 ft (9.1 m) above ground, 50 year recurrence period (Dunham and Fink, 1974, pp. 134) (1 mph = 1.61 kph)
\[
\frac{\bar{V}_{w}}{\bar{V}_{w1}} = \frac{\ln \left( \frac{z}{z_o} + 1 \right)}{\ln \left( \frac{z_1}{z_o} + 1 \right)} \quad \text{for } z_{w1} = 10.0 \text{ m}
\]

- \(z_o = 5 \text{ mm}\)
- \(z_o = 10 \text{ mm}\)

**Figure 3.5** Mean Wind Velocity versus Altitude (After Linsley, Kohler and Paulhus, 1975, p. 43)
\[ \frac{\bar{V}_w}{\bar{V}_{w1}} = \frac{\ln \left( \frac{z_1}{z_0} + 1 \right)}{\ln \left( \frac{z_1}{z_0} + 1 \right)} \quad \text{for} \quad z_0 = 0.005 \text{m} \]

Figure 3.6 Mean Wind Velocity versus Profile Height
(After Linsley, Kohler and Paulhus, 1975, p. 43)
<table>
<thead>
<tr>
<th>Terrain</th>
<th>Gradient Height $z_g$ (ft)</th>
<th>$z_g$ (m)</th>
<th>Surface Drag Coefficient $k$</th>
<th>Roughness Length $z_o$ (in.)</th>
<th>$z_o$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rough Sea</td>
<td>825</td>
<td>250</td>
<td>0.001</td>
<td>0.2 to 0.4</td>
<td>5 to 10</td>
</tr>
<tr>
<td>Open Farmland</td>
<td>975</td>
<td>300</td>
<td>0.005</td>
<td>0.4 to 5.0</td>
<td>10 to 100</td>
</tr>
<tr>
<td>Forest and Suburban Areas</td>
<td>1300</td>
<td>400</td>
<td>0.015</td>
<td>12.0 to 40.0</td>
<td>300 to 1000</td>
</tr>
<tr>
<td>City Centers</td>
<td>1650</td>
<td>500</td>
<td>0.050</td>
<td>40.0 to 200.0</td>
<td>1000 to 5000</td>
</tr>
</tbody>
</table>
sea. The use of the minimum suggested roughness length (0.2 in. or 5 mm) is a conservative assumption that would, to some degree, compensate for gust loads. Wind gusts are usually assumed to have negligible effect because of their short duration, the high inertia of the boats, and the flexibility of the mooring system.

After the mean wind velocity has been determined, it must be resolved into a force acting on the structure. Hubbell and Kulhawy (1979b) present the following expression relating mean wind velocity to the wind pressure ($P_w$) that acts on the above-water profile of a dock system:

$$ P_w = 0.004 \bar{V}_w^2 $$

(3.2)

in which: $\bar{V}_w = \text{mean wind velocity (mph)}$

$P_w = \text{wind pressure because of} \bar{V}_w \text{(psf)}$

This relationship is presented graphically in Figure 3.7. The wind pressure from Equation 3.2 or Figure 3.7 must be multiplied by the appropriate above-water profile area to determine the actual wind load. Figure 3.8 presents the profile height ($h$) of small craft as a function of their overall length ($l$). A sample calculation of wind force ($P_{w2}$) may then be performed as in Design Example 3.2. Note that in Design Example 3.2, the profile of the small craft is taken as its overall length times its height, resulting in a flat-sided, barge-like shape. For most modern recreational boats, this assumption is quite conservative.

During the structural analysis phase of dock design, wind loads must be applied to individual fingers as well as to the entire system. Wind loads over the length of a dock can be greatly reduced because of a shielding effect of the boats to the windward side. Winzler and Kelly
For use in determining design wind stress on small-craft berthing systems, and buildings only. These wind speeds are not to be used to determine the design waves.

Figure 3.7 Horizontal Wind Pressure on a Vertical Face
(Dunham and Finn, 1974, p. 134)
Figure 3.8 Above Water Profile Height versus Length of Craft (Dunham and Finn, 1974, p. 135)
Design Example 3.2

Given:

A single power boat 30 ft long, moored broadside to the direction of wind movement.

Find:

The wind force \( F_{w2} \) to be resisted by each finger pier because of a wind with an average velocity \( \bar{V}_w \) of 60 mph.

- **Figure 3.7** Wind Pressure \( P_w = 14.5 \text{ psf} \)
- **Figure 3.8** Profile height \( h = 4.9 \text{ ft} \)

\[
F_{w2} = 14.5 \text{ psf} \times 4.9 \text{ ft} \times 30 \text{ ft} \\
F_{w2} = 2131.5 \text{ lb} \\
= 2.1 \text{ kip}
\]
(1979) recommend a design value of 15 percent of the full wind pressure be applied to boats that are shielded by other boats, or structures, while Dunham and Finn (1974) suggest 20 percent.

3.3 CURRENT LOADS

The primary sources of water currents include river flow, tidal variations, and harbor surge. River currents vary little in magnitude (peaking at the flood stage of the river) and are constant in direction. Tides on the other hand produce a sinusoidal load that reverses direction in a predictable manner. Where river flow and tides interact along a coast line, currents may be much stronger in one direction controlled by the constant flow of the river. Harbor surge occurs primarily because of surf beat resonance.

When a resonant wave system is set up within a harbor, currents are produced at the nodal points of the waves. At this location, the vertical motion is negligible, but the horizontal motion of the water particles may be quite strong. Figure 3.9 illustrates the superposition of waves because of resonance and the resultant harmonic motion. A good discussion of resonance is presented by Calvin (1969). Nodal current velocity may be from 2 to 4 fps (0.6 to 1.2 m/s) while currents from tidal action are typically an order of magnitude less and may be considered negligible. Because of the large variation of current speeds within the berthing area, no standard minimum pressure has been adopted and current load design is performed on the basis of the maximum expected current. Current load from harbor surge must therefore be used uniformly unless it can be proven that resonance will not occur.
(a) SUPERPOSITION OF WAVES; (b) GEOMETRY OF NODE AND ANTINODE

DISTRIBUTION OF NODES AND ANTINODES: (a) SECOND HARMONIC FOR CLOSED HARBOR; (b) FUNDAMENTAL FOR OPEN HARBOR; (c) SECOND HARMONIC FOR OPEN HARBOR

Figure 3.9 Superposition of Waves and Harmonic Motion (Galvin, 1969, pp. 78 and 81)
Current force, $F_c$, is related to current velocity through the following expression:

$$F_c = P_c \times A_{c2} = \frac{\gamma_w \times V_c^2}{2g} \times A_{c2} \tag{3.3}$$

in which:  
- $P_c$ = current pressure (psf or kN/m²)  
- $\gamma_w$ = unit weight of water (pcf or kN/m³)  
  - 62.4 pcf (9.8 kN/m³) fresh water  
  - 64.0 pcf (10.1 kN/m³) salt water  
- $V_c$ = current velocity  
- $g$ = constant of gravitational acceleration  
  - 32.2 ft/sec² (9.81 m/sec²)  

and  
- $A_{c2}$ = underwater profile area

Figure 3.10 relates current pressure to current velocity for both salt and fresh water using Equation 3.3. Since there is only a 3 percent difference between these curves, the salt water curve may be used when the composition of the water is uncertain without being over-conservative. Figure 3.11 shows underwater boat profile height versus boat length and should be used to compute the underwater profile area, $A_c$. As in the case of wind loading, underwater profile height is often assumed to be 15 percent of slip length (Winzler and Kelly, 1979). Figure 3.11 shows, however, that this approximation is only accurate for commercial fishing boats and may be in error by ± 2 ft (0.61 m) depth for a 40 ft (12.2 m) boat.

The application of current loads to a berthing facility is performed in the same manner as wind loading. Winzler and Kelly (1979) again recommend applying 15 percent of the maximum current load to shielded
\[ P_c = \frac{\gamma_w \times V_c^2}{2g} \]

\( \gamma_w = 62.4 \text{pcf (9.8 kN/m}^3) \) freshwater
\( \gamma_w = 64.0 \text{pcf (10.1 kN/m}^3) \) saltwater
\( g = 32.2 \text{ ft/sec}^2 (9.81 \text{ m/sec}^2) \)

Current Velocity - \( V_c \) (fps)
(1 fps = 0.3048 m/s)

Figure 3.10 Current Pressure versus Current Velocity
(After Quinn, 1972, p. 296)
Figure 3.11 Underwater Profile Height versus Boat Length
(Weizler and Kelly, 1979, p. III-11)
bulls. Cheung and Kulhawy (1981) may be consulted for current forces on piles.

3.4 BOAT IMPACT

Impact loads occur as two objects collide. Minor collisions are a common event in small craft harbors as boats are maneuvered into their berths. Since direct contact of the boat and dock may result in damage to both, some form of protection or "fendering" is commonly provided to absorb the energy of impact:

Impact may be seen as a form of kinetic energy:

\[
\text{Impact} = \text{K.E. (Kinetic Energy)} = \frac{1}{2} W_{\text{min}} v_B^2 \frac{v_B}{g}
\]

(3.4)

where:
- \( g \) = constant of gravitational acceleration
  - \( = 32.2 \text{ ft/sec}^2 \) (9.81 m/sec\(^2\))
- \( V_B \) = velocity of boat normal to the dock
- \( W_{\text{min}} \) = weight of boat
  - \( = 12 \text{ L}^2 \) for pleasure boats
  - \( = 25 \text{ L}^2 \) for commercial boats

and
- \( L \) = length of boat

This relationship is presented in a convenient graphical form in Figure 3.12. Figure 3.2 depicts boat weight as a function of length.

The variables of boat impact as shown above are boat weight and velocity. Weight may not be considered a true variable since it depends on the geometry of the slip which must be designed to withstand the impact of the largest boat it can accommodate. Note that the weight curves of Figure 3.2 are minimum values and should be adjusted upward by the designer to account for special passenger or cargo loads.
Figure 3.12: Docking Impact Energy for Small Craft
(State of California, 1980, p. 6)
Impact velocity depends not only on pilot skill but also meteorological and oceanographic conditions (Cheung and Kulhawy, 1981). As the speed of approach must be assumed, herein lies the greatest uncertainty. The impact velocity is a critical parameter since impact energy varies with the square of the velocity. Winzler and Kelly (1979) recommend that a velocity of one foot per second (0.3 m/s) normal to the dock be assumed for small boats. This is also a minimum value and should be increased subject to the designer's discretion, if difficult docking conditions are anticipated.

Figure 3.13 illustrates a boat entering a slip and impacting one of its sides. In adverse conditions, the boat may not enter the slip perfectly and will, therefore, contact the slip at some small angle. Quinn (1972) suggests an approach angle of 10 degrees with respect to the face of the dock to be used for design purposes. Note that a velocity of 1 fps (0.3 m/s) normal to the dock corresponds to an approach velocity of \( \frac{3}{2} \) knots (1.0 m/s) for an angle of 10 degrees.

The energy to be absorbed by the dock and fender system is usually taken to be half of the kinetic energy as obtained in Figure 3.12. Figure 3.13 shows that the point of contact is assumed to be at the one fourth point along the boat, and that the impact energy acts through the center of gravity of the boat. As a result, the center of gravity tends to rotate about the point of contact, causing a hydrostatic pressure build-up along the side of the boat that absorbs some of the energy of impact. Figure 3.14 relates the percentage of impact energy \( (K_b) \) that must be absorbed by the fender system, to the berthing contact
c.g. - Center of Gravity
$V_A$ - Approach Velocity
$V_B$ - Velocity of Boat Normal to Dock
$E$ - Impact Energy
$L$ - Overall Boat Length

Figure 3.13 Assumed Geometry for Boat Impact Analysis
(After Quinn, 1972, p. 384)
Figure 3.14 Impact Energy Reduction Factor versus Berthing Point (Quinn, 1972, p. 385)
point along the boat ($K_b = 0.5$ for contact of $L/4$). Note that if the boat strikes at its midpoint all of the impact energy must be acting on the dock directly. This maximum value ($K_b = 1.0$) should be used for small boats when adverse conditions such as high winds or turbulent water are expected since the orientation of approach will be uncertain. The reduction in impact energy should be made, however, for larger boats and calm weather conditions.

Impact loading is the primary design criterion of fendering systems. Fenders act to distribute impacts into the major structural components of the dock without causing large stress concentrations. Since impact tends to be of short duration, it should not be combined with large wind or current forces. Impact energy has the dimensional units of work ($ft$-$lbs$) and must be absorbed through a displacement of the fender. Impact is obviously a case of dynamic loading, but analysis of a structural system for such a load is very complex. It is common practice to apply impact as a static load so that the relationship between impact energy and fender deformation becomes:

$$KE \times K_b = k\Delta^2 \tag{3.5}$$

where:

- $K_b =$ reduction coefficient for berthing point (see Figure 3.14)
- $k =$ stiffness of dock and fender system ($k/ft$)
- $\Delta =$ deformation of the fender under impact ($ft$)

The stiffness ($k$) of a fender is defined as the force caused by a unit displacement, and is a function of the material properties of the fender, and the structural design of the supporting framework. The necessary
stiffness is determined by the maximum deformation that can be tolerated.

3.5 ICE LOADS

Northern harbors present a very hostile environment for harbor structures. Ice forces are responsible for extensive damage to boats, docks and piers. While it is possible to avoid damage to most boats by removing them from the water, large floating docks, fixed docks, and piers must be designed to resist ice loads. The ice forces themselves may be minimized by reducing the ice sheet thickness with compressed air bubbler systems. The design details and limitations of these bubbler systems are discussed by Ashton (1974).

Significant vertical and horizontal loads result from the formation of ice in a harbor. Vertical loads are caused by ice "grip" as ice adheres to the surface of floats, piles and bracing. Fluctuation of the water level then imposes load from the water-ice system on to the structure. Sieche action (responsible for most winter water level fluctuation) is the short term rise and fall of water level caused by persistent strong winds piling up water, or because of changes in barometric pressure over the lake (Wortley, 1979). The period of a sieche may range from a few minutes for a bay or harbor to ten hours for a very large lake, and water level changes of 3 in. (76 mm) in ten minutes are common. Uplift forces are caused by bouyancy of the ice as the water level seeks to rise. Downdrag results when the water level lowers and the ice sheet becomes an added "dead load" as it hangs up on the structural members.

Lateral ice loads may be broken down into two categories, including
thermal expansion loads and ice floe impact loads. Expansion of ice as it freezes generates tremendous compressive forces if the ice is confined. In some cases horizontal pressures of 400 psi (2.7 MN/m²) (the approximate crushing strength of ice) have been used for pier design, as reported by Wortley (1979) but this may be overly conservative. In practice, ice crushing strength is less because of impurities and cracking. Ice floe impact is caused by the momentum of moving ice blocks and is often ignored as a design criterion. Wortley (1979) observed that the horizontal forces of the moving pieces generally do not exceed the mooring forces for which the docks are designed, so no special analysis is required.

The engineering characteristics of ice and the different forms of ice loading are discussed further by Hubbell and Kulhawy (1979b). Procedures are presented to estimate the magnitude of ice loads. Note that the flexibility of the dock system, and the rate of deformation have considerable influence on ice forces. These variables are often site specific and must be recognized by the designer when applying ice loading to the trial design. Another important parameter, ice thickness, is dependent on location, severity of the winter, and the salinity of the water. Salt water typically freezes at 28°F (-2°C) while fresh water freezes at 32°F (0°C). An ice sheet in salt water will always be thinner than in fresh water for the same temperature conditions, thus reducing the ice load. Where salinity is uncertain, it is conservative to assume fresh water.

3.6 DEAD AND LIVE LOADS

A dead load by definition includes the combined weights of all the components that are considered permanent in a structural system. For a fixed pier, the dead load will be the sum of the weights of all
piling, pile caps, stringers, decking, fenders, hardware, and any 
permanently attached accessories. In the case of a floating dock, the 
piling, pile caps, and stringers are replaced by flotation units, wales, 
and gangways. Permanent accessories include pipes, pumps, utilities, 
fire fighting equipment, storage lockers, etc. The dead load in its 
various combinations must be added to the live load to develop a design 
load for each structural component.

The unit weight of the timber used in construction should be assumed 
to be a minimum of 35 pcf (5.5 kN/m$^3$). Dry Douglas fir or Southern pine 
for example may have a dry unit weight of 35-40 pcf (5.5 - 6.3 kN/m$^3$), 
disregarding the retention weight of water or preservatives. Actual density 
in service will depend on the species of tree, moisture content, and the 
type of preservative treatment. Hubbell and Kulhawy (1979a) may be con-
sulted on wood properties and preservatives. Reinforced concrete using 
standard aggregate has a unit weight of 150 pcf (23.6 kN/m$^3$). The unit 
weight of steel is 490 pcf (77.2 kN/m$^3$) but weight per lineal foot is 
usually specified by the manufacturer for common structural shapes. 
Aluminum has a unit weight of approximately 169 pcf (26.6 kN/m$^3$), but this 
may vary considerably depending on the type of alloy. As in the case of 
steel, manufacturers supply the weight per lineal foot of aluminum members.

Live load criteria are usually specified by local building codes. 
In the absence of such specifications, there are accepted minimum loads 
that should be applied to the various types of structures.

Fixed piers are normally designed for a deck live load of not less 
than 100 psf (4.8 kN/m$^2$) of deck area on main walkways. Finger piers 
having limited access are often designed for 50 psf (Dunham and Finn,
1974). This design live load must be adjusted by the designer if vehicular traffic or heavy cargo is anticipated. Piers upon which a vehicle may be driven should be designed for at least H 10-44 loading as specified by the AASHTO Standard Specifications for Highway Bridges (State of California, 1980). H 10-44 loading specifies a 20,000 lb (89 kN) truck having a 14 ft (4.3 m) wheelbase and a front/rear axle weight distribution of 4,000 lb (17.8 kN) and 16,000 lb (71.2 kN) respectively.

Floating docks must be provided with sufficient flotation to support their dead load plus a uniform live load of 20 psf (1.0 kN/m²). A floating pier used solely for pedestrian access to a floating structure, however, should be designed to support its dead load plus a minimum live load of 40 psf (1.9 kN/m²) since there will be heavy traffic. An exception may be made in the case of a rough water installation using a thin-deck laminated wood float system (State of California, 1980). Under these conditions, it is advantageous to maintain a "flexible" structure, and the use of the 20 psf (1.0 kN/m²) live load requirement results in a large number of flotation pontoons that act to "stiffen" the system. Under no circumstances should a live load less than 12 psf (0.6 kN/m²) be used for any floating dock.

Both fixed and floating structures must also be able to support a 400 lb (1.8 kN) concentrated load without overstressing the framing members or creating more than a 6° tilt of the deck surface. This concentrated load need not be applied simultaneously with the uniform live load previously discussed.

The State of California (1980) recommends that gangways up to 6 ft (1.8 m) in width be designed to support a minimum live load of 40 psf (1.9 kN/m²) while those greater than 6 ft (1.8 m) wide must support
at least 100 psf (4.8 kN/m²). Note that half of the live load on
the gangway must be transmitted to the end of a floating pier. Extra
flotation must be provided to support this concentrated load.

A final live load specification refers to walkway and gangway rail-
ings. The railings should be capable of withstanding a horizontal force
of 20 ppf (0.29 kN/m) at their highest point from the deck (State
of California, 1980).

3.7 CATASTROPHIC LOADS

Catastrophic loads are caused by meteorological events such as
earthquakes, hurricanes, and tsunamis. As stated in the introduction,
it is not practical to design docks, piers, and wharves to resist these
severe loads directly. Protection in the form of breakwaters or jetties
should be provided during the harbor planning and layout phase so that
the harbor performs as an integrated system.

In areas where the probability of catastrophic loading is high,
heavy emphasis should be placed on early warning and emergency evacuation
systems. If the marina operator is given sufficient warning of the ap-
proach of a storm it may be possible to secure the harbor and avoid major
damage. Some preparations that may be made include: the transfer of
small boats from the water to dry storage to minimize wind and current
loads, inspection of moorings to make sure that all lines are snug and
in good condition, checking the placement of portable fenders, disconne-
tion of electric and fuel lines in case of rupture, and clearing all
personnel from the area.

Tsunamis are long period (10-20 minute), high velocity (several
hundred miles per hour) waves of seismic origin (Dunham and Finn, 1974).
As these waves approach shore, they cause water level fluctuations like a rapid tide with a magnitude of several feet. Some design considerations for tsunami prone areas are as follows: anchorages must resist the lateral loads of tsunami-generated currents, anchor pile tops in a floating slip system must be high enough that the floats do not rise above them, pile guides should be provided with a barnacle shearing device to keep floats from hanging up on anchor piles as the trough passes, and the basin area should be dredged deep enough so that the berthed craft are not lowered to the bottom.

3.8 COMBINATION OF LOADS

It is clear that harbor structures must be designed to resist individually each of the load categories previously mentioned. While it is not likely that these loads would act on the structure simultaneously, it is important to consider possible combinations. Most of the loads are caused by the environment and typically fluctuate in both magnitude and direction with time. Current, wind, and wave forces may in some cases be directly additive, but it would not be reasonable to combine them all indiscriminantly at their maximum values. Two cases of combined loading will be considered for this report. Case 1 will apply wind pressure directed perpendicular to the structural element with waves of a suitable length to maximize lateral load also approaching perpendicular to the element. This combination produces the maximum horizontal load that can normally be expected. Case 2 maintains the wind pressure normal to the dock element, but applies a variable length wave moving parallel to the structure. The result is a combination of vertical and horizontal loads that may be critical for wale design and connections.
Current forces have not been included in the above combinations because the loads produced by harbor surge are very severe and harbor surge is a relatively rare occurrence that can be avoided through proper layout and planning. This is not to say that such current loads will not or do not occur. Winzler and Kelly (1979) suggest that to accommodate such site specific conditions, designs should assess environmental loading conditions that may require more stringent loads, or which may require reductions in allowable stress because of fatigue considerations. To be thorough, the analysis should be sufficiently detailed to apply to specific locations within the berthing area where increased loads may be experienced.

3.9 SUMMARY

The several types of loads that should be addressed in the design of harbor structures such as docks, piers, and wharves have been introduced herein. Procedures are presented to determine the forces because of wind, wave, current, boat impact, ice, and dead and live loads. Catastrophic loads caused by earthquakes, hurricanes, and tsunamis are also discussed briefly but it is not usually practical to incorporate them in structural design.

Most of these loads are a product of environmental conditions and are highly variable in magnitude and direction over time. The loads used in design must be appropriate to the location of the structure, taking into consideration site specific conditions. Underestimation of the design loads will usually result in premature failure while being overconservative causes costs to become excessive. Good judgment in the estimation of design loads is the first step toward obtaining proper performance at an acceptable price.