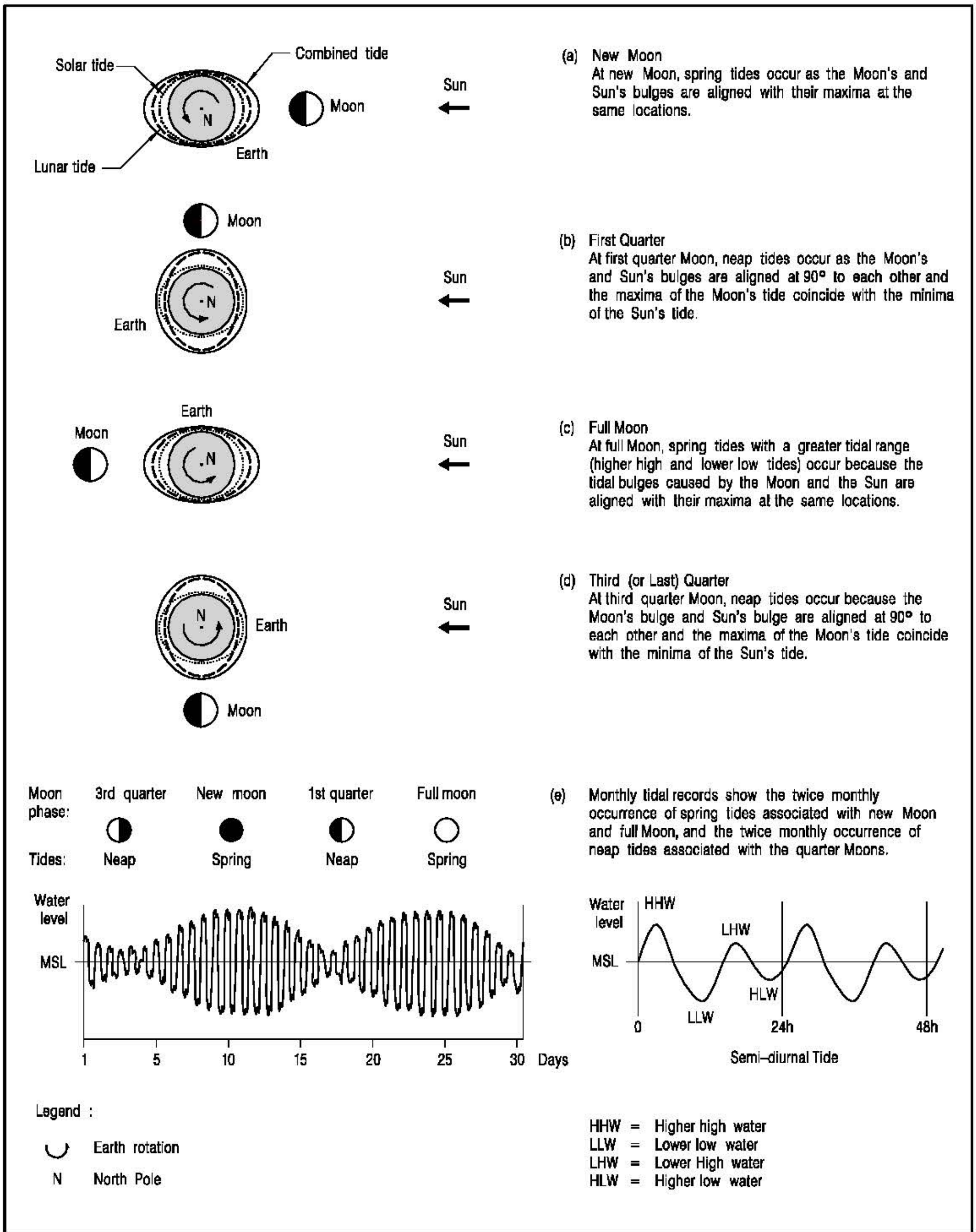
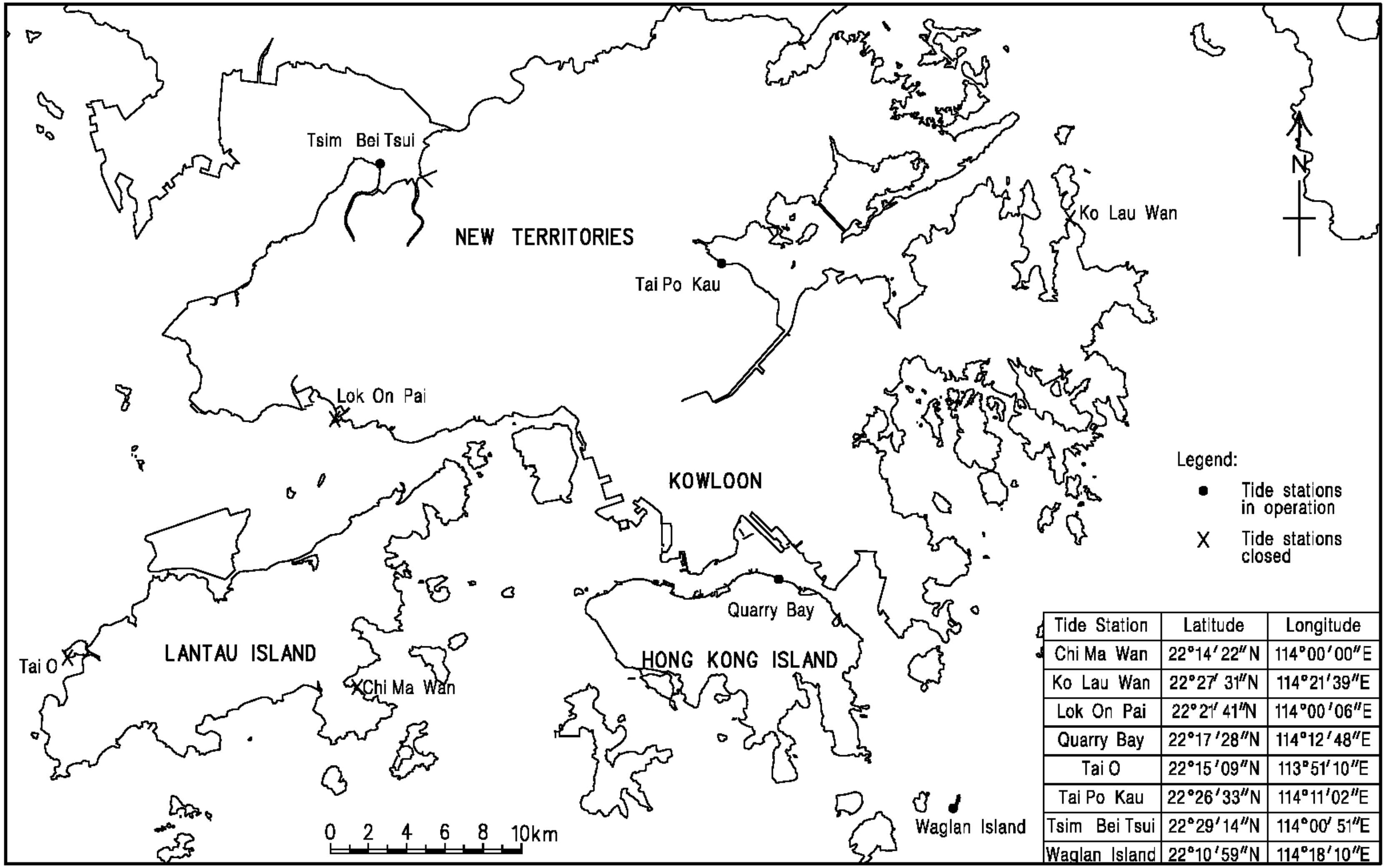


Notes:

1. The observed spring and neap sequences on Earth are more complicated for reasons that the orbits of the Moon and Sun are not in the same plane as this simplified figure depicts. These orbits are inclined to each other and the inclination changes with time.

2. Lunar Tide : Tide due to Moon only
   Solar Tide : Tide due to Sun only

Figure 1 – Simplified Earth/Moon/Sun System for Spring and Neap Tides



Tide Station	Latitude	Longitude
Chi Ma Wan	22°14′22″N	114°00′00″E
Ko Lau Wan	22°27′31″N	114°21′39″E
Lok On Pai	22°21′41″N	114°00′06″E
Quarry Bay	22°17′28″N	114°12′48″E
Tai O	22°15′09″N	113°51′10″E
Tai Po Kau	22°26′33″N	114°11′02″E
Tsim Bei Tsui	22°29′14″N	114°00′51″E
Waglan Island	22°10′59″N	114°18′10″E

Figure 2 – Locations of Tide Stations

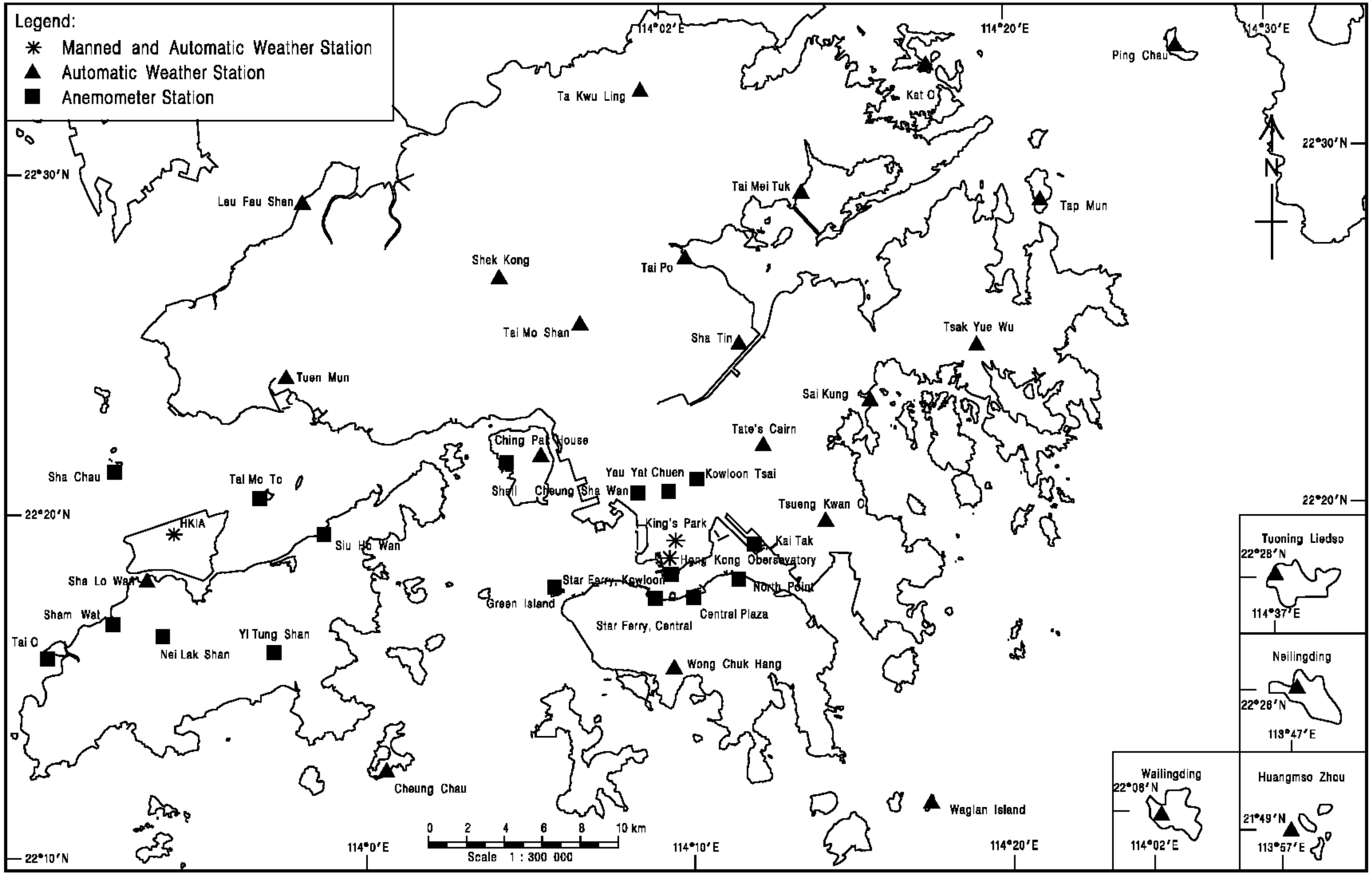
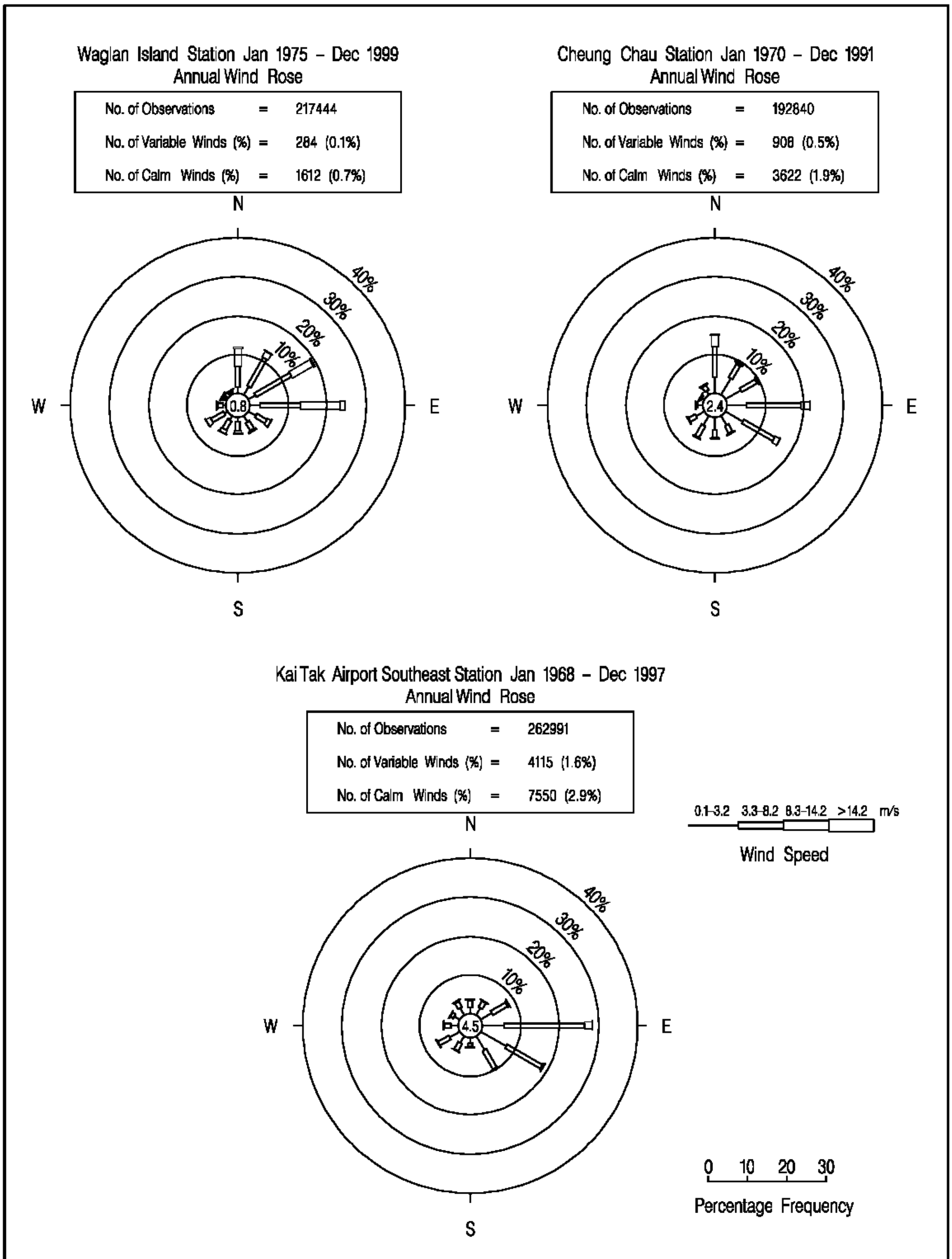


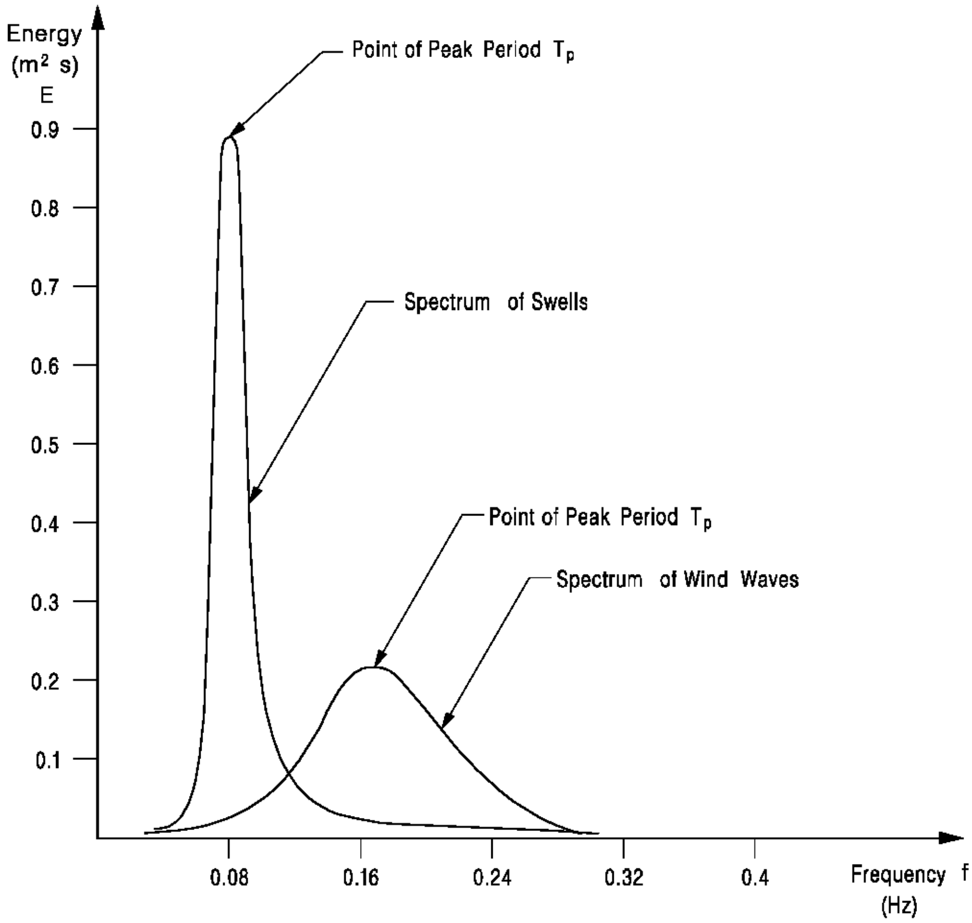
Figure 3 – Locations of Weather Stations



Note:

1. The number in the inner circle is the percentage frequency of occurrence of calm and variable winds.

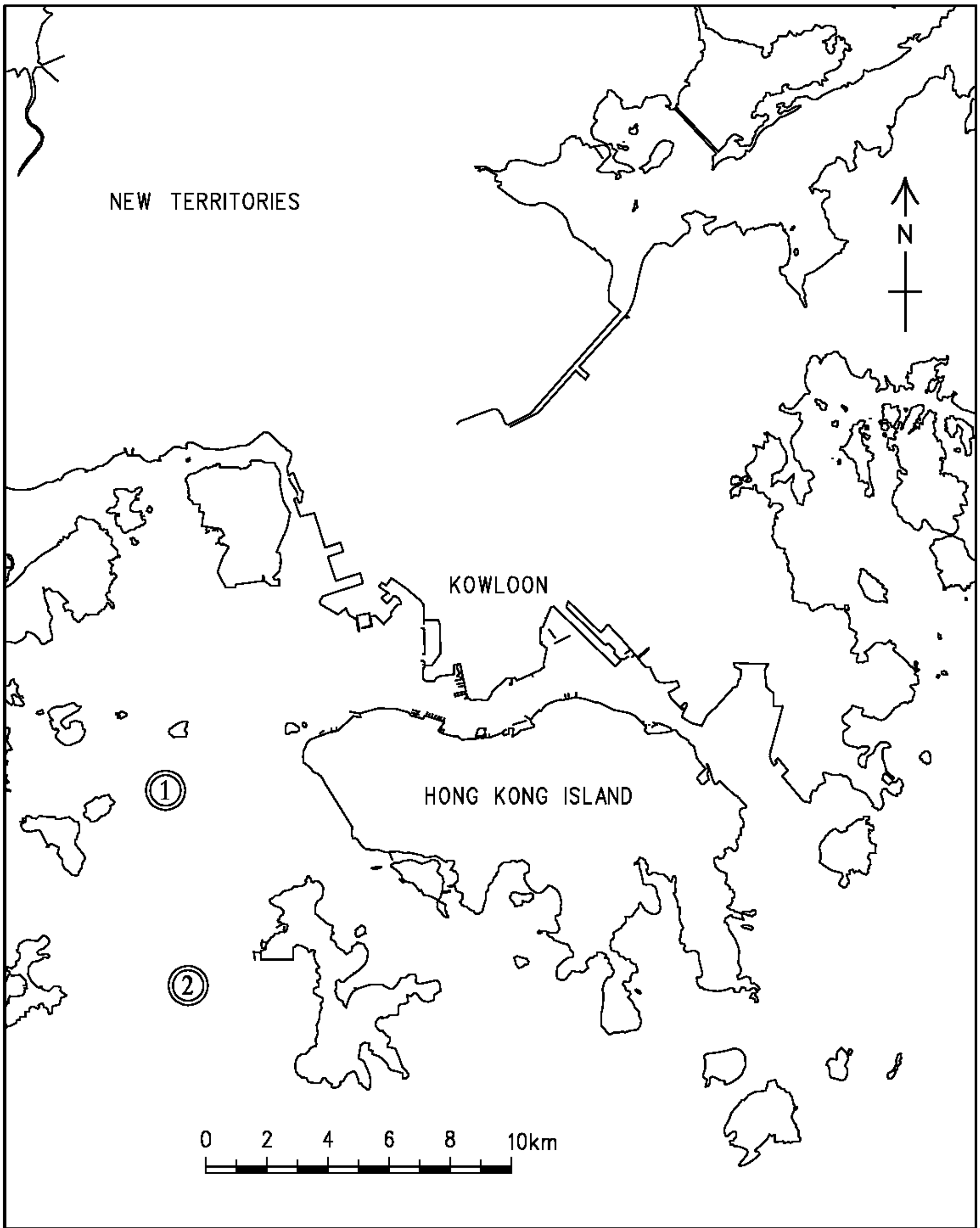
Figure 4 – Annual Wind Rose



Notes:

1. Period = $\frac{1}{\text{Frequency}}$
2. The figures on the two axes are indicative only.
3. m_i = i–th moment of the spectrum = $\int_0^\infty f^i E(t)df$; i = 0, 1, 2, 3....... and t = time domain.

Figure 5 – General Shape of Wave Spectrum

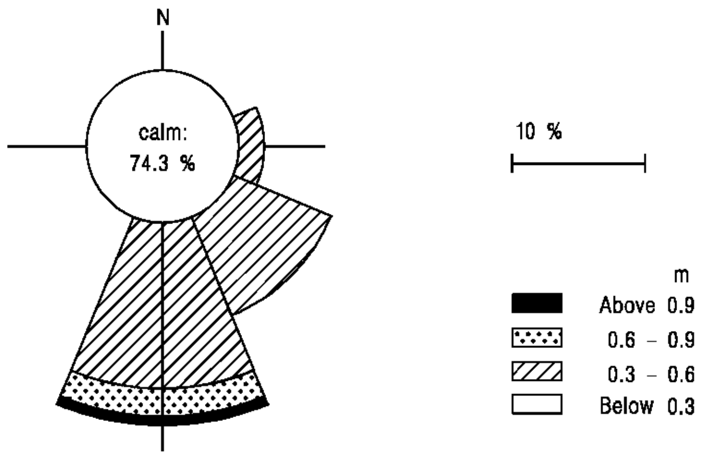


Notes:

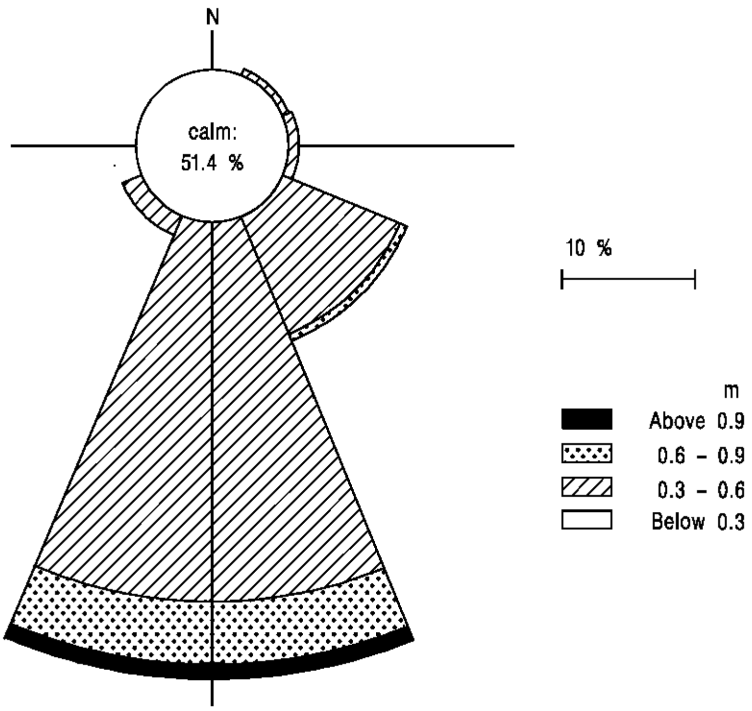
1. Kau Yi Chau Recorder at: Longitude 114° 4.28′ Latitude 22° 16.46′ in 1994, moved to Longitude 114° 3.7′ Latitude 22° 16.00′ on 26.1.1995

2. West Lamma Channel Recorder at: Longitude 114° 4.76′ Latitude 22° 13.02′

Figure 6 – Locations of Wave Stations



Wave Roses at Kau Yi Chau

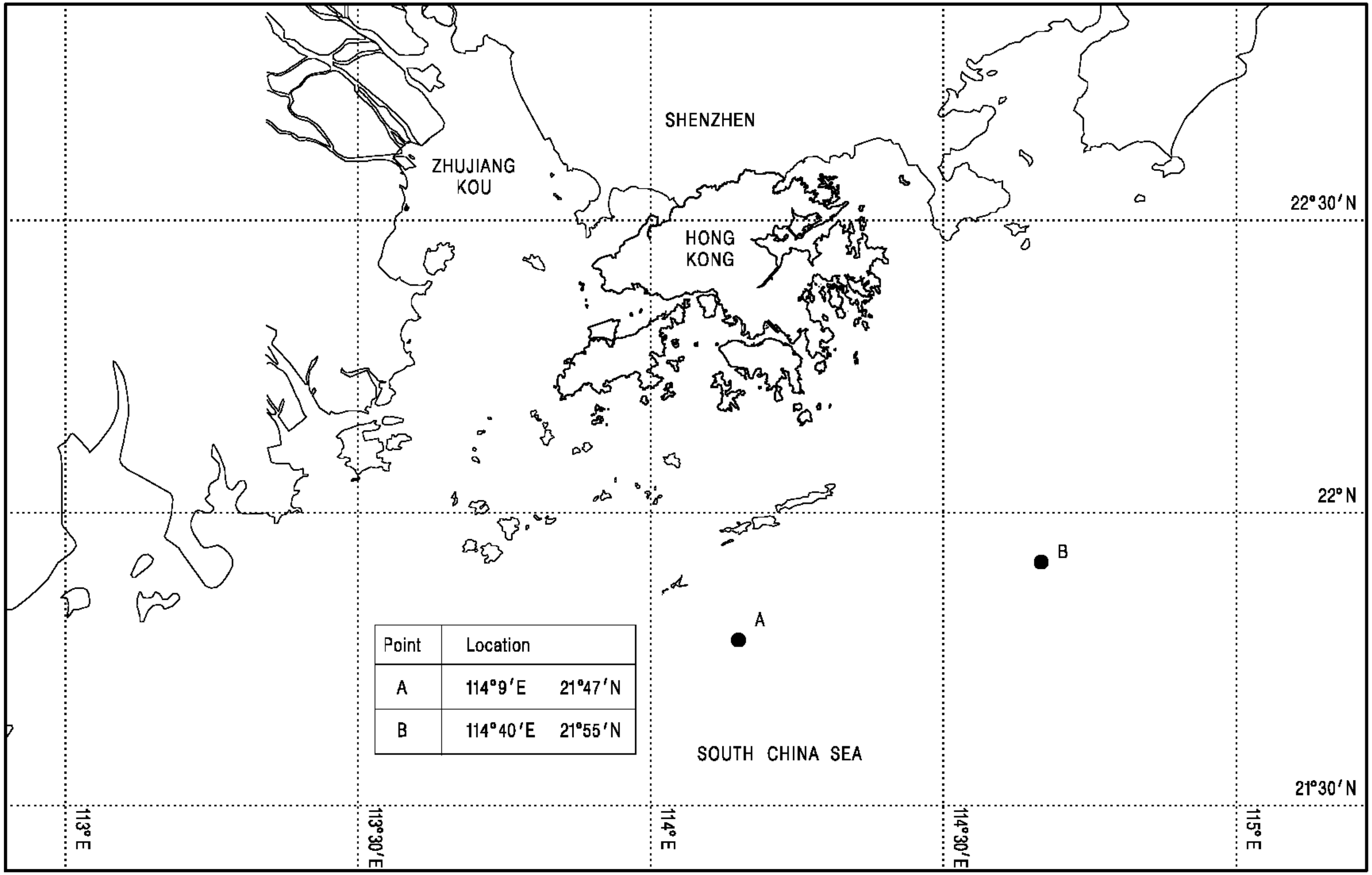


Wave Roses at West Lamma Channel

Notes:

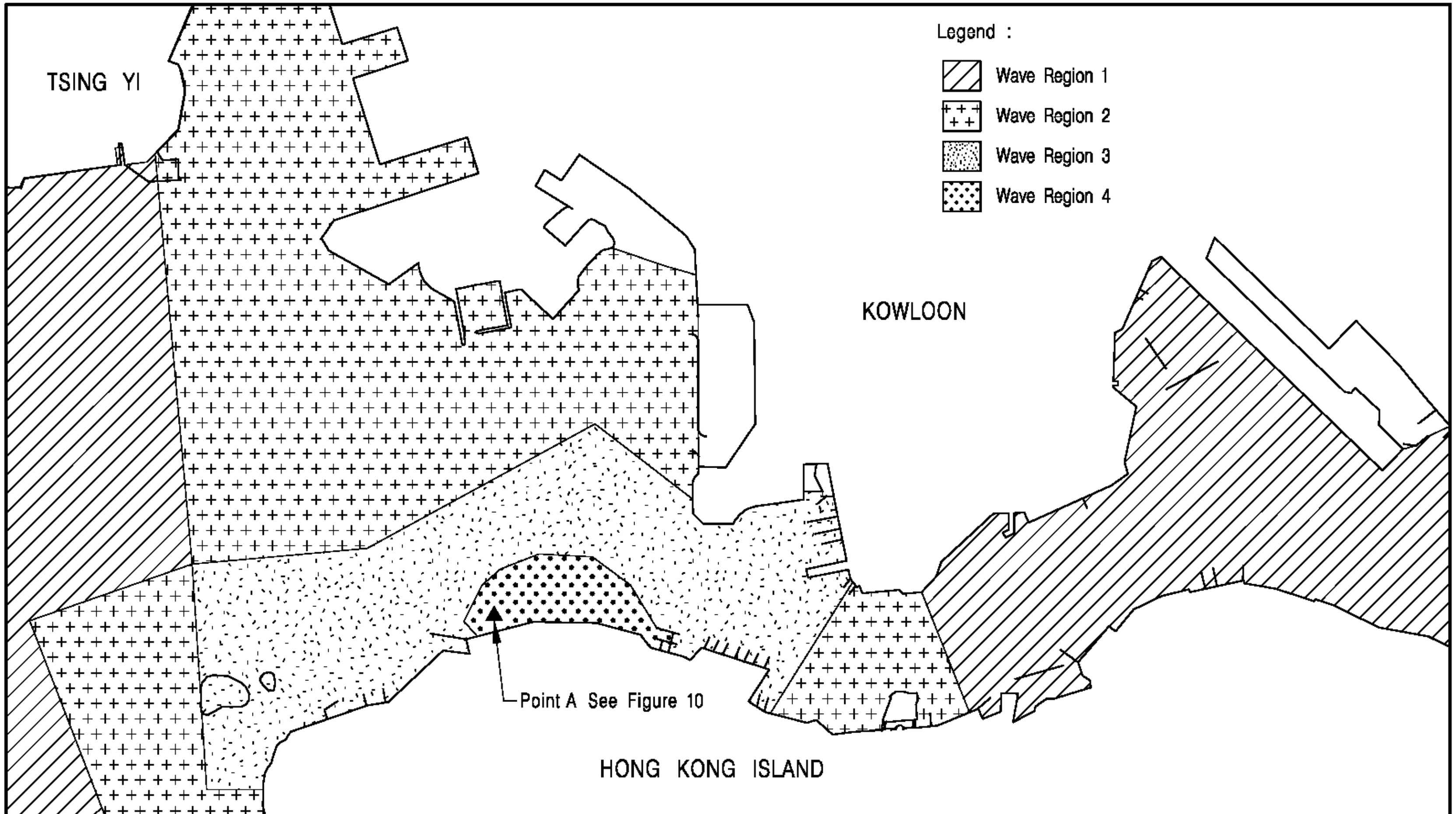
1. Data Collection Period 1994–2000.
2. Wave heights with occurrence percentage less than 0.5% are not shown.

Figure 7 – Wave Roses at Kau Yi Chau and West Lamma Channel Wave Stations



Point	Location	
A	114°9′E	21°47′N
B	114°40′E	21°55′N

Figure 8 – Locations of Offshore Wave Data from Storm Hindcasting



Notes:

1. The coastline is the coastline in 1996.
2. Description of each wave region is given in Table 35.

Figure 9 – Wave Regions in the Harbour Area

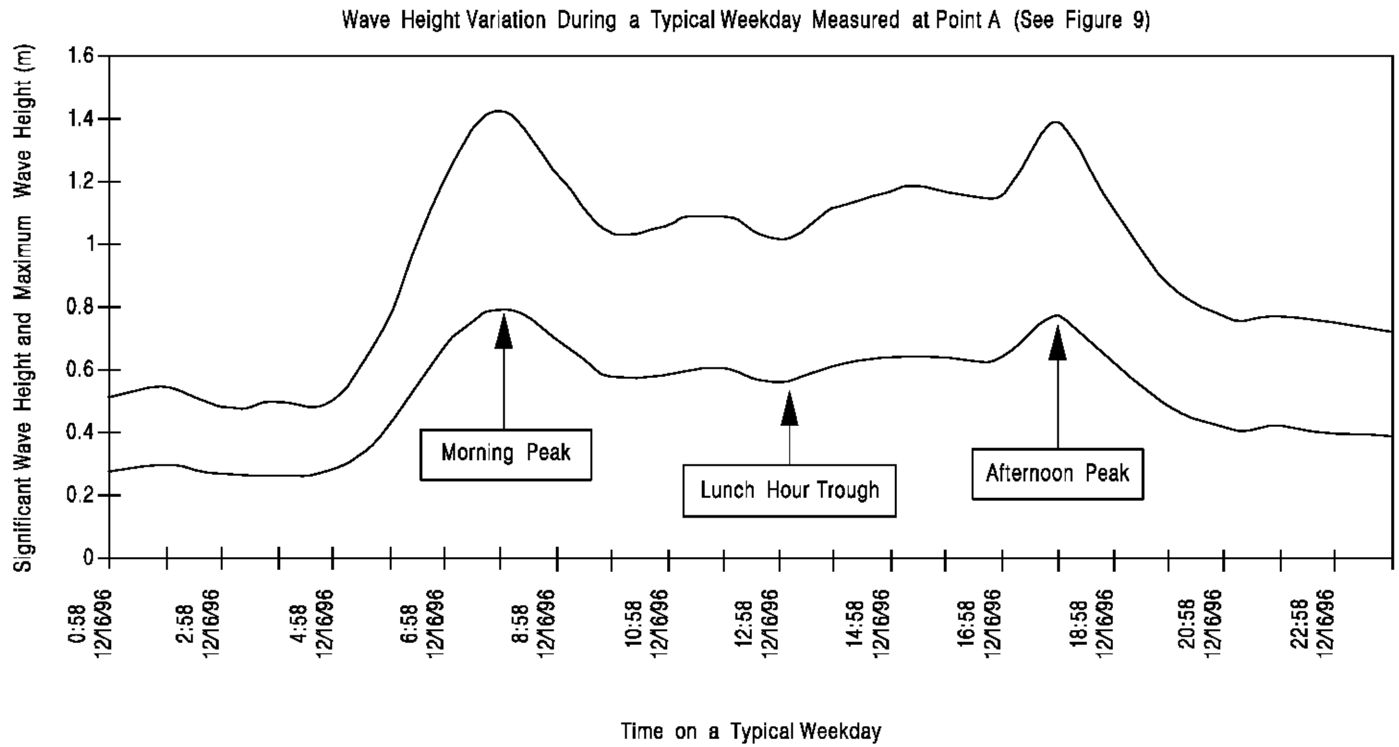
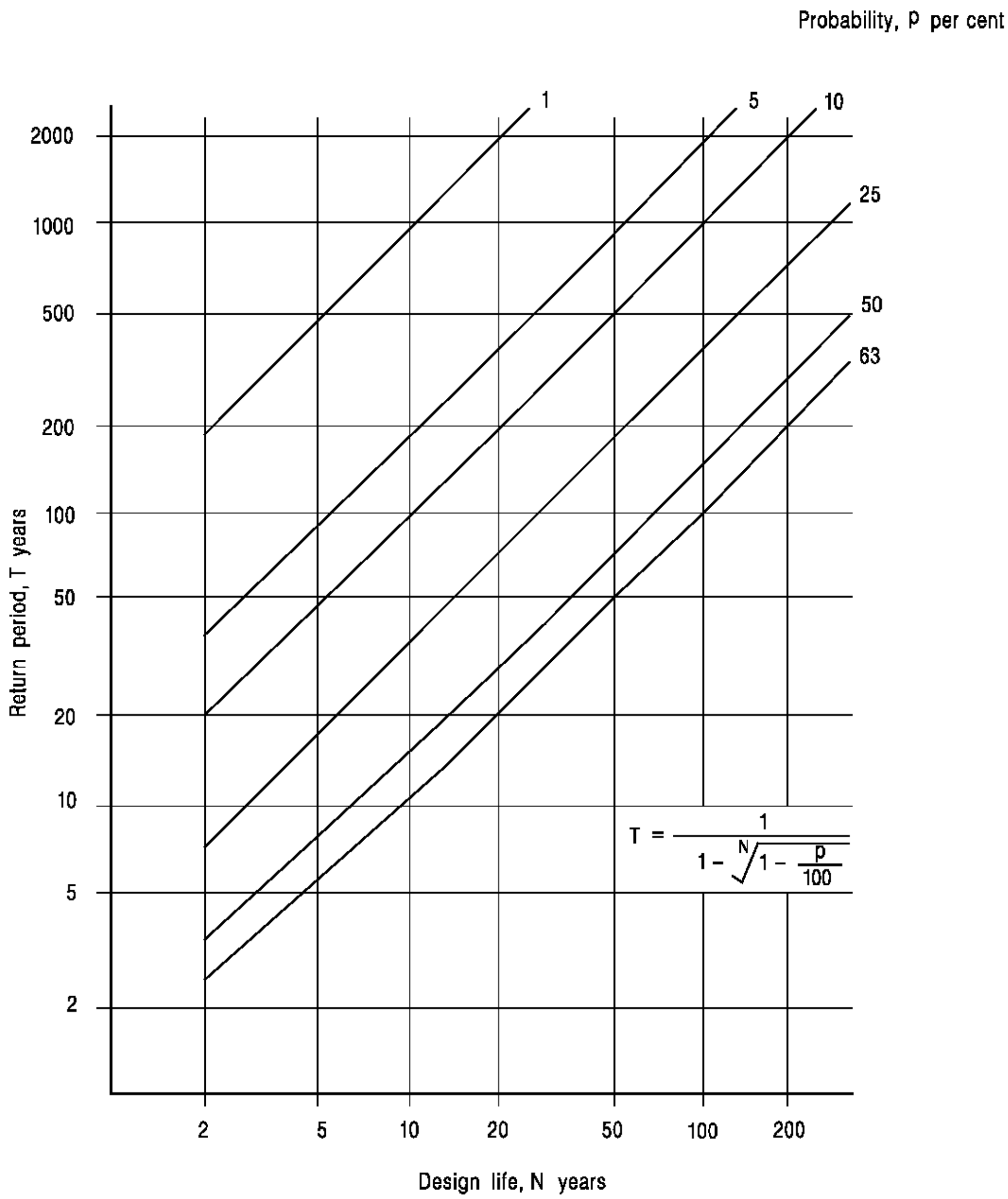


Figure 10 – Daily Wave Variation around a Harbour Wave Station close to Busy Fairways



Legend:

T = return period of a particular extreme wave condition

p = the probability of a particular extreme wave condition occurring during design life N years

Source: BSI (2000)

Figure 11 – Relationship between Design Life, Return Period and Probability of Exceedence

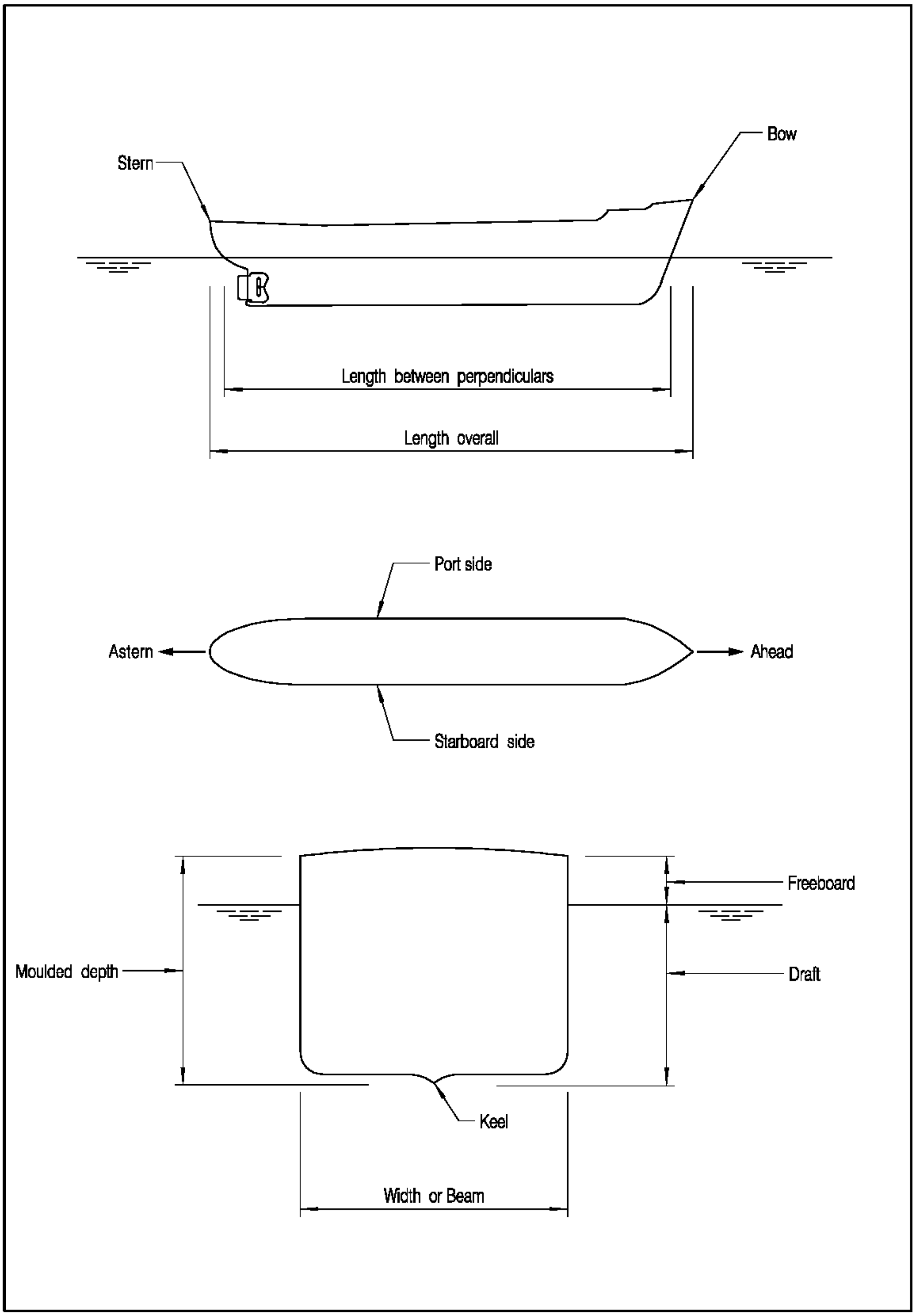
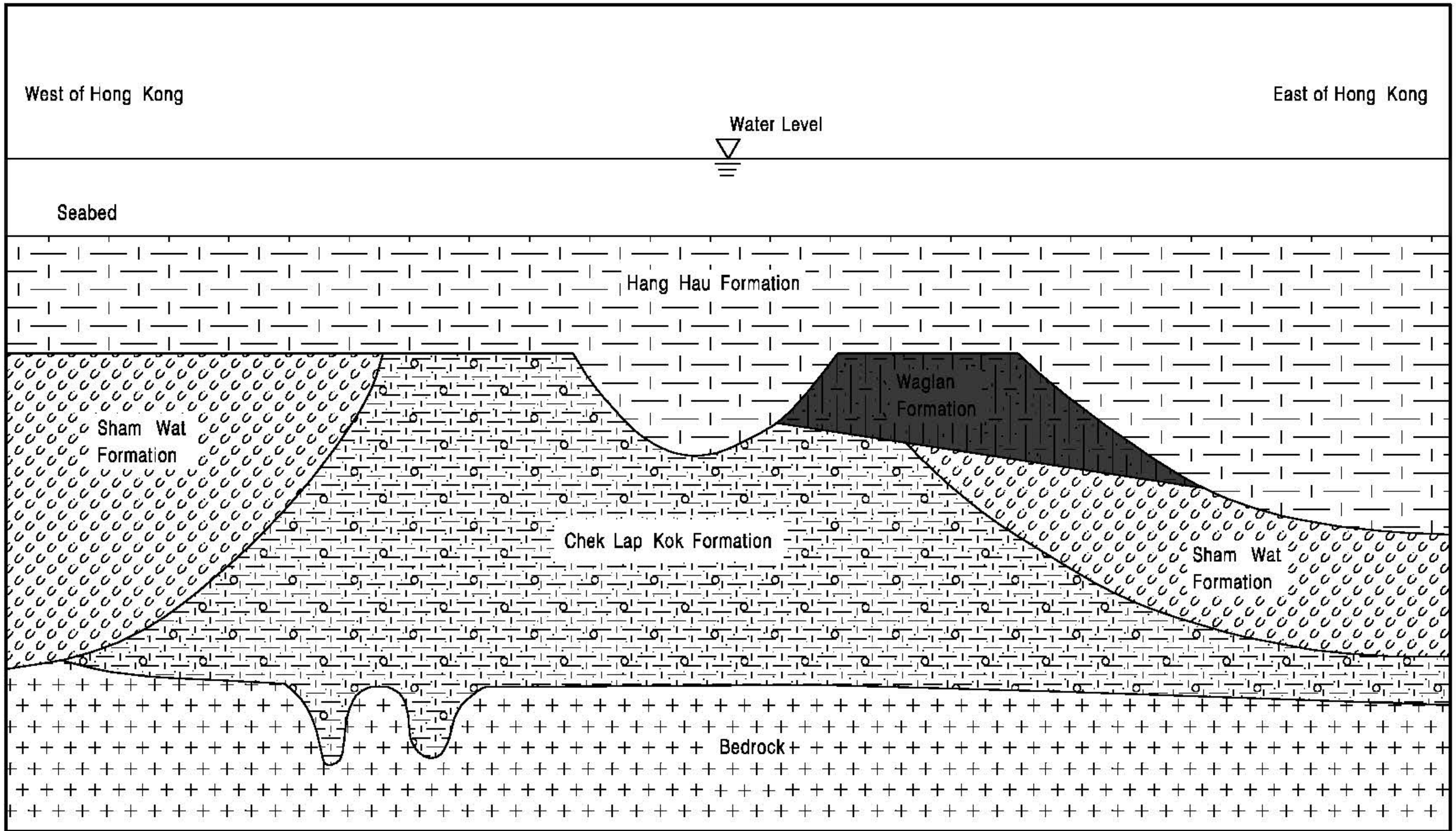


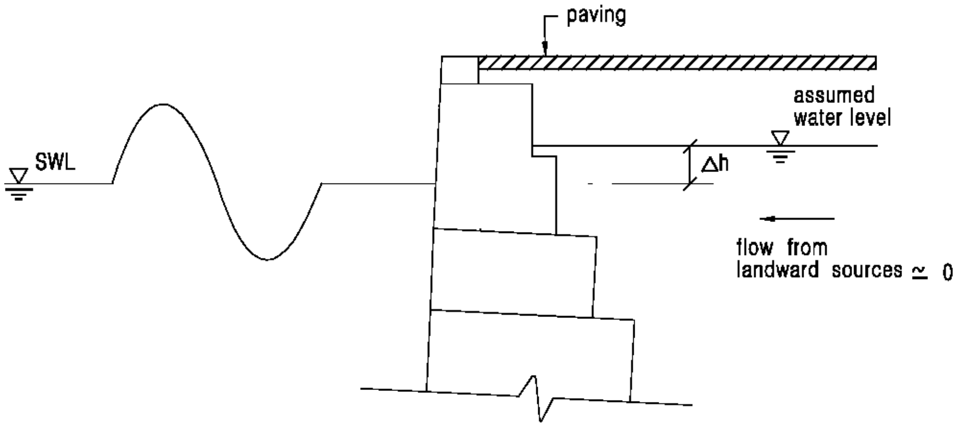
Figure 12 – Ship Definitions



Note:

1. Description of various formations is given in Table 38.

Figure 13 – Schematic Diagram of Offshore Quaternary Stratigraphy of Hong Kong



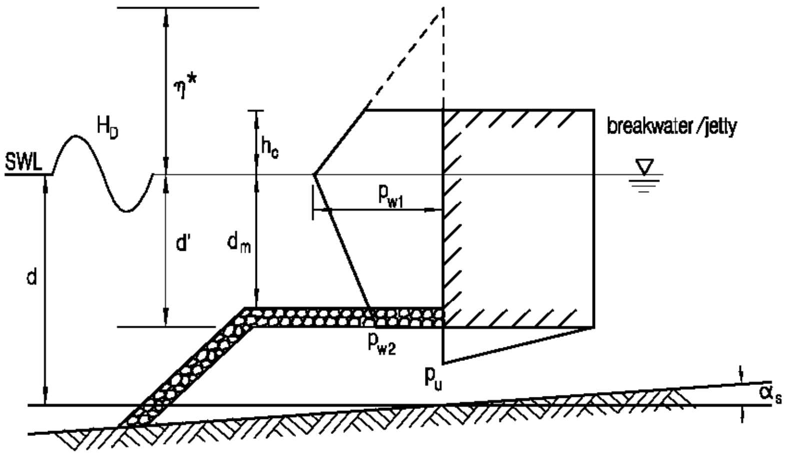
Minimum tidal lag (Δh) to be :

0.7m for Normal Loading Conditions
1.0m for Extreme Loading Conditions

Note:

1. Assumptions - land behind seawall is paved.
- flow from landward sources is negligible.
- adequate surface and back drainage are provided.

Figure 14 – Ground Water Profile behind Seawalls – Area Paved



(hydrostatic pressure and buoyancy not shown)

The elevation to which wave pressure is exerted :

$$\eta^* = 0.75\ (1 + \cos\beta)\ H_D$$

Where β is the nominal angle between the direction of wave approach and a line normal to the structure. When the actual angle between the direction of approach and the normal is 15°or less, $\beta = 0$. When the actual angle exceeds 15°, β =actual angle of 15°

Wave pressure on the front face of the structure (Hydrostatic pressure not included) :

$$p_{w1} = \frac{1}{2}(1 + \cos\beta)(\alpha_1 + \alpha_2 \cos^2\beta)\gamma_w H_D$$

$$p_{w2} = \alpha_3\ p_{w1}$$

Uplift pressure at the toe of the structure (buoyancy not included) :

$$p_u = \frac{1}{2}\alpha_1\ \alpha_3\ (1 + \cos\beta)\ \gamma_w H_D$$

where

$$\alpha_1 = 0.6 + \frac{1}{2}\left(\frac{4\pi d / L}{\sinh(4\pi d / L)}\right)^2$$

α_2 is the lesser of $\frac{1}{3}\left(\frac{d_b - d_m}{d_b}\right)\left(\frac{H_D}{d_m}\right)^2$ or $\alpha_2 = \frac{2 d_m}{H_D}$

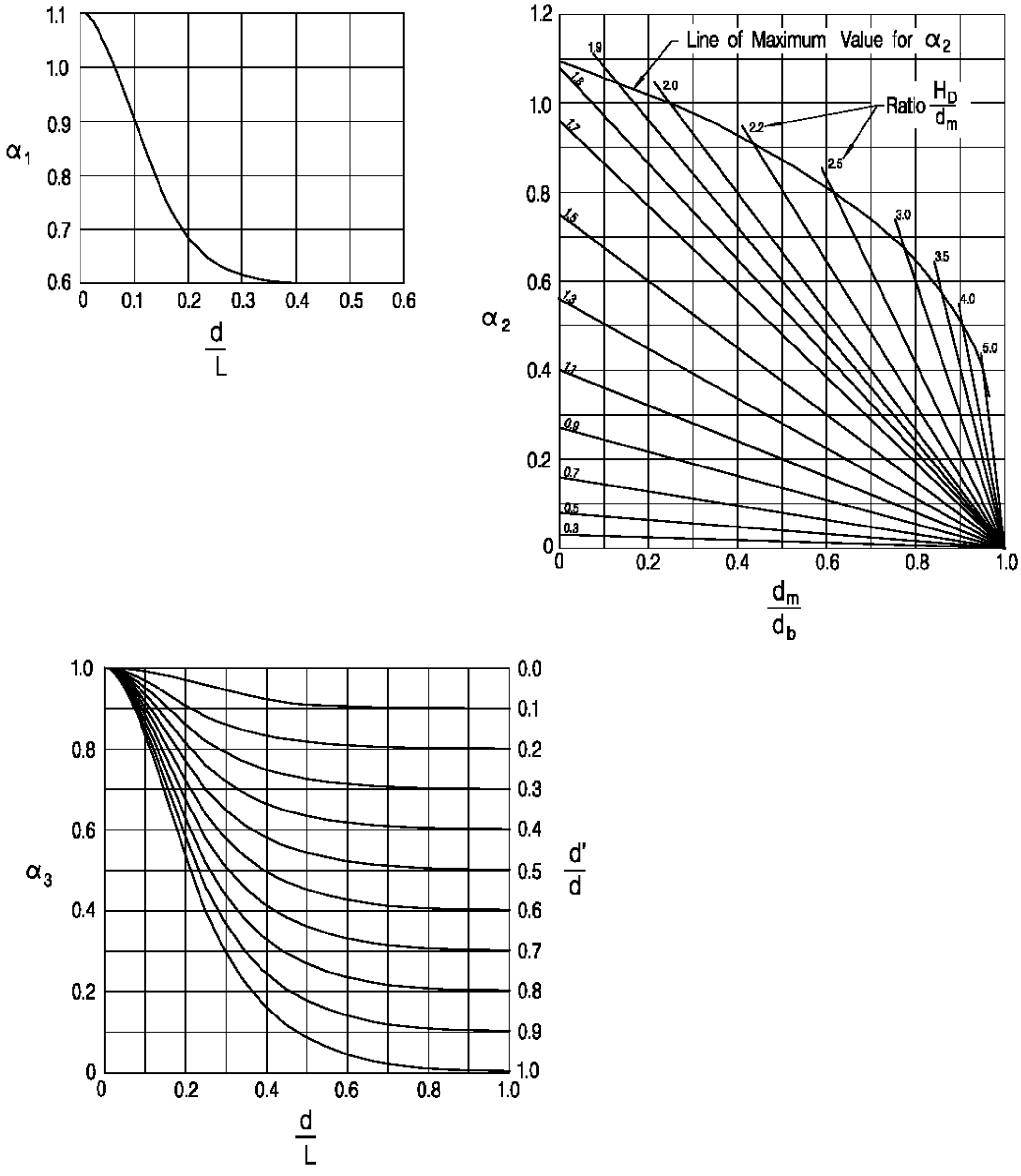
$$\alpha_3 = 1 - \frac{d'}{d}\left(1 - \frac{1}{\cosh(2\pi d / L)}\right)$$

Note:

1. α_1, α_2 and α_3 may also be estimated from design curves in Figure 16

Source: BSI (2000)

Figure 15 – Maximum Wave Pressure on Vertical Structures (Breaking and Non-breaking Waves) – Pressure Distribution



Legend:

L = wavelength of design wave in depth d corresponding to T_s

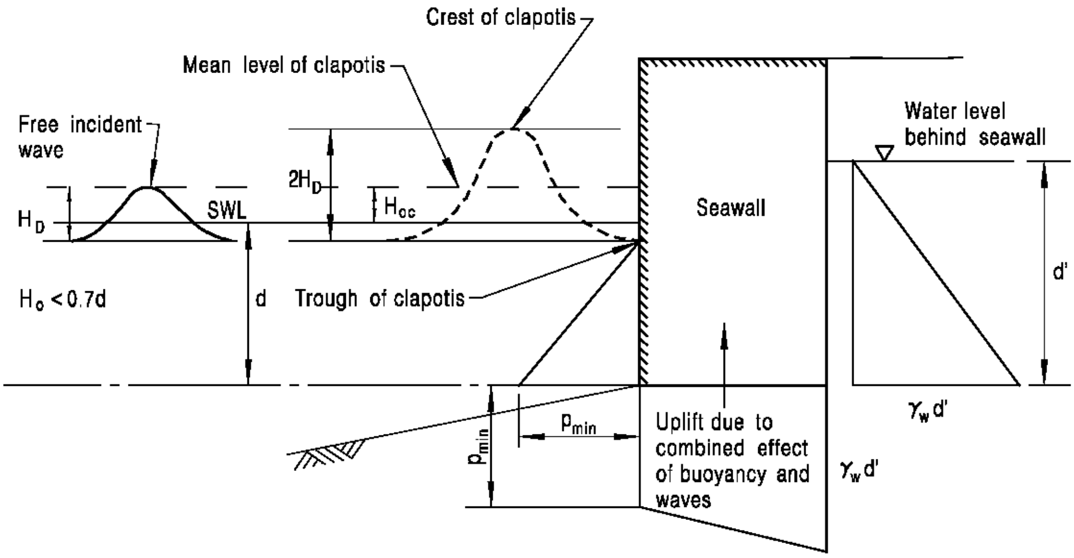
H_D = design wave height

Note:

1. See Figure 15 for d, d', d_m and d_b

Source : BSI (2000)

Figure 16 – Maximum Wave Pressure on Vertical Structures (Breaking and Non–breaking Waves) – Alpha Values



Pressure at the toe of the structure (Effect of hydrostatic pressure included) :

$p_{min} = \gamma_w d - \gamma_w H_D / \cosh(2\pi d/L)$ under trough condition

Height of clapotis ≃ 2 times height of free incident wave

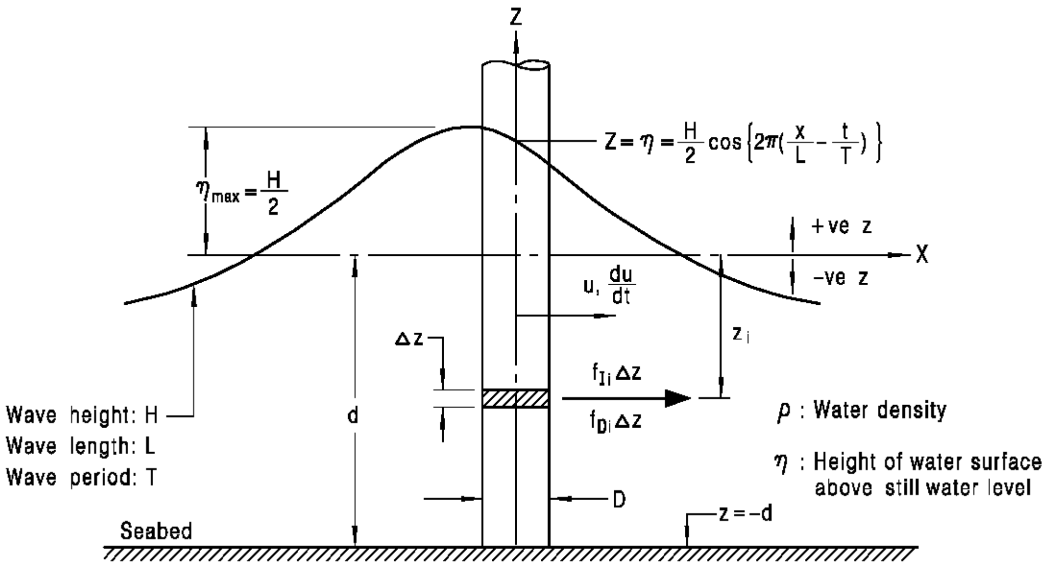
Clapotis set-up, $H_{oc} = \dfrac{\pi H_D{}^2}{L} \coth(2\pi d/L)$

Notes:

1. The wave pressure distribution on the wall, based on the theory of Sainflou, is assumed to be linear with depth.
2. The figure of 0.7 might not be correct for steep wave conditions, steeply sloping seabeds and composite structures.

Source: BSI (2000)

Figure 17 – Wave Pressure under Wave Trough



$$f_{Ii\,max} = C_I \frac{\rho g \pi D^2}{4} H \left\{ \frac{\pi}{L} \frac{\cosh\,[2\pi(z+d)/L]}{\cosh\,[2\pi d/L]} \right\}$$

$$f_{Di\,max} = C_D \frac{\rho g D H^2}{2} \left\{ \frac{gT^2}{4L^2} \left[\frac{\cosh\,[2\pi(z+d)/L]}{\cosh\,[2\pi d/L]} \right]^2 \right\}$$

Where f_{Ii} = inertia force per unit length at depth z_i ; 'max' denotes its maximum value.
f_{Di} = drag force per unit length at depth z_i ; 'max' denotes its maximum value.
C_I = inertia coefficient
C_D = drag coefficient
D = pile diameter
g = acceleration due to gravity

For $D/W_P > 0.2$, inertia force is dominant:

Wave Force on Pile = 1.4 x [Summation of $f_{Ii\,max}$ from water surface to seabed]
= $1.4 \times \sum_i f_{Ii\,max} \Delta z$

For $D/W_P < 0.2$, drag force is dominant:

Wave Force on Pile = 1.4 x [Summation of $f_{Di\,max}$ from water surface to seabed]
= $1.4 \times \sum_i f_{Di\,max} \Delta z$

and W_P = orbit width of water particles at the surface
$= \frac{H}{\tanh(\frac{2\pi d}{L})}$

Notes:

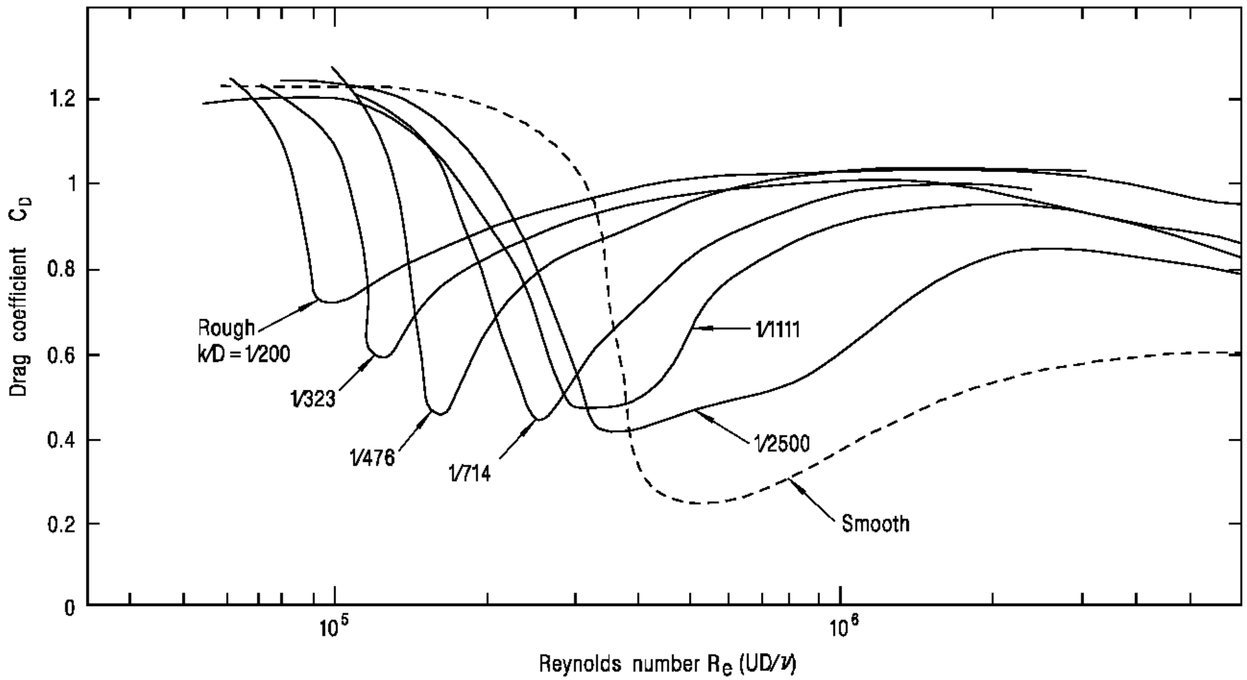
1. For inclined piles, the maximum force per unit length perpendicular to the actual inclined pile is taken to the horizontal force per unit length of a fictitious vertical pile at the same location.

2. The pile diameter should include an allowance for wave growth.

3. The above expression is based on linear wave (Airy) theory.

Source: BSI (2000)

Figure 18 – Wave Forces on Piles

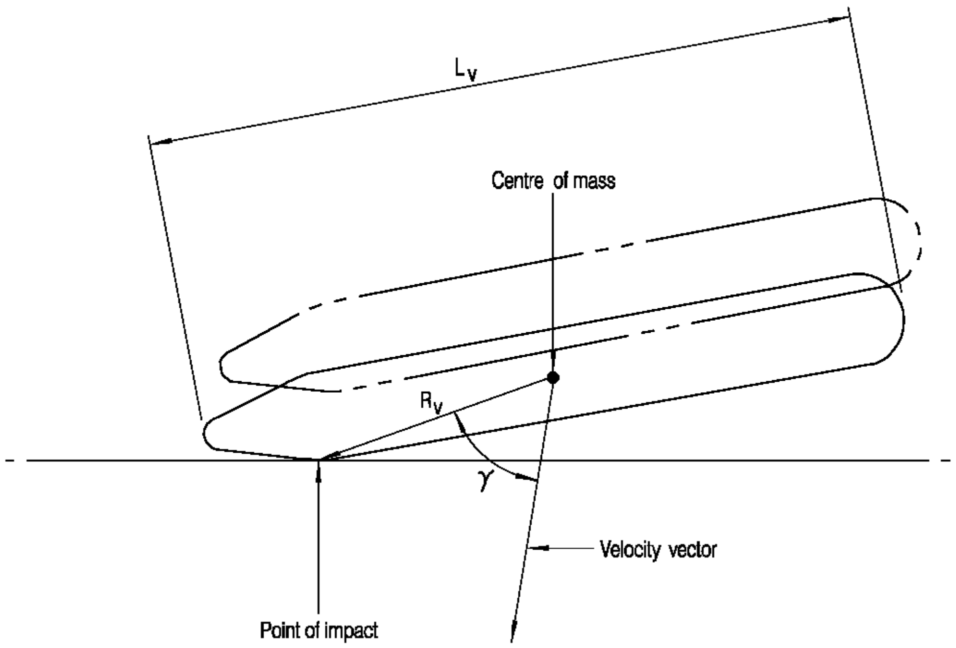


Legend:

D = Diamater of cylinder/pile
U = Velocity normal to member
ν = Kinematic viscosity of water
k = Roughness

Source : BSI(2000)

Figure 19 – Drag Coefficient Values for Circular Cylinders



Legend:

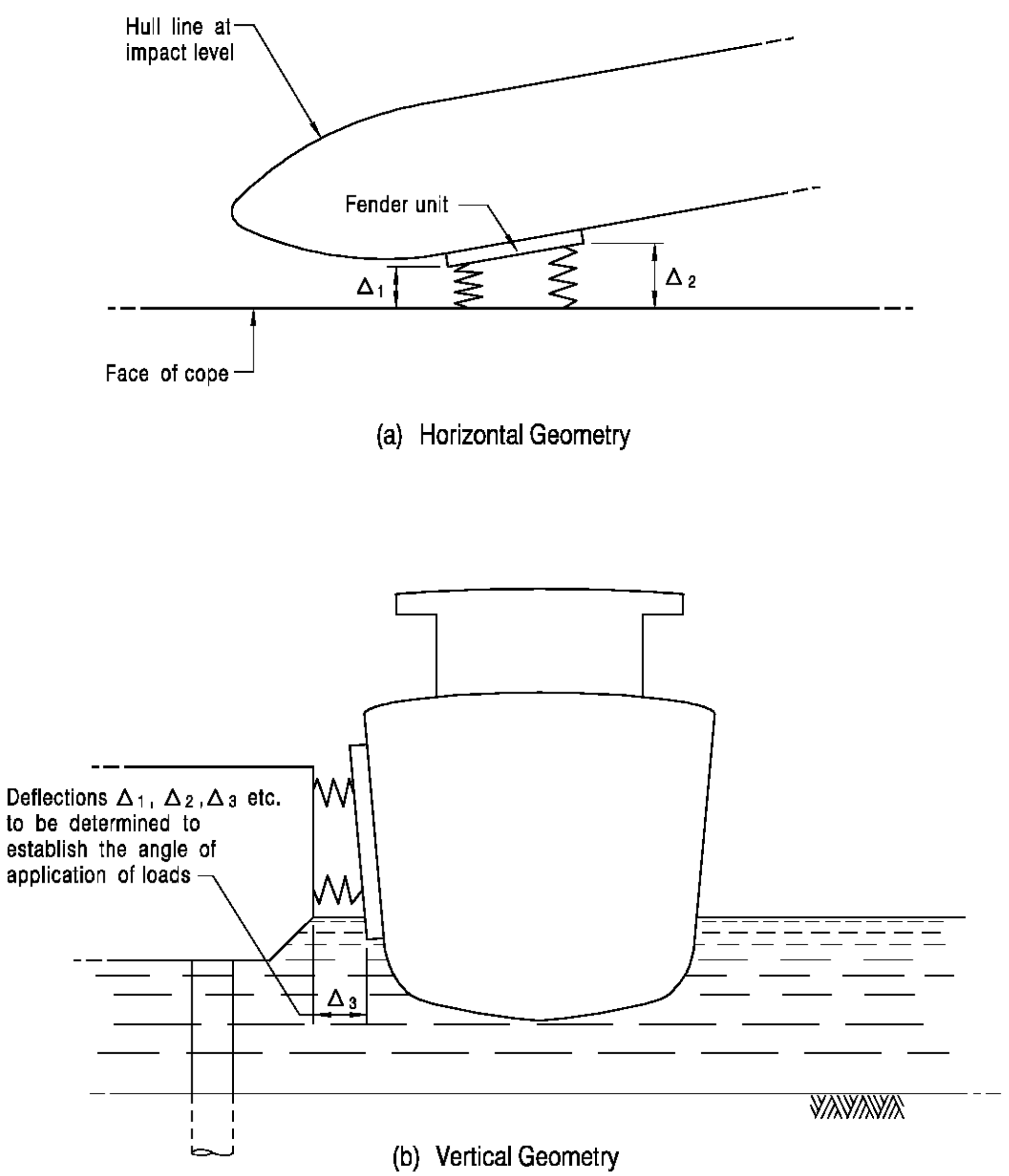
R_V = distance between the point of contact from the centre of mass

γ = angle between the line joining the point of contact to the centre to mass and the velocity vector

L_V = length of the hull between perpendiculars

Source : BSI (1994b)

Figure 20 – Geometry of Vessel Approach to Berth



Note:

1. Δ_1, Δ_2, Δ_3 are deflections of the fender unit.

Source: BSI (1994b)

Figure 21 – Hull and Fender Geometry at Impact