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1.

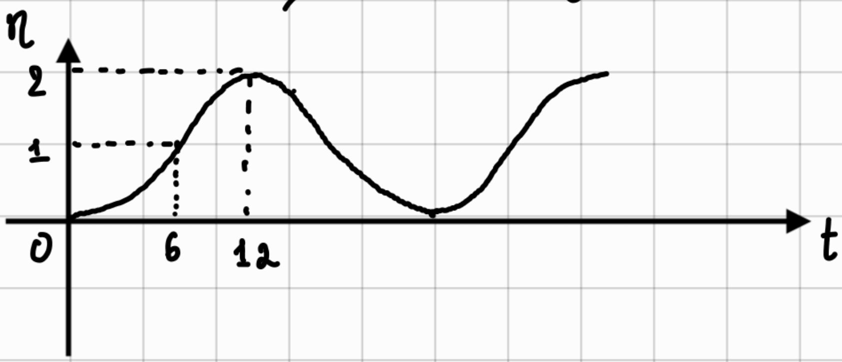
(a) $\eta = 1.0 + \cos(2\pi t/24 + \pi)$

(i) MSL = 1.0 m

$\eta = 1.0 \Rightarrow 1.0 = 1.0 + \cos(2\pi t/24 + \pi)$

$\Rightarrow \cos(2\pi t/24 + \pi) = 0$

$\Rightarrow t = 6$ hours



$\eta = 1.0 + \cos(2\pi t/24 + \pi)$

$\eta = 1.0 - \cos(\pi t/12)$

$t = \frac{12}{\pi} \operatorname{acos}(1-\eta)$

Within a period of 12 hours, for any point (η_p, t_p)

$$P(0 < \eta < \eta_p) = \frac{t_p}{12}$$

$\Rightarrow P(0 < \eta < 1) = \frac{6}{12} = 0.5$

$\Rightarrow P(\eta > 1) = 1 - P(0 < \eta < 1) = 0.5$

(ii) $-1 \leq \cos(\pi t/12) \leq 1$

$\Rightarrow 1 - 1 \leq 1 - \cos(\pi t/12) \leq 1 - (-1)$

$\Rightarrow 0 \leq \eta \leq 2$

$\therefore P(\eta > 0) = 1$

$P(\eta > 2) = 0$

(iii) $\eta = 0.35 \Rightarrow t = \frac{12}{\pi} a\cos(1-0.35) = 3.30$ Hoang Thu Minh

$$P(\eta > 0.35) = 1 - P(0 < \eta < 0.35)$$
$$= 1 - \frac{3.30}{12}$$
$$= 0.725$$

$\eta = 1.75 \Rightarrow t = \frac{12}{\pi} a\cos(1-1.75) = 9.24$

$$P(\eta > 1.75) = 1 - P(0 < \eta < 1.75)$$
$$= 1 - \frac{9.24}{12}$$
$$= 0.230$$

(iv) $P(0 < \eta < n_p) = \frac{t_p}{12}$ for any point (n_p, t_p) within 12 hrs

$$\Rightarrow P(0 < \eta < n_p + \Delta n_p) = \frac{t_p + \Delta t_p}{12}$$

$$\Rightarrow P(0 < \eta < n_p + \Delta n_p) - P(0 < \eta < n_p) = \frac{t_p + \Delta t_p}{12} - \frac{t_p}{12}$$
$$= \frac{\Delta t_p}{12}$$

$$p(\eta_p) = \frac{P(0 < \eta < n_p + \Delta \eta_p) - P(0 < \eta < n_p)}{\Delta \eta_p}$$

$$= \frac{1}{12} \left. \frac{dt}{d\eta} \right|_{\eta = \eta_p} = \frac{1}{12} \times \frac{12}{\pi} \times \frac{1}{\sqrt{1 - \eta_p^2}}$$

$$= \frac{1}{\pi \sqrt{1 - \eta_p^2}}$$

(b) $U_{10} = 20$ m/s , $D = 5$ hours

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(i) Fetch is very long $\Rightarrow$ Duration - limited wave

From CEM Fig II-2-25 : $H = 6.5$m

From CEM Fig II-2-26 : $T = 3$s

(ii) Maximum Water elevation due to tidal component when the maximum surge occurs $= 2.2 - 0.3 = 1.9$ m

$\Rightarrow \quad 1.0 + \cos\left(\frac{2\pi t}{24} + \pi\right) = 1.9$

$\Rightarrow \quad -\cos\left(\pi t / 12\right) = 0.9$

$\Rightarrow \quad t = 10.3$ or $t = 13.7$

Possible time interval when maximum surge occurred is $[0, 10.3]$ or $[13.7, 24]$

2.

(a) $T = 10$s , $H_o = 2.1$ m , $\rho_w = 1025$ kg/m^3

(i). Deep water : $\frac{d}{L} > \frac{1}{2} \Rightarrow d > \frac{1}{2} L_o$

$$L_o = \frac{gT^2}{2\pi} = \frac{9.81 \times 10^2}{2\pi} = 156\text{m}$$

$\Rightarrow d_1 = 156 \times \frac{1}{2} = 78$ m

. Shallow water : $\frac{d}{L} < \frac{1}{20} \Rightarrow d < \frac{1}{20} L$

$L = T\sqrt{gd} \Rightarrow d < \frac{1}{20} T\sqrt{gd}$

$\Rightarrow d < \frac{1}{20^2} T^2 g$

$\Rightarrow d_2 = \frac{1}{20^2} \times 10^2 \times 9.81 = 2.45$ m

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(ii) $K_s = \sqrt{\frac{C_{go}}{C_g}}$

$$C_{go} = \frac{gT}{4\pi} = \frac{9.81 \times 10}{4\pi} = 7.81 \text{ m/s}$$

At $d_1 = 78$m (deep water) : $K_s = \sqrt{\frac{C_{go}}{C_{go}}} = 1$

At $d_2 = 2.45$m (shallow water) :

$$C_g = \sqrt{gd} = \sqrt{9.81 \times 2.45} = 4.90 \text{ m/s}$$

$$\bar{K_s} = \sqrt{\frac{C_{go}}{C_g}} = \sqrt{\frac{7.81}{4.90}} = 1.26$$

$\therefore$ Range of shoaling coefficient is from 1 to 1.26

(iii) • At $d_1 = 78$m (deep water): $H = H_o = 2.1$m

$$u_{max} = \frac{\pi H}{T} e^{\frac{2\pi(-d)}{L}} = \frac{\pi \times 2.1}{T} e^{\frac{2\pi \times (-78)}{156}} = 0.0285 \text{ m/s}$$

$$P_{max} = \rho g \frac{H}{2} e^{\frac{2\pi(-d)}{L}} - \rho g(-d)$$

$$= 1025 \times 9.81 \times \frac{2.1}{2} \times e^{\frac{2\pi(-78)}{156}} - 1025 \times 9.81 \times (-78)$$

$$= 785 \text{ kPa}$$

• At $d_2 = 2.45$m (shallow water): $H = K_s H_o = 1.26 \times 2.1 = 2.65$m

$$u_{max} = \frac{H}{2}\sqrt{\frac{g}{d}} = \frac{2.65}{2}\sqrt{\frac{9.81}{2.45}} = 2.65 \text{ m/s}$$

$$P_{max} = \rho g \left(\frac{H}{2} - (-d)\right) = 9.81 \times 1025 \left(\frac{2.65}{2} + 2.45\right)$$

$$= 38.0 \text{ kPa}$$

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(iv) At $d_1 = 78m$: $\frac{H}{d} = \frac{2.1}{78} < 0.78 \Rightarrow$ non-breaking

At $d_2 = 2.45m$: $\frac{H}{d} = \frac{2.65}{2.45} = 1.08 > 0.78 \Rightarrow$ breaking

As the wave propagates towards the shore, wave height increases. Hence, wave breaks before it reaches location d_2

(b) As the wave approaches at an oblique angle, the refraction effect happens and reduces the wave height.

Since the wave period does not change, the wave length will not change and thus the transitional range will not change.

Since wave height is reduced further due to refraction, the possibility of wave breaking is reduced.

Check for breaking at $d_2 = 2.45m$:

From Fig II-3-6 : $\frac{d}{gT^2} = 0.0025$, $\theta_0 = 60° \Rightarrow K_R K_S = 0.87$

$\Rightarrow H = K_R K_S H_0 = 0.87 \times 2.1 = 1.83\ m$

$\frac{H}{d} = \frac{1.83}{2.45} = 0.75 < 0.78 \Rightarrow$ non-breaking

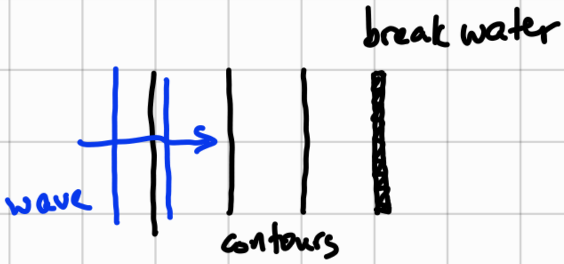
3.

$d = 2.5m$

slope 1:50

$H_0 = 1.7m$

$T = 9s$



(a) $L_0 = \frac{gT^2}{2\pi} = \frac{9.81 \times 9^2}{2\pi} = 126.5\ m$

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Wave propagates at normal incidence towards the breakwater
$\Rightarrow$ Wave crest is parallel to bottom contours
$\Rightarrow$ No refraction effect
$\Rightarrow H_0' = H_0 = 1.7\,m$

Breaker height index $\Omega_b = \dfrac{H_b}{H_0} = 0.56\left(\dfrac{H_0'}{L_0}\right)^{-1/5}$

$$\Omega_b = 0.56\left(\frac{1.7}{126.5}\right)^{-1/5} = 1.33$$

$$\Rightarrow H_b = 1.33\,H_0 = 1.33 \times 1.7 = 2.25\,m$$

From Fig II-4-2 : $\left.\begin{array}{l}\dfrac{H_b}{gT^2} = \dfrac{2.25}{9.81 \times 9^2} = 0.0028 \\ \tan\beta = 0.02\end{array}\right\} \Rightarrow \dfrac{H_b}{d_b} = 0.89$

$\Rightarrow d_b = \dfrac{2.25}{0.89} = 2.52\,m$ = design water depth

(b) Calm water on lee side $\Rightarrow$ hydrostatic forces cancel
$\Rightarrow$ Net force = Dynamic force by wave breaking

From SPM Fig 7-100 : $\left.\begin{array}{l}\dfrac{d_s}{gT^2} = \dfrac{2.5}{9.81 \times 9^2} = 0.003 \\ m = 0.02\end{array}\right\} \Rightarrow \dfrac{3R_m}{wH_b^2} = 8.5$

$$\Rightarrow R_m = \frac{1}{3} \times 1025 \times 9.81 \times 2.25^2 \times 8.5 = 144.2\ kN/m$$

$\therefore$ Maximum total force = 144.2 kN/m width on wall

(c) $\xi_0 = \tan\beta\left(\dfrac{H_0}{L_0}\right)^{-1/2} = 0.02\left(\dfrac{1.7}{126.5}\right)^{-1/2} = 0.173 < 0.5$

$\Rightarrow$ Breaking type is spilling : Wave crest becomes unstable and cascades down shoreward face. This breaking type is mild.

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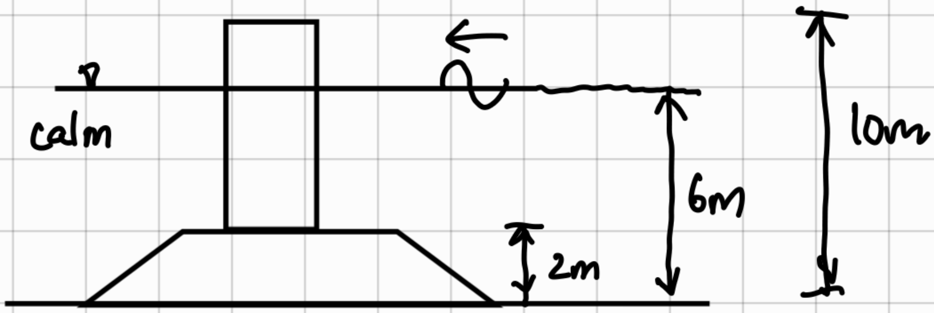
(d) Minikin's method is not suitable in this case because this method tries to account for peak pressures (air pockets) and wave slamming force (plunging wave break). The design thus will be very conservative and costly.

Goda's method would be more suitable because it caters for slower breaking (eg. spilling) process.

4.

(a)



$H_s = 1.5m$

$T = 7s$

Caisson wall is a rigid structure $\Rightarrow$ Use $H_{1/100}$ for design

$\Rightarrow H = 1.67 H_s = 1.67 \times 1.5 = 2.51$ m

Check for breaking: $\frac{H}{d} = \frac{2.51}{6} = 0.42 < 0.78 \Rightarrow$ non-breaking

From SPM Fig 7-90 : $\left.\begin{array}{l} \frac{H}{gT^2} = \frac{2.51}{9.81 \times 7^2} = 0.005 \\ \frac{H}{d} = 0.42 \end{array}\right\} \Rightarrow \frac{h_o}{H_i} = 0.51$

$\Rightarrow h_o = 0.51 \times 2.51 = 1.28$ m

$y_c = h_o + d + \frac{H}{2} = 1.28 + 6 + \frac{2.51}{2} = 8.54$ m < 10m

$\Rightarrow$ no overtopping

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Assume no rubble foundation:

From SPM Fig 7-91 : $\left.\begin{array}{l}\dfrac{H}{gT^2} = 0.005 \\ \dfrac{H}{d} = 0.42\end{array}\right\} \Rightarrow \left\{\begin{array}{l} F_c/wd^2 = 0.50 \\ F_t/wd^2 = -0.30\end{array}\right.$

Calm water on lee side $\Rightarrow$ hydrostatic forces cancel

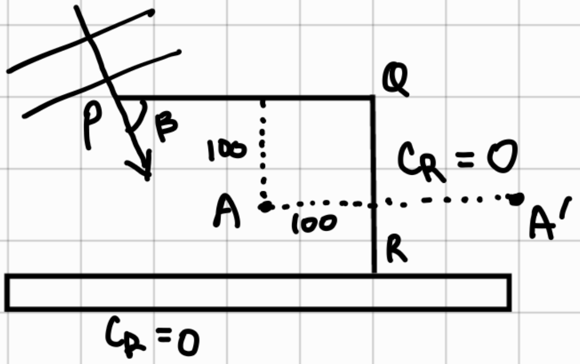
$$\Rightarrow F_M = F_c = 0.50\,wd^2$$
$$= 0.50 \times 1025 \times 9.81 \times 6^2$$
$$= 181 \text{ kN}/m$$

Reduction due to rubble foundation:

SPM Fig 7-97 : $b/y = \dfrac{2}{8.54} = 0.23 \Rightarrow 1 - r_f = 0.59$

$$F'' = (1-r_f)F_M = 0.59 \times 181 = 106 \text{ kN}$$

(b)



$\beta = 60°$
$H = 1m$
$H_A = 0.2m$
$L = 50m$

(i) $K_A = \dfrac{H_A}{H} = 0.2$

Distance from A to breakwater $= 100$ m $= 2L$

From SPM Fig 2-31 $\Rightarrow \widehat{APQ} = 28°$

$\Rightarrow PQ = 100\cot 28° + 100 = 288$ m

(ii) $H_A = K_A H + K_{A'} H C_R$

Increase in wave height $= K_{A'} H C_R$

$R_{A'} = 288 + 100 = 388\,m \Rightarrow R_A/L = 7.76$ and $\widehat{A'PQ} = 14°$

From SPM Fig 2-31 $\Rightarrow K_{A'} = 0.127$

Increase $= 0.127 \times 1 \times 0.3 = 0.038$ m

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(iii) Predominant wave direction would be towards arm PQ of the breakwater because the breakwater would protect the harbour from the most severe wave attack.