UNIVERSITY OF RUHUNA

Faculty of Engineering

End-Semester 7 Examination in Engineering: July 2016

Module Number: CE 7251 **Module Name: Coastal Engineering Applications and Management**

[Three Hours]

[Answer **all questions**, each question carries TWELVE marks]

Q1 (a) Explain the process of how storm surges and tsunami waves are generated, propagate and flood the land ? [2 marks]

(b) What are the coastal defence policy options in a shoreline management plan (SMP)?

[4 marks]

(c) Choose appropriate coastal defence policy options for the following areas where there is severe coastal erosion ?. Explain the reasons behind your choice.

(i) Dikkowita (i.e. immediately north of Colombo Port)
(ii) An uninhabited coastal area in Putlam

[4 marks]

(d) (i) Define the Coastal Zone of Sri Lanka (ii) Describe the statutory functions of the Department of Coast & Coastal Resources Conservation.

[2 marks]

Q2 (a) (i) What are the assumptions made in deriving linear wave theory ?. (ii) Explain why linear wave theory is not valid in shallow water depths.

[3 marks]

(b) Explain how wave induced drag and inertia forces on a vertical mono-pile driven to the seabed are generated using principles of physics.

[3 marks]

(c) A cylindrical mono-pile with diameter D=3m is to be driven into a sandy sea bed in water depth h=10m to construct a jetty in a harbour. The design wave height, H=2.5m and wave period is T=8 sec.

Linearised form of the velocity potential of a surface gravity wave is given by:

$$\phi = \frac{gH}{2\omega}\frac{\cosh k(z+h)}{\cosh(kh)}\sin(kx-\omega t)$$

(i) Derive horizontal orbital velocity(U) and acceleration (a_x) at the mid-depth, i.e., z = - 0.5h for (kx-ωt) = π/3.

[3 marks]

(ii) Calculate drag and inertia forces on the pile for (kx-ωt) = π/3 and estimate the total force. Assume velocity and acceleration at the mid depth are applicable to the full submerged part of the pile.

[3 marks]

Drag force $= \frac{1}{2}\rho C_D A U^2$
Inertia force $= \rho C_M \forall a_x$

Where, A= cross sectional area = D x h and the displaced volume of water, $\forall = \frac{\pi D^2}{4} h$, C_D=0.7, C_M=2.0, density of water =1030 kg/m³.

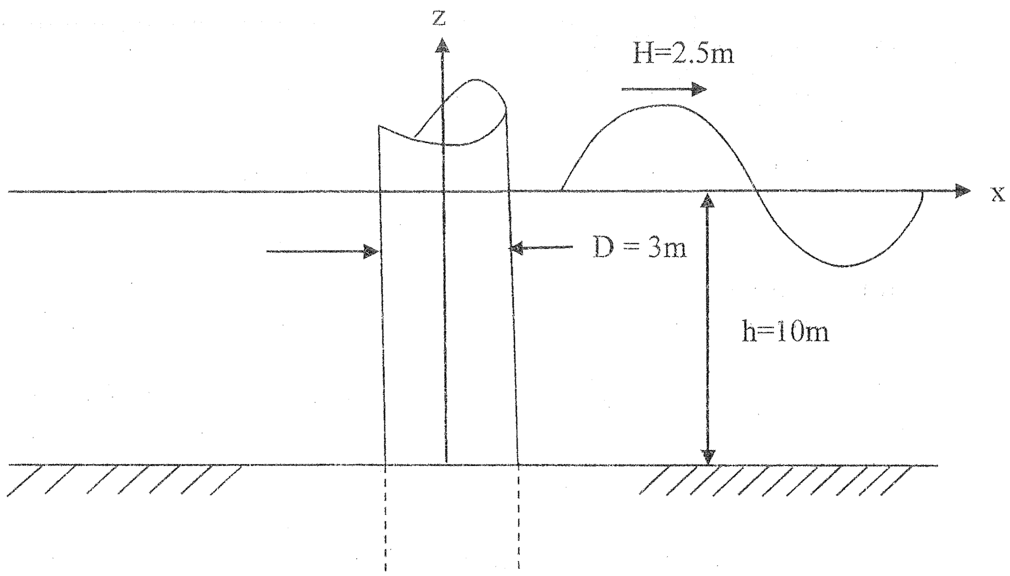

Figure Q2. Mono-pile

Q3 It is proposed to construct a vertical quay wall in a harbour for berthing of vessels using concrete caisson boxes (refer to Figure Q3). Height of a Caisson box is 2.2m. The width of the box is 1.1m. Assume the length of the caisson is 1m. Incoming wave height, Hinc=1m and wave period , T=8 seconds, density of sea water ρ= 1030kg/m³. Use Wave Table given in the Appendix.

(a) What is the height of the reflected wave (Hr) ?

[1 mark]

(b) Calculate the hydrostatic pressure and peak dynamic wave pressure (Pw) at the sea bed ? Calculate P max= (hydrostatic pressure + Pw).

$$P_w = \frac{\rho g H_r}{2\cosh(\frac{2\pi h}{L})}$$

[5 marks]

(c) Calculate the total pressure P0 (i.e., hydrostatic + dynamic wave pressure) at the base of the Caisson - A. Assume $Hoc = \frac{\pi H_{inc}^2}{L}\coth\frac{2\pi h}{L}$

[3 marks]

(d) Calculate the total hydraulic force per metre length on the seaward face of the Caisson- A.

[1 mark]

(e) Assuming the length of the Caisson is 1m, calculate the restoring moment due to gravity and hence, determine the wall thickness (t) of the Caisson box to counter balance the overturning moment due to hydraulic forces. Density of concrete is 2400 kg/m³ and density of sea water = 1030 kg/m³. Neglect the weight of sand inside the box and soil/water pressure from the landward side.

[2 marks]

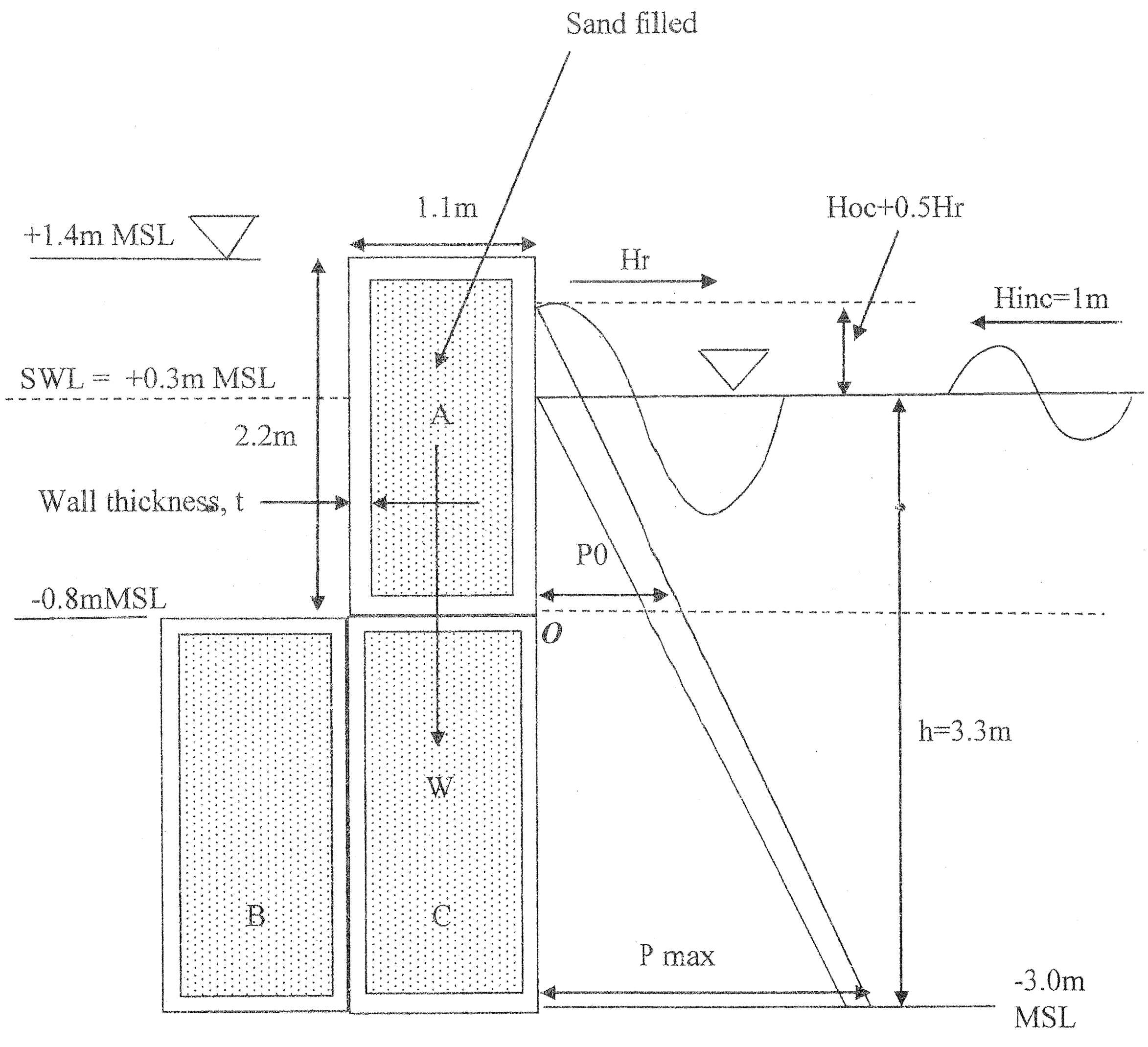

Figure Q3. Caisson Quay Wall Structure

Q4. (a) In selecting concrete armour units for a breakwater construction project what features/functions of the armour unit would you consider as important ?. Give the name/s of armour unit/s currently used in the industry fitting the required features.

[4 marks]

(b) Propose a solution to reduce (i) toe scour (ii) overtopping along a vertical concrete seawall with the aid of sketches.

[4 mark]

(c) Use Van der Meer equation to calculate median rock armour diameter and weight.

Assuming plunging waves, the equation reads:

$$\frac{H_s}{\Delta D_{n50}} = 6.2S^{0.2}P^{0.18}N_z^{-0.1}\xi_m^{-0.5} \quad \text{Eq. 4.1}$$

Where, S is relative eroded area equal to 2 for initial damage. P is porosity equal to 0.3. Number of waves in a storm N_z=5000. Significant wave height, H_s=2.8m, wave period T=8 sec, $S_m=H_s/L_0$. Breakwater slope is tan α = 1/2. $\Delta=[(\rho_s/\rho)-1]$ where density of rock ρ_s=2650kg/m^3 and density of seawater ρ=1030kg/m^3.

$$\xi_m = \frac{\tan\alpha}{\sqrt{S_m}} \quad \text{Eq. 4.2}$$

[4 marks]

Q5 Assume that you have been tasked to design a rock breakwater consisting of an armour layer, secondary (under-layer) and a core made of quarry run.

(a) Explain why wave run-up level on a sloping concrete seawall could be higher than on a rubble mound seawall having the same slope ?

[2 marks]

(b) Explain the functions of (i) armour layer (ii) secondary/under layer (iii) core

[3 marks]

(c) Sketch the time evolution of beach plan form after the construction of the breakwater. Assume the initial shoreline is a straight line.

[2 marks]

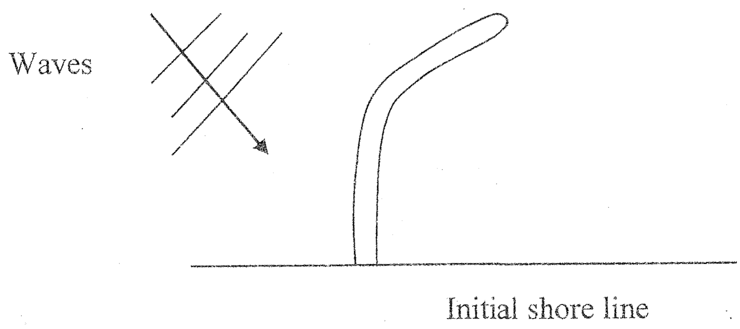

Figure Q5a.

(d) Determine the crest level of the breakwater for 2% exceedance probability providing a 0.6m freeboard for settlement and water level/wave exceedance.

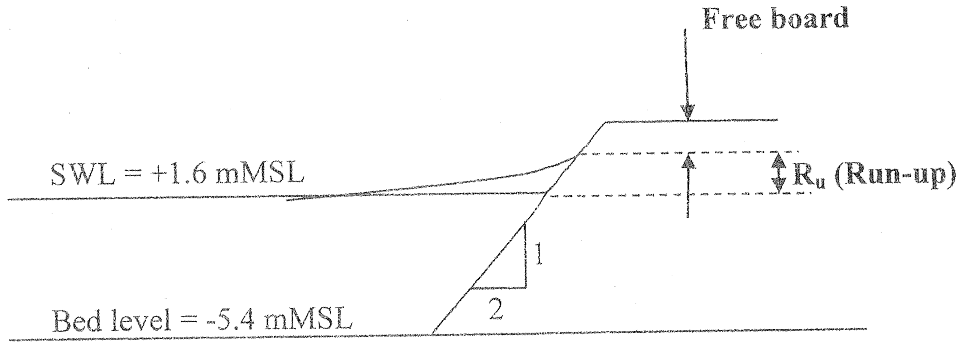

Figure Q5b.

Maximum wave run-up height (R_u) is given by:

$$\frac{R_{ui\%}}{H_s} = A\xi_{0m} \text{ for } 1.0<\xi_{0m} \leq 1.5 \qquad \text{Eq. 5.1}$$

$$= B(\xi_{0m})^C \text{ for } 1.5<\xi_{0m} \leq (D/B)^{1/C} \qquad \text{Eq. 5.2}$$

$$= D \text{ for } (D/B)^{1/C} \leq \xi_{0m} < 7.5 \qquad \text{Eq. 5.3}$$

$$\xi_{0m} = \frac{\tan\alpha}{\sqrt{S_{0m}}} \qquad \text{Eq. 5.4}$$

$$S_{0m} = \frac{H_{s0}}{L_0} \qquad \text{Eq. 5.5}$$

Assume a breakwater slope of 1:2. Design water level (SWL) is +1.6mMSL. Bed level at the toe of the structure is -5.4mMSL. Deep water wave height, H_{s0}=2.5m, and wave period, T_m=8 sec. Nearshore transformed wave height, H_s=2.8m. Assume a breaker index, γ_b=0.78.

[5 marks]

Table Q5. Coefficients A, B, C, D for Run-up calculation (for use in Eq 5.1 – 5.3)

Table VI-5-5
Coefficients in Equations VI-5-12 and VI-5-13 for Runup of Irregular Head-On Waves on Impermeable and Permeable Rock Armored Slopes

Percent [1]	A	B	C	D [2]
0.1	1.12	1.34	0.55	2.58
2.0	0.96	1.17	0.46	1.97
5	0.86	1.05	0.44	1.68
10	0.77	0.94	0.42	1.45
(significant)	0.72	0.88	0.41	1.35
50 (mean)	0.47	0.60	0.34	0.82

[1] Exceedence level related to number of waves
[2] Only relevant for permeable slopes

APPENDIX:

Table 1. Wave table

h/L0	h/L	Sinh(2π h/L)	Cosh(2πh/L)
0.030	0.07135	0.4634	1.1021
0.031	0.07260	0.4721	1.1059
0.032	0.07385	0.4808	1.1096
0.033	0.07507	0.4894	1.1133
0.034	0.07630	0.4980	1.1171
0.035	0.07748	0.5064	1.1209
0.036	0.07867	0.5147	1.1247
0.037	0.07984	0.5230	1.1285
0.038	0.08100	0.5312	1.1324
0.039	0.08215	0.5394	1.1362
0.040	.08329	0.5475	1.1401
0.041	.08442	0.5556	1.1440
0.042	.08553	0.5637	1.1479
0.043	.08664	0.5717	1.1518
0.044	.08774	0.5796	1.1558
0.060	0.1043	0.7033	1.2225
0.061	0.1053	0.7110	1.2270
0.062	0.1063	0.7187	1.2315
0.063	0.1073	0.7256	1.2355
0.064	0.1082	0.7335	1.2402
0.095	.1366	0.9677	1.3917
0.096	.1375	0.9755	1.3970
0.097	.1384	0.9832	1.4023
0.098	.1392	0.9908	1.4077
0.099	.1401	0.9985	1.4131
0.1000	.1410	1.006	1.4187
0.1010	.1419	1.014	1.4242
0.1020	.1427	1.022	1.4297
0.1030	.1436	1.030	1.4354
0.1040	.1445	1.037	1.4410