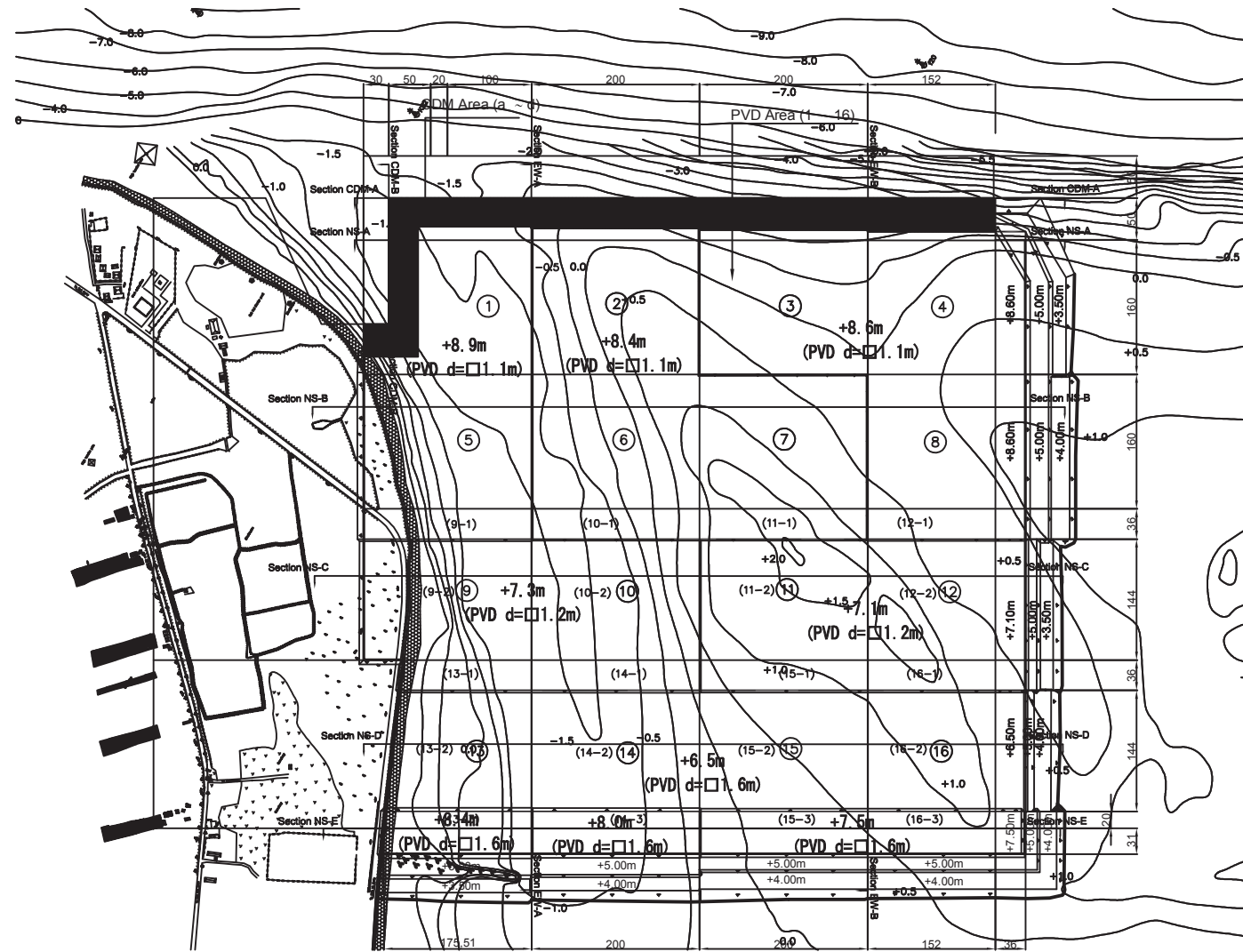


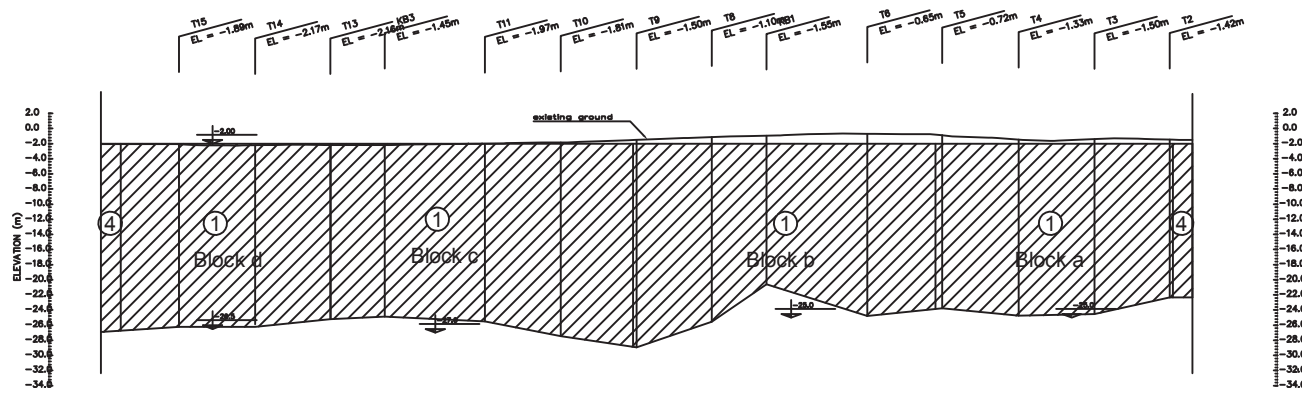
Phụ lục 16-1

Các bản vẽ mặt bằng cải tạo nền đất yếu

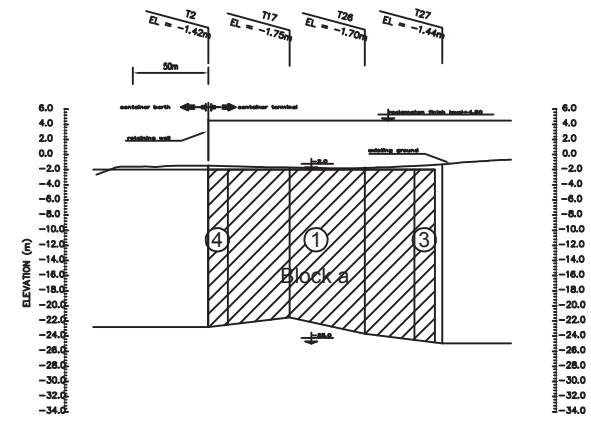
CDM LAYOUT PLAN  
Scale: 1/8000



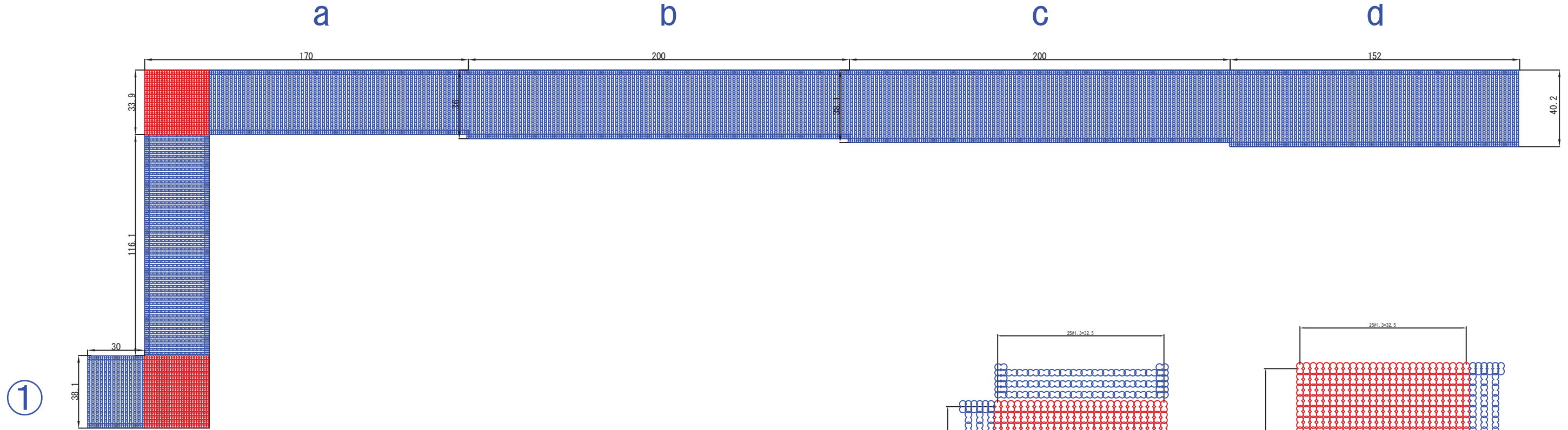
Cross Section (CDM - A)



Cross Section (CDM-B)

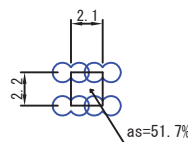
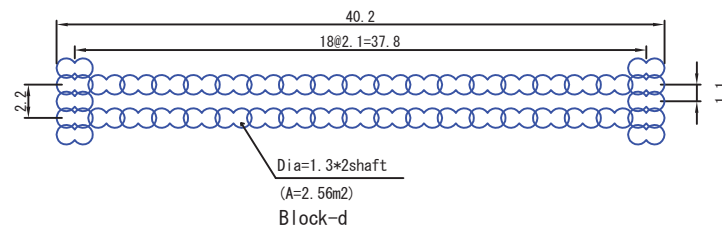
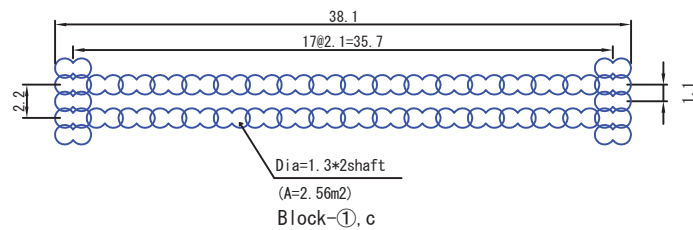
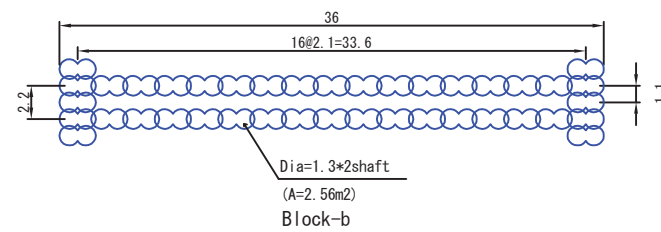
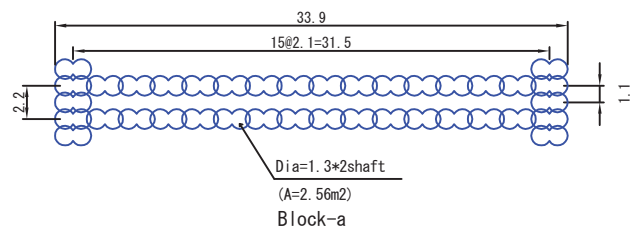
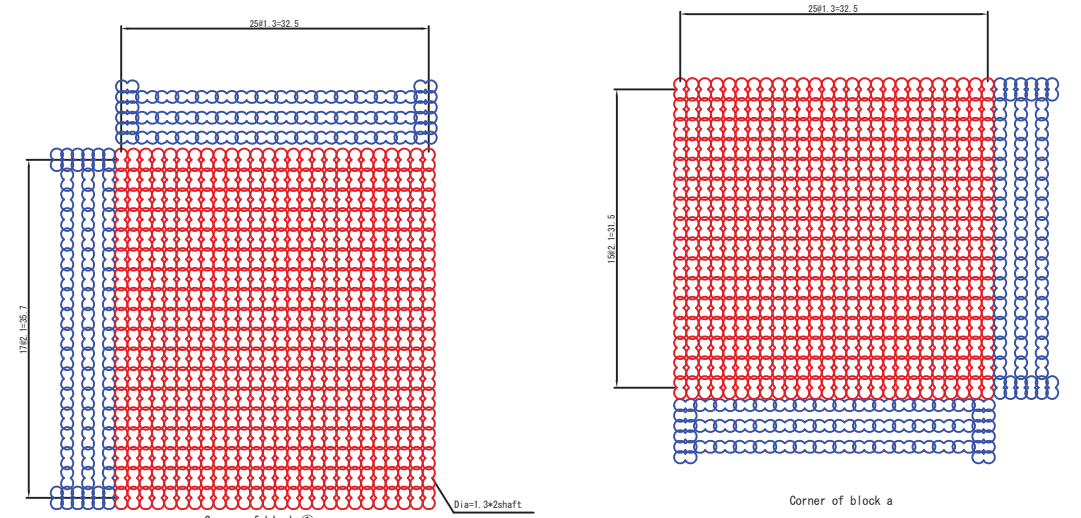


CDM DETAILED LAYOUT PLAN



Standard Layout of CDM Piles

Beside Steel Sheet Pipe Pile

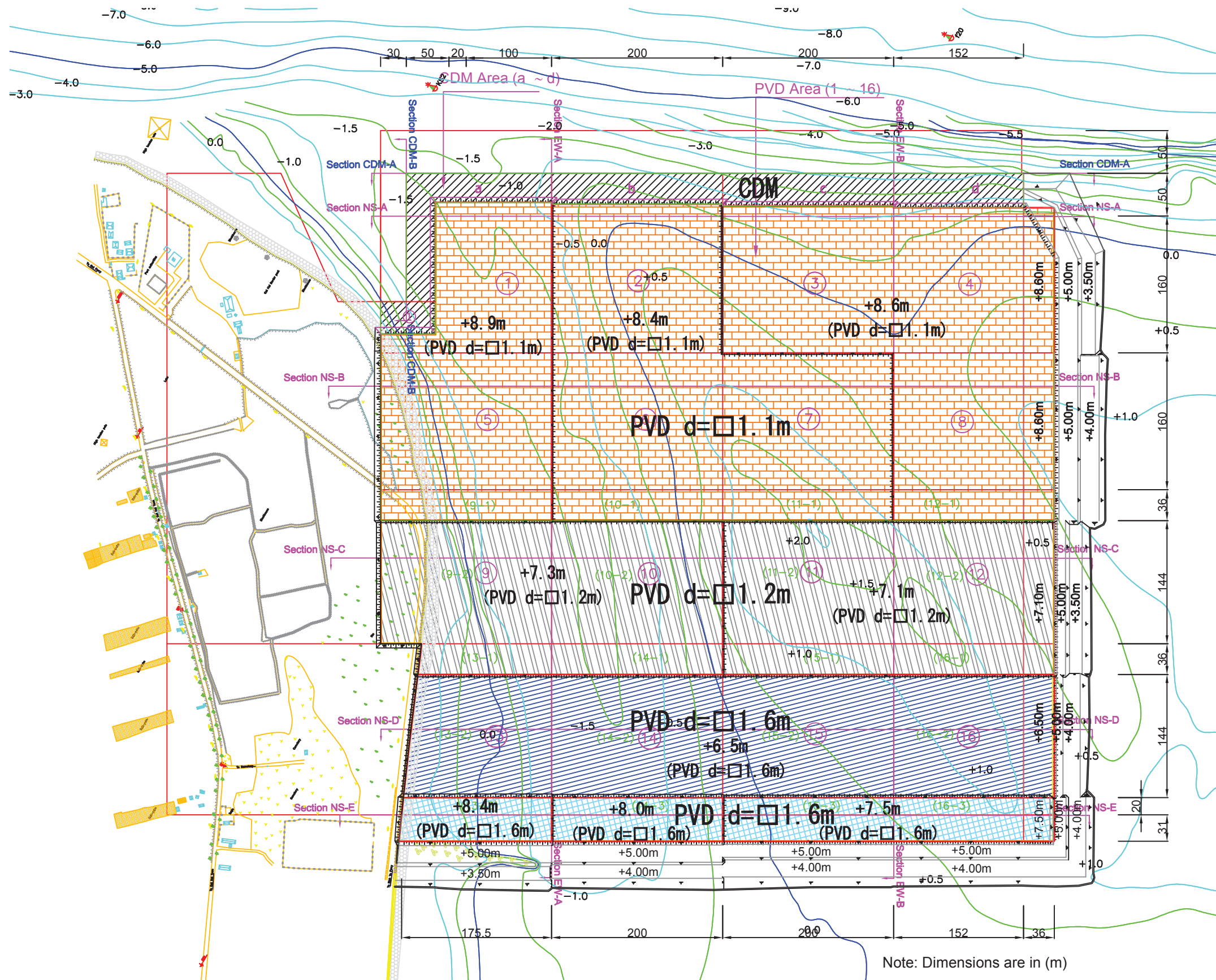


Note:  
- Dimensions are in (m)

Block	Area	Length of area	Number of piles per 1m of length	Piles of improvement	Specification of improvement					Soil volume of improvement	
					Formation level	Upper end	Lower end	Length of unimprovement	Length of improvement	Part of unimprovement	Part of improvement
		(m)	(piles/m)	(piles)	CDL (m)	CDL (m)	CDL (m)	(m)	(m)	(m³)	(m³)
①		204.0	3.64	746.0	5.0	-2.0	-26.0	7.0	24.0	13,368.3	45,834.2
a		640.0	3.64	2,485.0	5.0	-2.0	-25.0	7.0	23.0	44,531.2	146,316.8
b		472.0	3.64	1,727.0	5.0	-2.0	-25.0	7.0	23.0	30,947.8	101,685.8
c		476.2	3.64	1,821.0	5.0	-2.0	-27.0	7.0	25.0	32,632.3	116,544.0
d		384.7	3.64	1,450.0	5.0	-2.0	-26.5	7.0	24.5	25,984.0	90,944.0
Total		2,176.9		8,229.0						147,463.7	501,324.8

PRELOAD LAYOUT PLAN

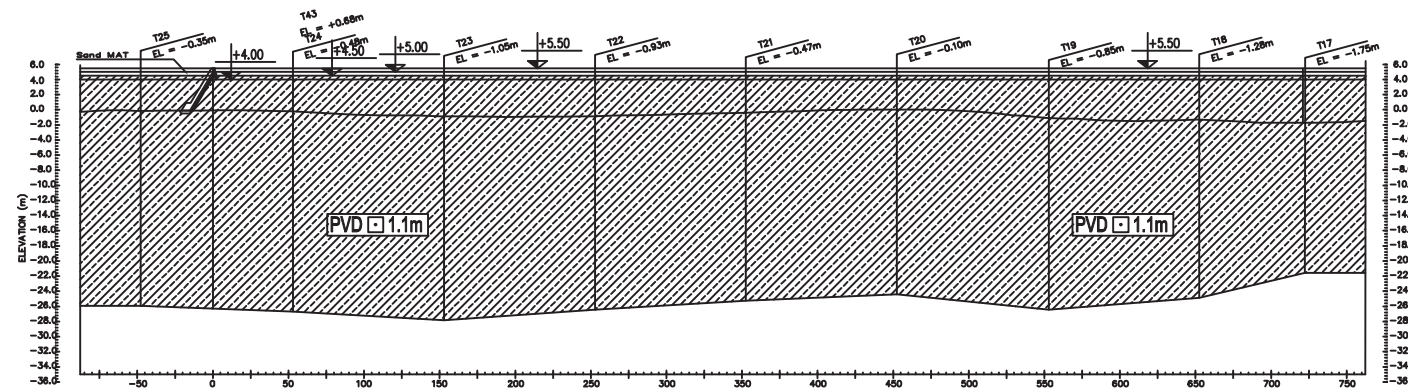
SCALE: 1/5000



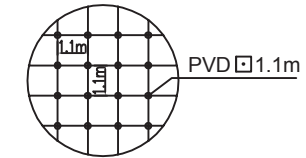
PVD CROSS SECTIONS (1/3)

Vertical scale: 1/1000  
Horizontal scale: 1/5000

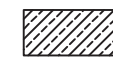
Section NS - A



PVD arrangement

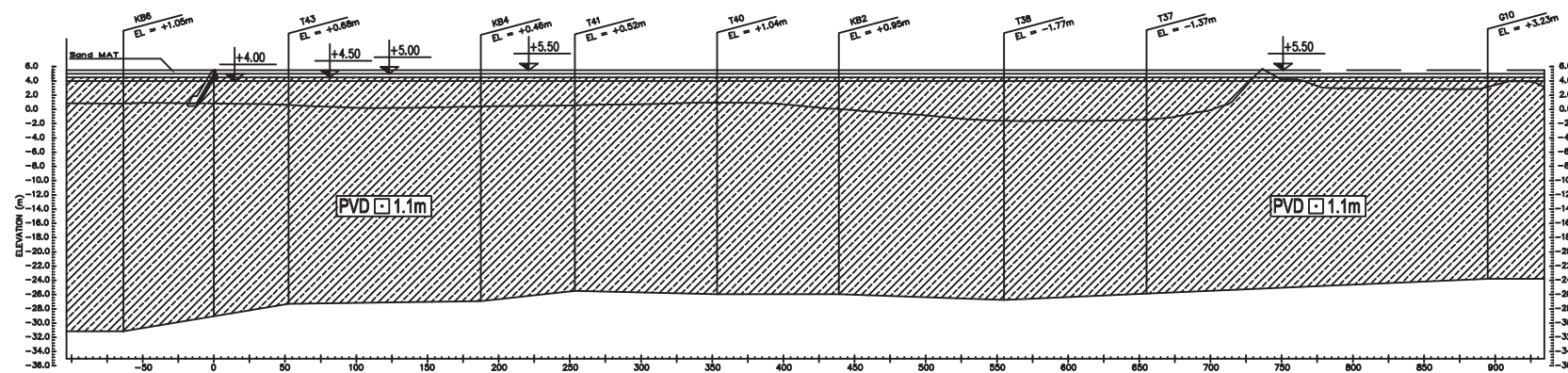


Note:

 Soil improvement (PVD □ 1.1m)

- Dimensions are in (m)
- Elevations are in (m) - chart datum

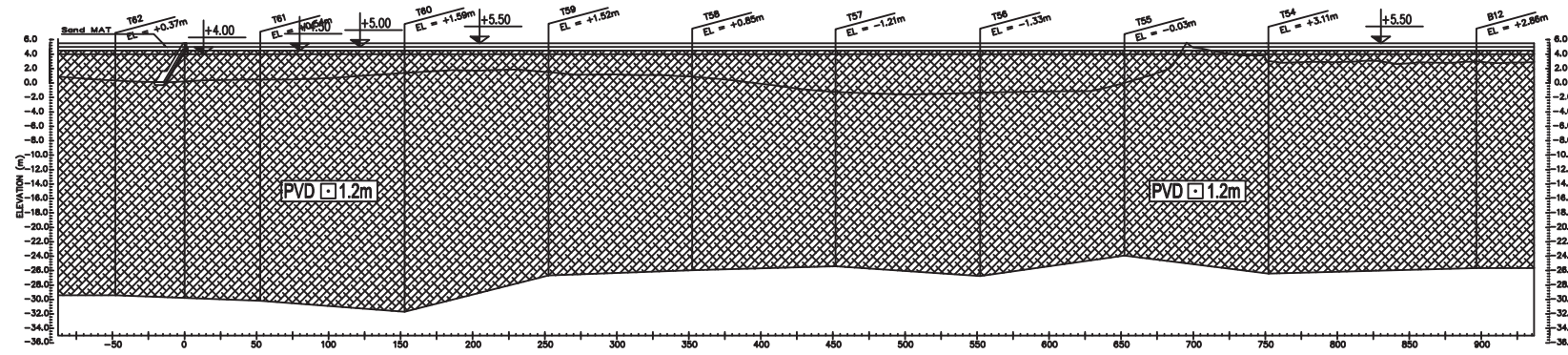
Section NS - B



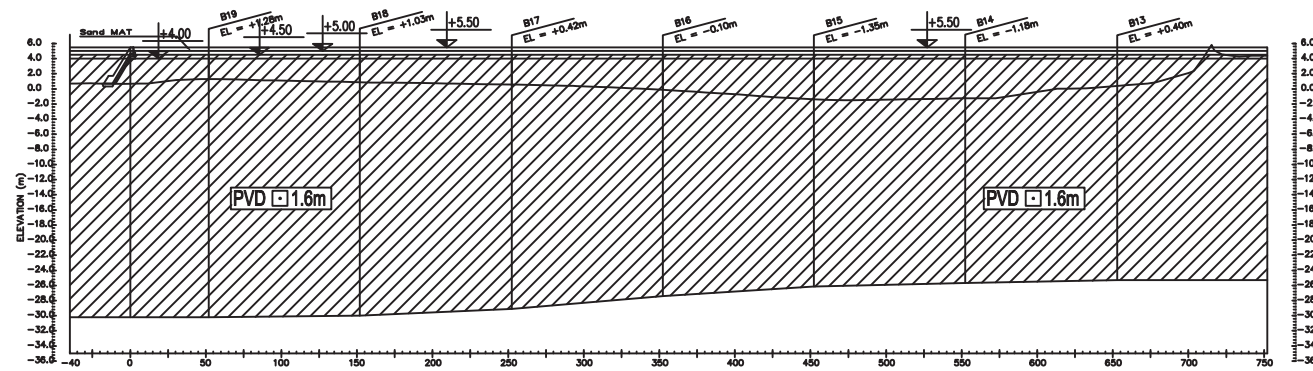
PVD CROSS SECTIONS (2/3)

Vertical scale: 1/1000  
Horizontal scale: 1/5000

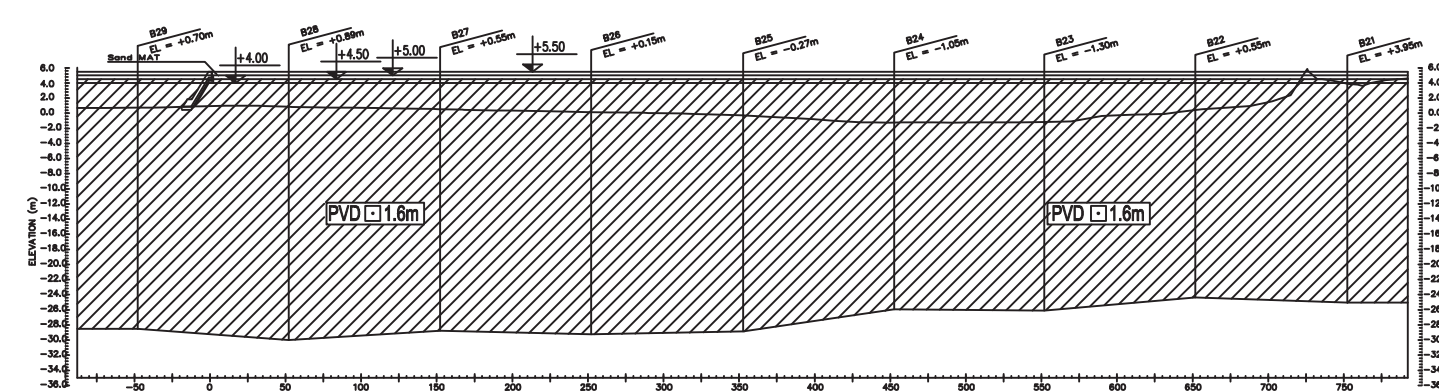
Section NS - C



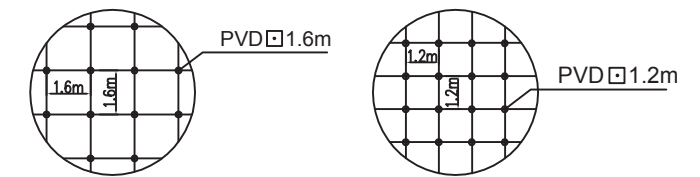
Section NS - D





Section NS - E



PVD arrangement



Note:

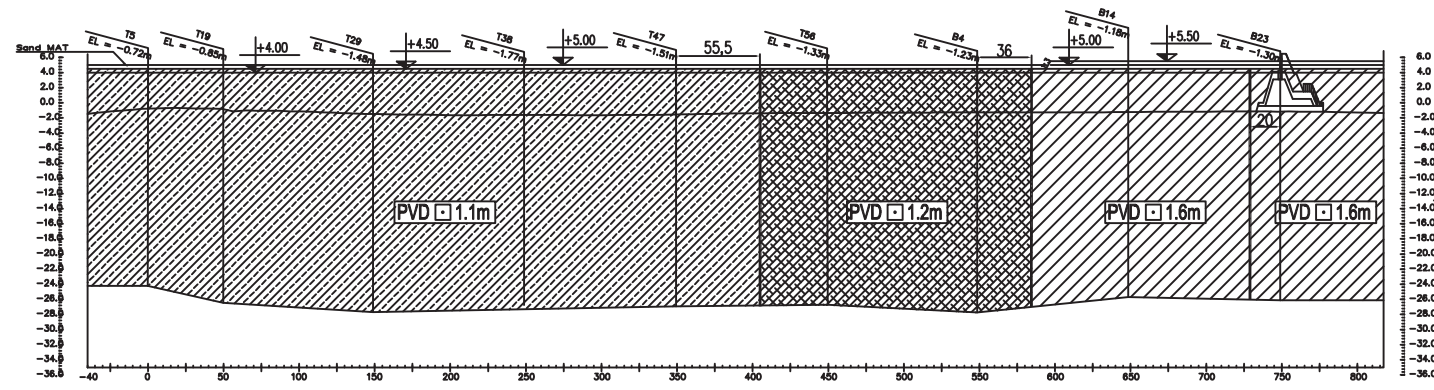
-  Soil improvement (PVD □ 1.6m)
-  Soil improvement (PVD □ 1.2m)

- Dimensions are in (m)
- Elevations are in (m) - chart datum

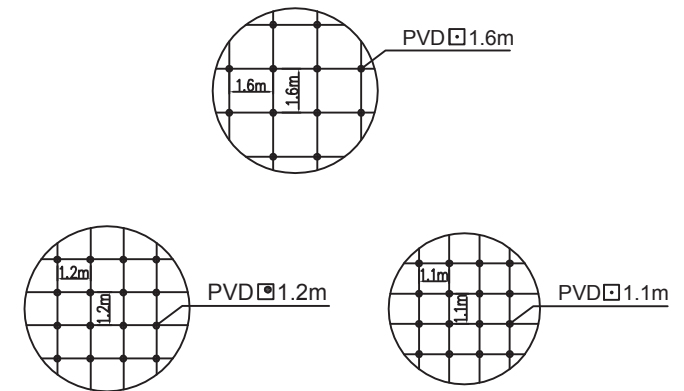
PVD CROSS SECTIONS (3/3)

Vertical scale: 1/1000  
Horizontal scale: 1/5000

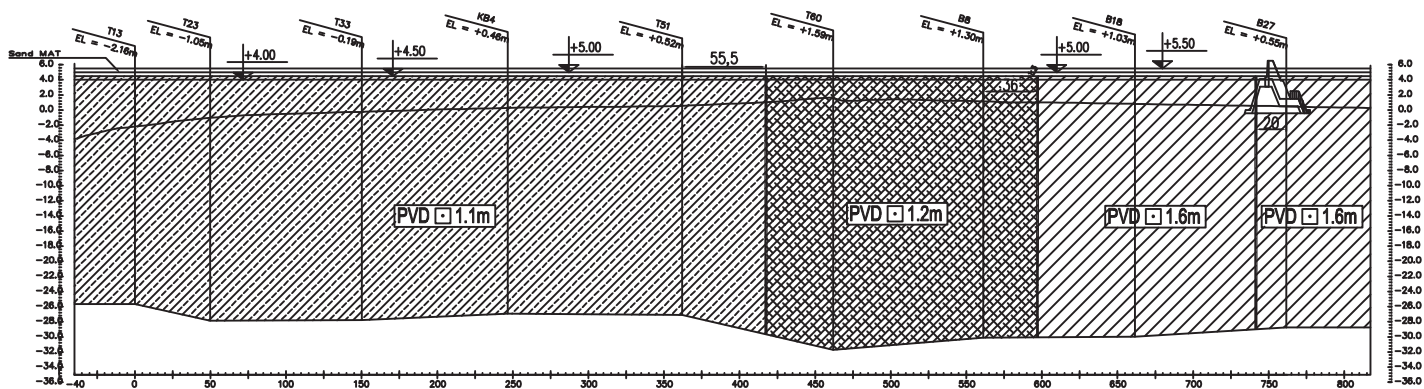
Section EW - A






PVD arrangement



Section EW - B

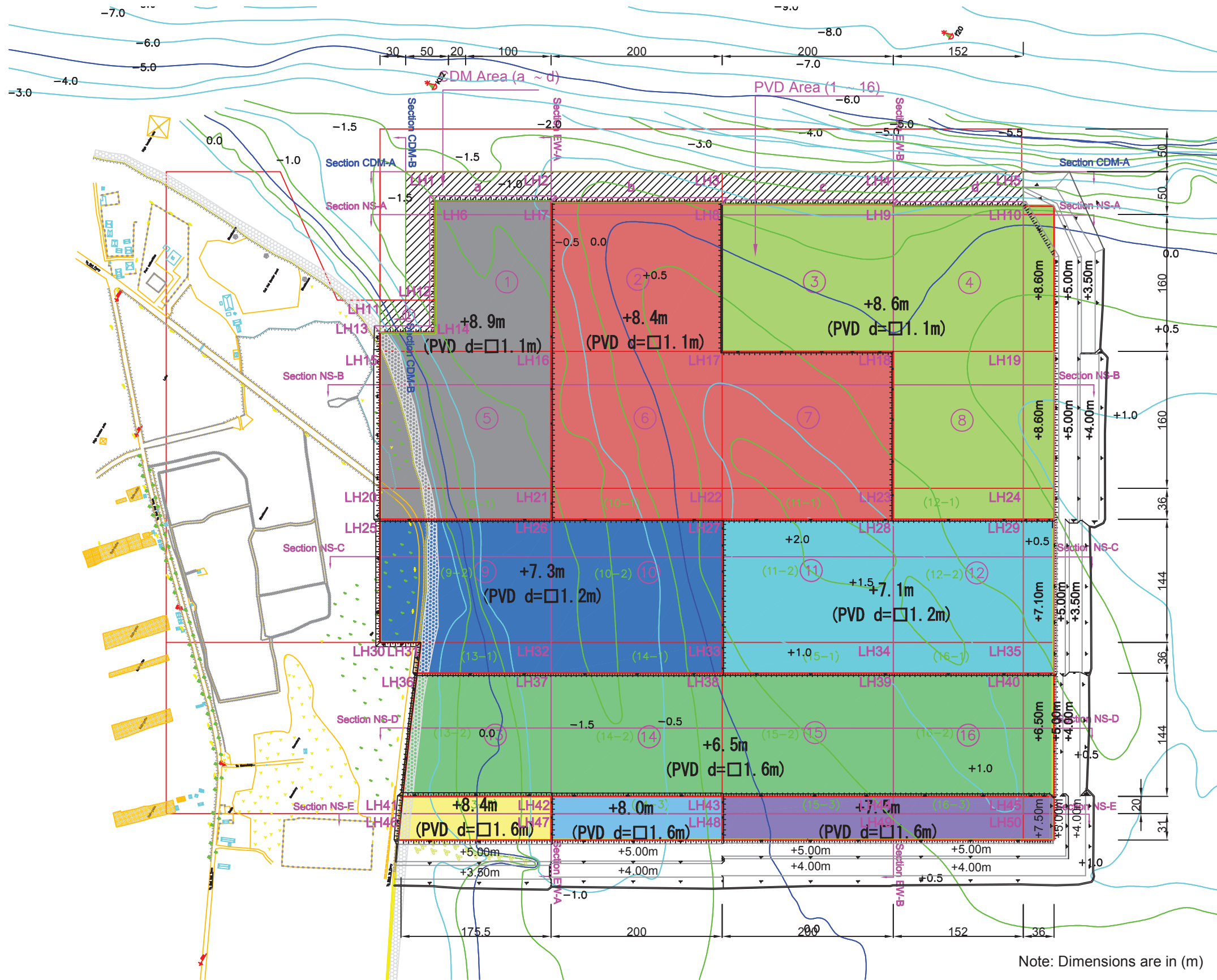


Note:

-  Soil improvement (PVD □ 1.1m)
-  Soil improvement (PVD □ 1.2m)
-  Soil improvement (PVD □ 1.6m)
- Dimensions are in (m)
- Elevations are in (m) - chart datum

PRELOAD LAYOUT PLAN

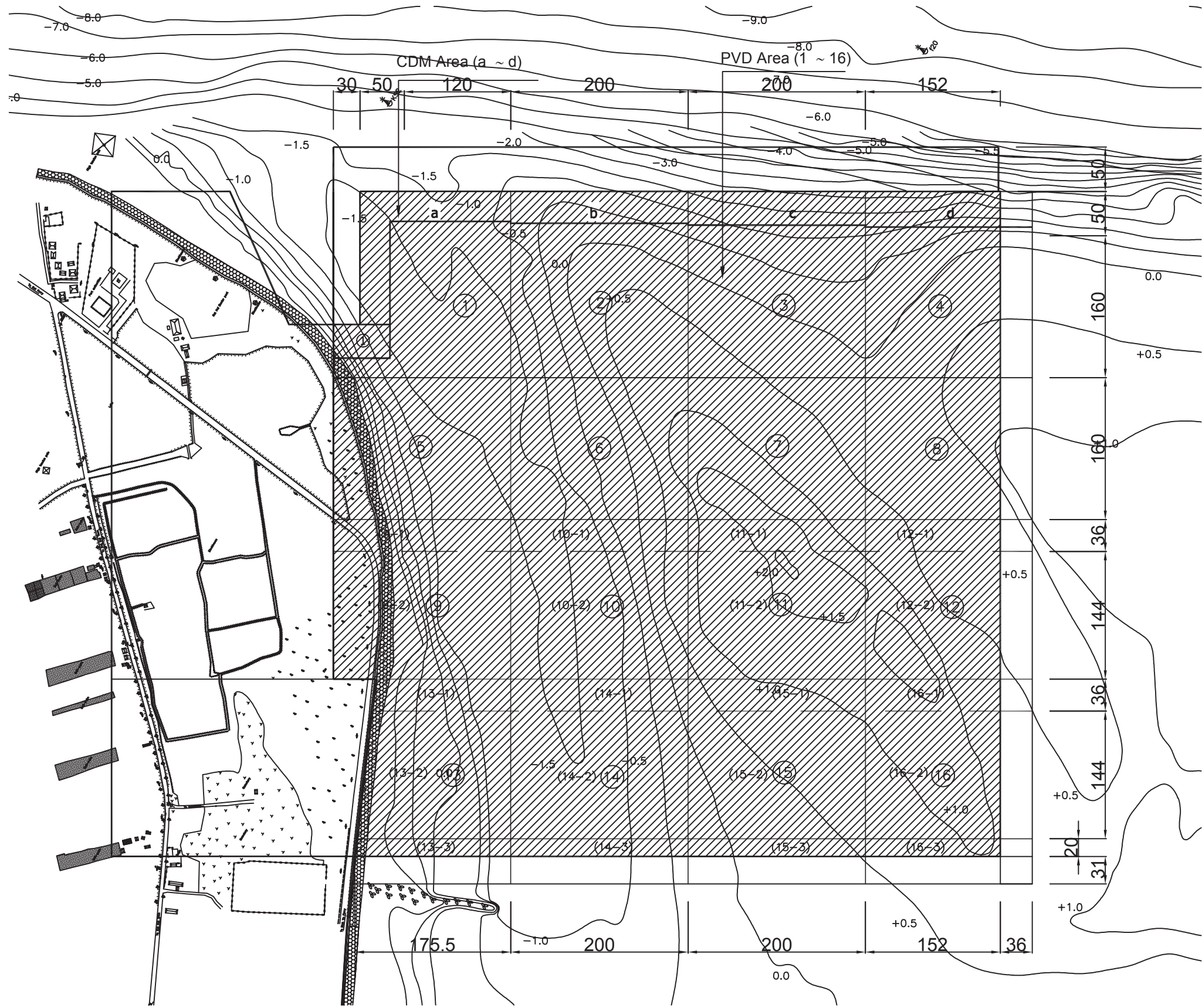
SCALE: 1/5000



CONTROLLING POINT'S COORDINATES OF RECLAMATION AREA

Coordinate		
Name	X	Y
LH1	620,134.700	2,301,485.748
LH2	620,221.865	2,301,339.795
LH3	620,324.413	2,301,168.086
LH4	620,426.960	2,300,996.377
LH5	620,504.896	2,300,865.878
LH6	620,122.991	2,301,439.270
LH7	620,192.774	2,301,322.422
LH8	620,293.519	2,301,149.636
LH9	620,394.263	2,300,976.850
LH10	620,470.397	2,300,845.274
LH11	619,990.536	2,301,434.594
LH12	620,005.918	2,301,408.838
LH13	619,957.836	2,301,415.072
LH14	619,990.603	2,301,360.206
LH15	619,939.021	2,301,403.834
LH16	620,041.571	2,301,232.121
LH17	620,144.118	2,301,060.411
LH18	620,246.666	2,300,888.702
LH19	620,324.602	2,300,758.203
LH20	619,801.656	2,301,321.792
LH21	619,904.204	2,301,150.083
LH22	620,006.751	2,300,978.373
LH23	620,109.298	2,300,806.664
LH24	620,187.235	2,300,676.165
LH25	619,770.748	2,301,303.333
LH26	619,873.296	2,301,131.624
LH27	619,975.843	2,300,959.915
LH28	620,078.391	2,300,788.206
LH29	620,156.327	2,300,657.707
LH30	619,647.118	2,301,229.499
LH31	619,671.583	2,301,188.534
LH32	619,749.665	2,301,057.790
LH33	619,852.213	2,300,886.081
LH34	619,954.760	2,300,714.372
LH35	620,032.696	2,300,583.873
LH36	619,638.518	2,301,173.688
LH37	619,718.758	2,301,039.331
LH38	619,821.305	2,300,867.622
LH39	619,923.853	2,300,695.913
LH40	620,001.789	2,300,565.414
LH41	619,506.218	2,301,114.369
LH42	619,595.127	2,300,965.497
LH43	619,697.675	2,300,793.788
LH44	619,800.222	2,300,622.079
LH45	619,878.158	2,300,491.580
LH46	619,487.964	2,301,105.928
LH47	619,577.956	2,300,955.242
LH48	619,680.504	2,300,783.533
LH49	619,783.051	2,300,611.824
LH50	619,860.987	2,300,481.325

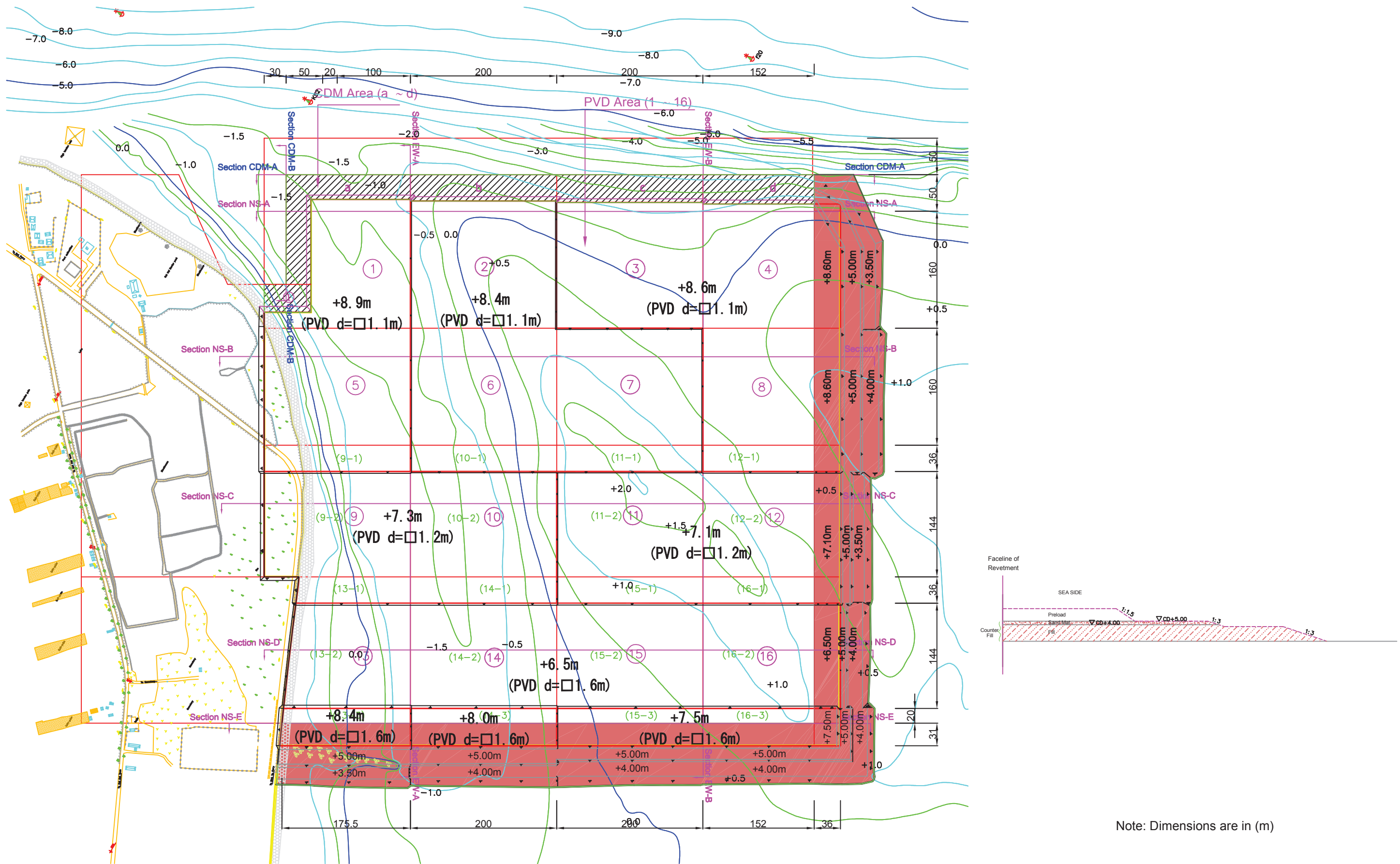




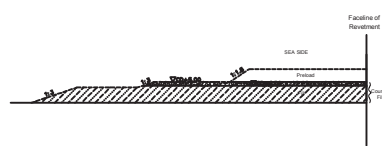
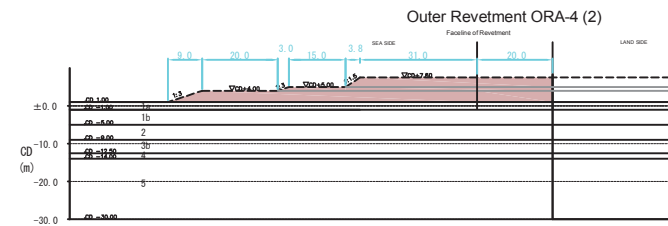
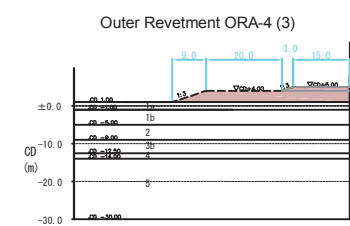
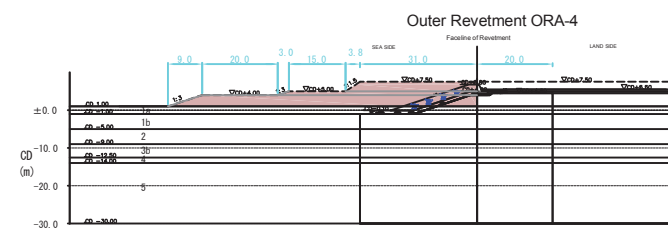
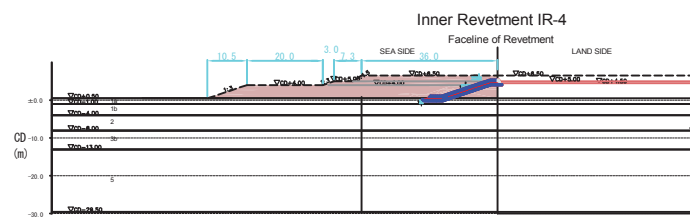
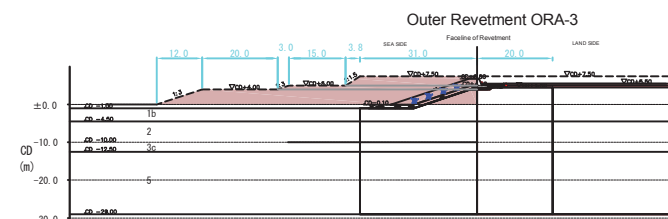
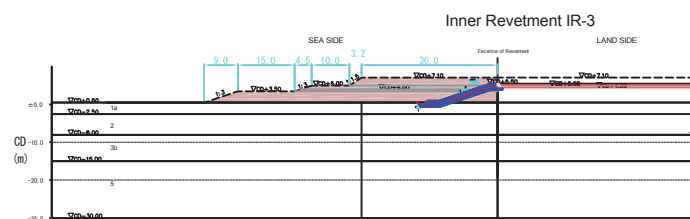
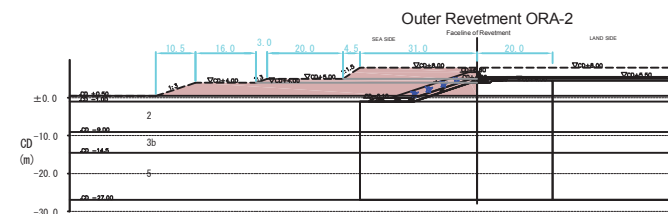
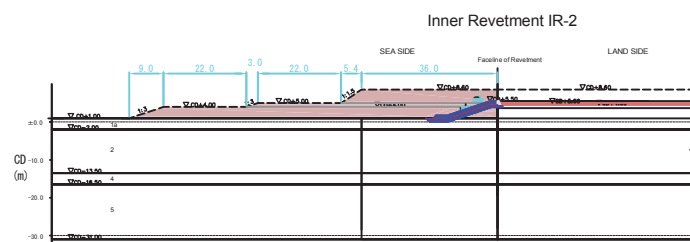
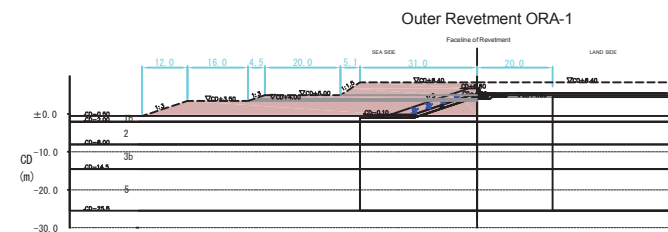
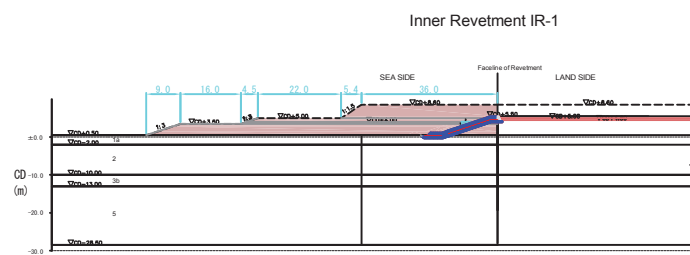
		Preload			Sand mat (CD+4.0m to +5.0m)			
		Area	Height	Quantity	Area	Height	Quantity	
		(m2)	(m)	(m3)	(m2)	(m)	(m3)	
CDM Area	a	9,699.5	-	-	9,699.5	1.0	9,699.5	
	b	7,200.0	-	-	7,200.0	1.0	7,200.0	
	c	7,611.6	-	-	7,611.6	1.0	7,611.6	
	d	6,110.4	-	-	6,110.4	1.0	6,110.4	
	①	2,434.8	-	-	2,434.8	1.0	2,434.8	
Reclamation Area	1	25,365.7	3.9	98,926.3	25,365.7	1.0	25,365.7	
	2	34,800.0	3.4	118,320.0	34,800.0	1.0	34,800.0	
	3	34,388.4	3.6	123,798.3	34,388.4	1.0	34,388.4	
	4	25,809.6	3.6	92,914.6	25,809.6	1.0	25,809.6	
	5	32,000.0	3.9	124,800.0	32,000.0	1.0	32,000.0	
	6	32,000.0	3.4	108,800.0	32,000.0	1.0	32,000.0	
	7	32,000.0	3.4	108,800.0	32,000.0	1.0	32,000.0	
	8	24,320.0	3.6	87,552.0	24,320.0	1.0	24,320.0	
	9-1	7,200.0	3.9	28,080.0	7,200.0	1.0	7,200.0	
	9-2	28,800.0	2.3	66,240.0	28,800.0	1.0	28,800.0	
	10-1	7,200.0	3.4	24,480.0	7,200.0	1.0	7,200.0	
	10-2	28,800.0	2.3	66,240.0	28,800.0	1.0	28,800.0	
	11-1	7,200.0	3.4	24,480.0	7,200.0	1.0	7,200.0	
	11-2	28,800.0	2.1	60,480.0	28,800.0	1.0	28,800.0	
	12-1	5,472.0	3.6	19,699.2	5,472.0	1.0	5,472.0	
	12-2	21,888.0	2.1	45,964.8	21,888.0	1.0	21,888.0	
	13-1	5,551.1	2.3	12,767.4	5,551.1	1.0	5,551.1	
	13-2	23,787.0	1.5	35,680.6	23,787.0	1.0	23,787.0	
	13-3	3,489.3	3.4	11,863.5	3,489.3	1.0	3,489.3	
	14-1	7,200.0	2.3	16,560.0	7,200.0	1.0	7,200.0	
	14-2	28,800.0	1.5	43,200.0	28,800.0	1.0	28,800.0	
	14-3	4,000.0	3.0	12,000.0	4,000.0	1.0	4,000.0	
	15-1	7,200.0	2.1	15,120.0	7,200.0	1.0	7,200.0	
	15-2	28,800.0	1.5	43,200.0	28,800.0	1.0	28,800.0	
	15-3	4,000.0	2.5	10,000.0	4,000.0	1.0	4,000.0	
	16-1	5,472.0	2.1	11,491.2	5,472.0	1.0	5,472.0	
	16-2	21,888.0	1.5	32,832.0	21,888.0	1.0	21,888.0	
	16-3	3,040.0	2.5	7,600.0	3,040.0	1.0	3,040.0	
	Total		552,327.3		1,451,889.8	552,327.3		552,327.3

TEMPORARY COUNTER FILL LAYOUT PLAN

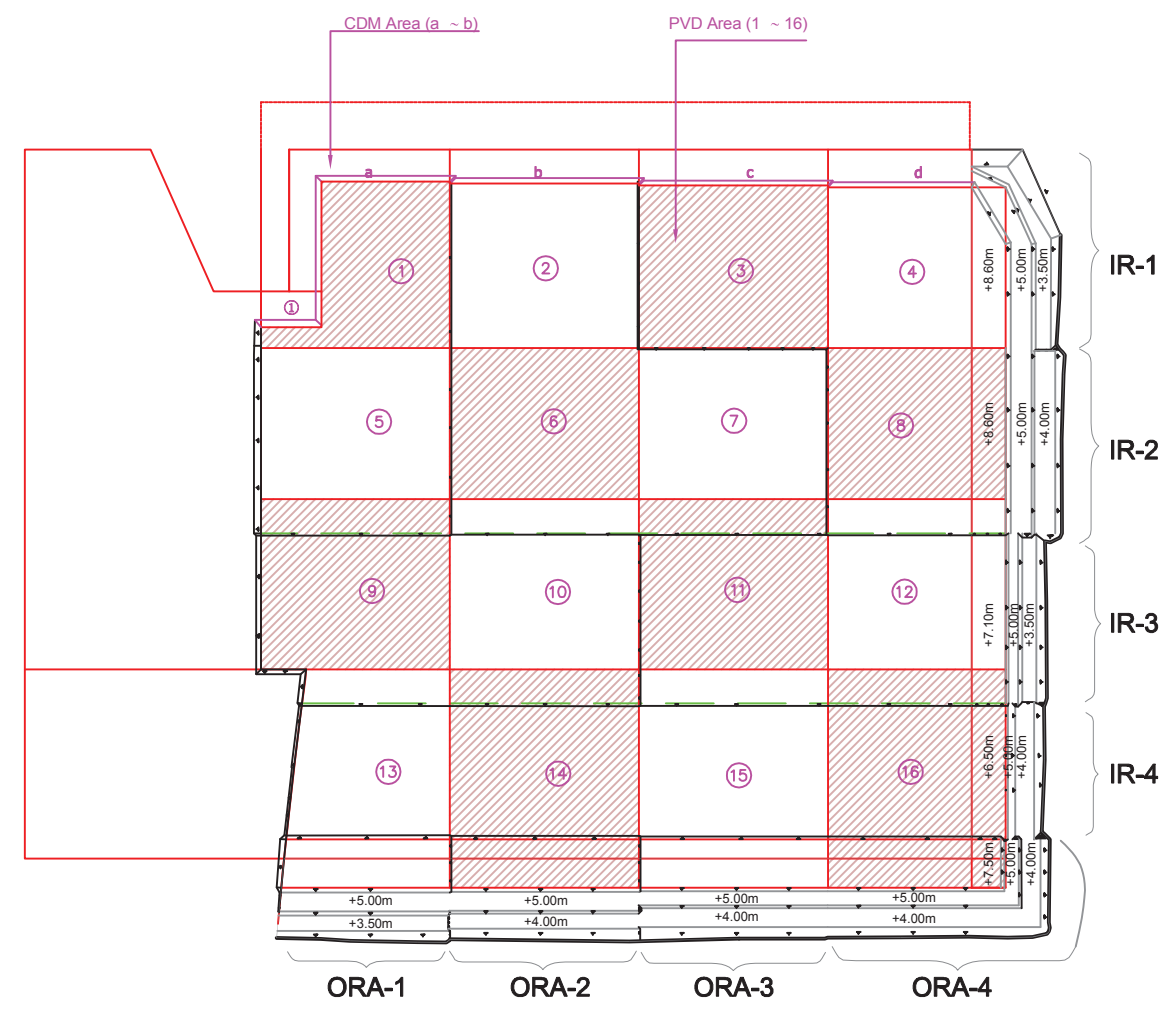
SCALE: 1/5000



Note: Dimensions are in (m)



SCALE: 1/2000

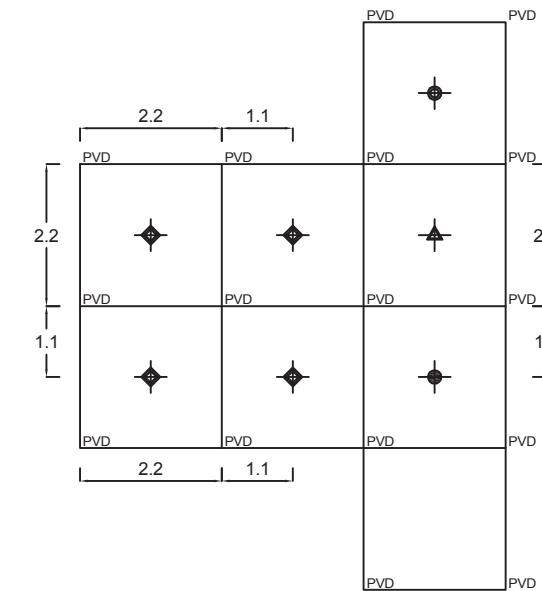
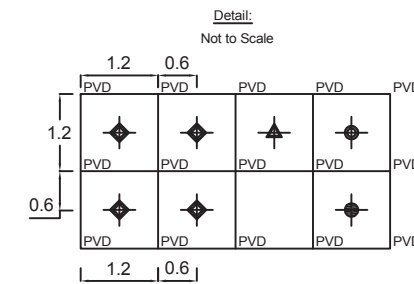
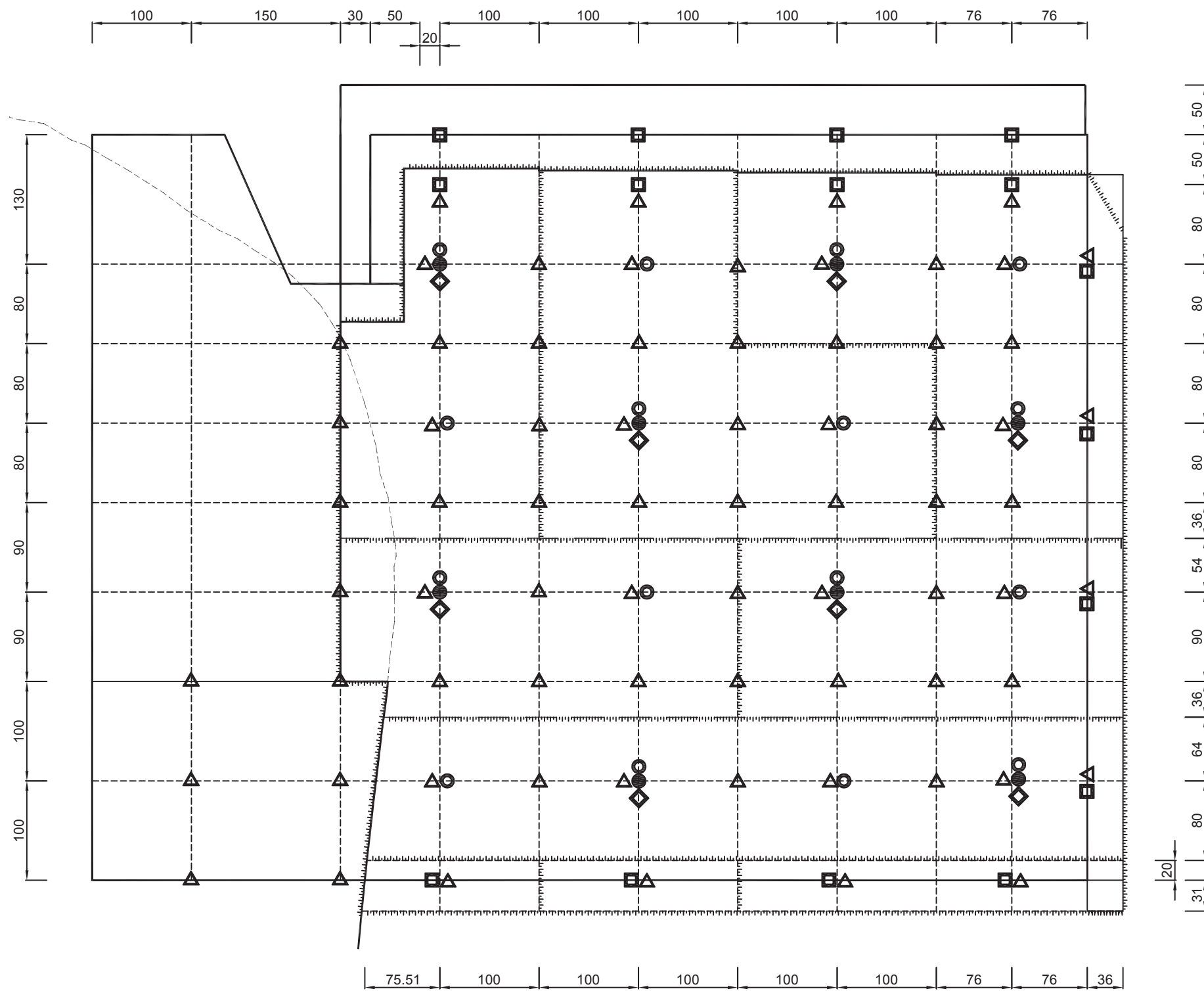


SCALE: 1/8000

TABLE OF QUANTITIES

		1st Counter Fill (< CD+4.0m)				2nd Counter Fill (Upto CD+4.0m)				Sand mat (CD+4.0m to +5.0m)				Preload (> CD+5.0m)				Fill protection sectional length (m)	Fill protection area (m <sup>2</sup> )
		Sectional Area (m <sup>2</sup> )	Height (m)	Distance along Revet. (m)	Volume (m <sup>3</sup> )	Sectional Area (m <sup>2</sup> )	Height (m)	Distance along Revet. (m)	Volume (m <sup>3</sup> )	Sectional Area (m <sup>2</sup> )	Height (m)	Distance along Revet. (m)	Volume (m <sup>3</sup> )	Sectional Area (m <sup>2</sup> )	Height (m)	Distance along Revet. (m)	Volume (m <sup>3</sup> )		
IR-1	1-1	265.2	3.0	110.0	29,172.0	33.6	0.5	110.0	3,692.7	64.9	1.0	110.0	7,139.0	139.3	3.6	110.0	15,325.2	30.2	3,325.3
	1-2				33,851.1				2,225.4				3,607.3				4,575.6	27.6	3,508.2
IR-2		278.7	3.0	196.0	54,625.2					64.9	1.0	196.0	12,720.4	139.3	3.6	196.0	27,306.7	34.7	6,791.4
IR-3		219.5	3.0	180.0	39,501.0	26.5	0.5	180.0	4,761.0	50.7	1.0	180.0	9,117.0	78.9	2.1	180.0	14,203.8	29.2	5,261.4
IR-4		245.0	3.5	144.0	35,280.0					43.4	1.0	144.0	6,242.4	55.7	1.5	144.0	8,019.4	34.2	4,924.8
ORA-1		330.4	4.0	178.4	58,946.7	29.9	0.5	178.4	5,338.0	57.6	1.0	178.4	10,276.4	114.1	3.4	178.4	20,351.2	33.4	5,957.1
ORA-2		297.1	4.0	200.0	59,426.0					57.0	1.0	200.0	11,400.0	99.8	3.0	200.0	19,950.0	30.2	6,046.0
ORA-3		315.0	4.0	200.0	63,000.0					51.3	1.0	200.0	10,250.0	82.2	2.5	200.0	16,438.0	35.8	7,162.0
ORA-4	4-1	231.8	4.0	152.0	35,226.0					51.3	1.0	152.0	7,790.0	82.2	2.5	152.0	12,492.9	32.7	4,964.3
	4-2	291.8	3.0	36.0	10,503.0					71.3	1.0	36.0	2,565.0	132.2	2.5	36.0	4,758.8	32.7	1,175.4
	4-3	127.5	3.0	101.7	12,966.8					16.5	1.0	101.7	1,678.1					32.7	3,320.5
Total		2,601.9	34.5	1,498.1	432,497.7	89.9	1.5	468.4	16,017.1	528.7	10.0	1,498.1	82,785.6	923.6	24.7	1,396.4	143,421.6	353.3	52,436.4

Note:  
- Dimensions are in (m)



### Frequency of Monitoring

Instrument	During construction of Surcharge Fill	After Completion of Surcharge Fill			After Removed Surcharge Fill
		First 1 month	1 to 3 months	Over 3 months	
Settlement Plate	Once a day	Once/2days	Once/1week	Once/2week	Timely
Extensometer	Once a day	Once/2days	Once/1week	Once/2week	Timely
Piezometers, Stand Pipe	Once a day	Once/2days	Once/1week	Once/2week	Timely
Stand Pipe	Once a day	Once/2days	Once/1week	Once/2week	Timely
Inclinometers	Once a day	Once/2days	Once/1week	Timely	—

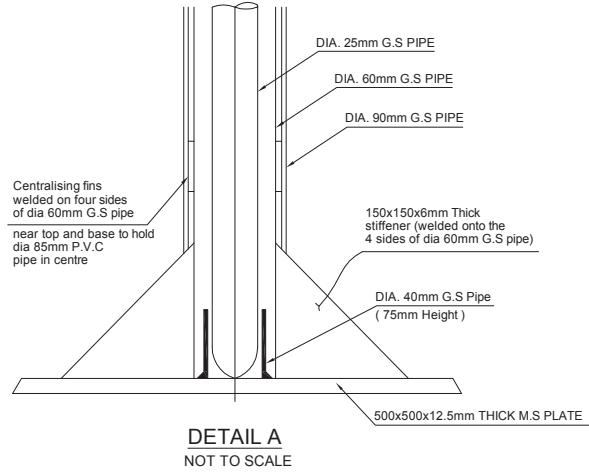
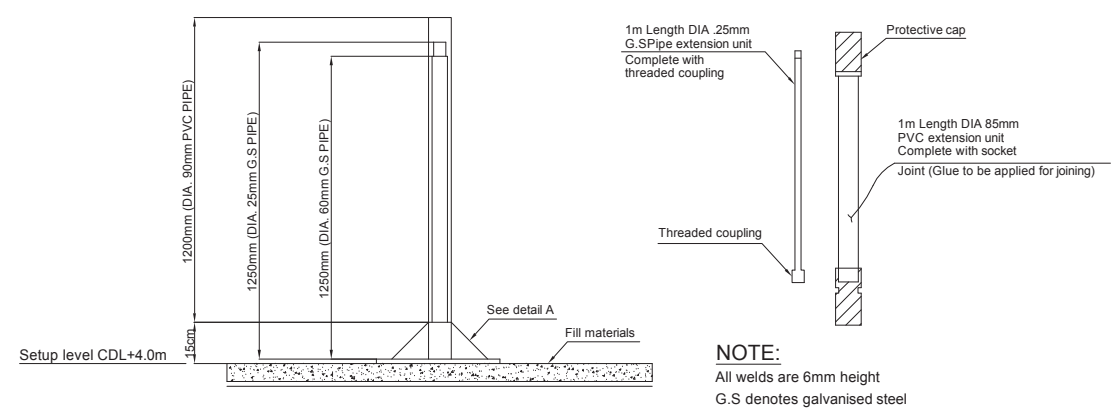
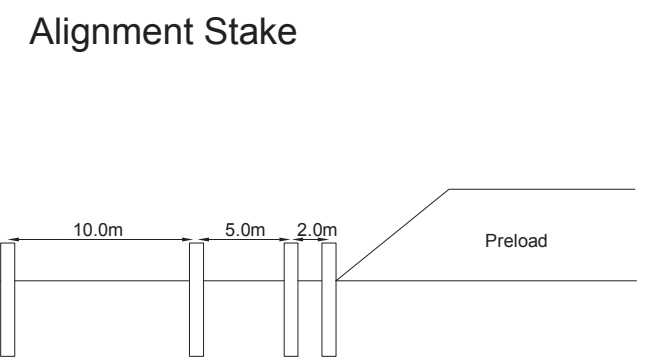
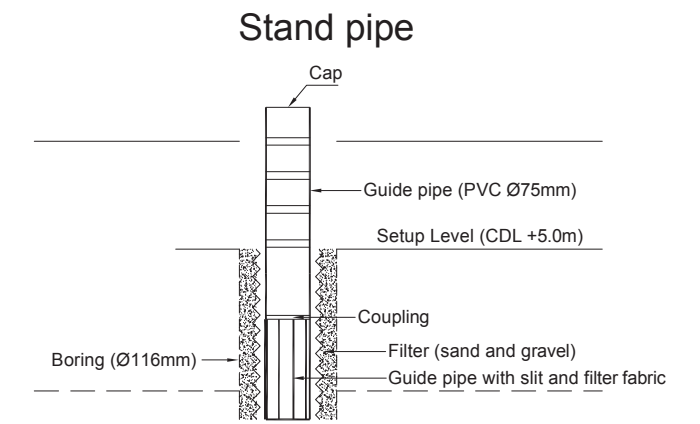
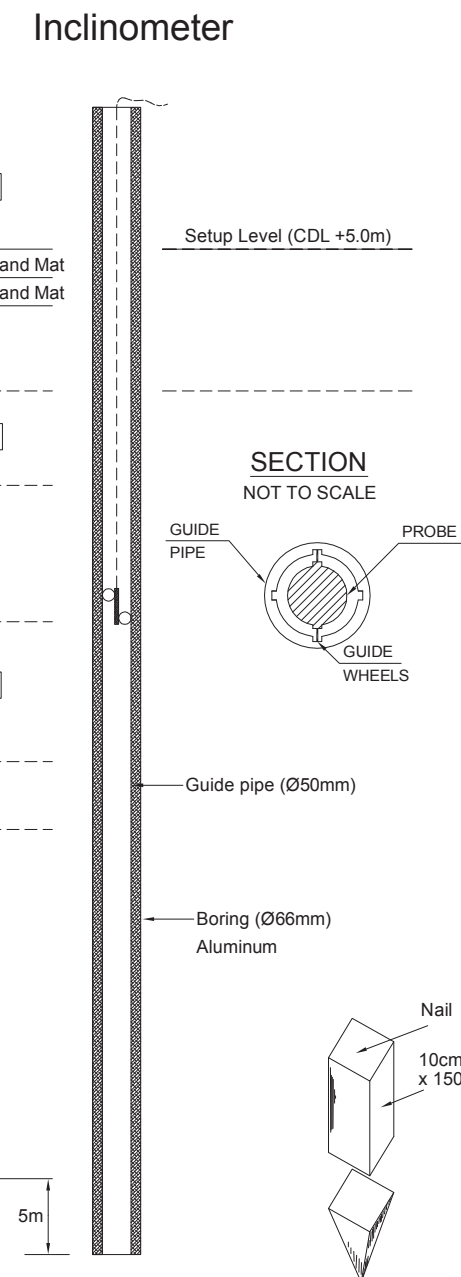
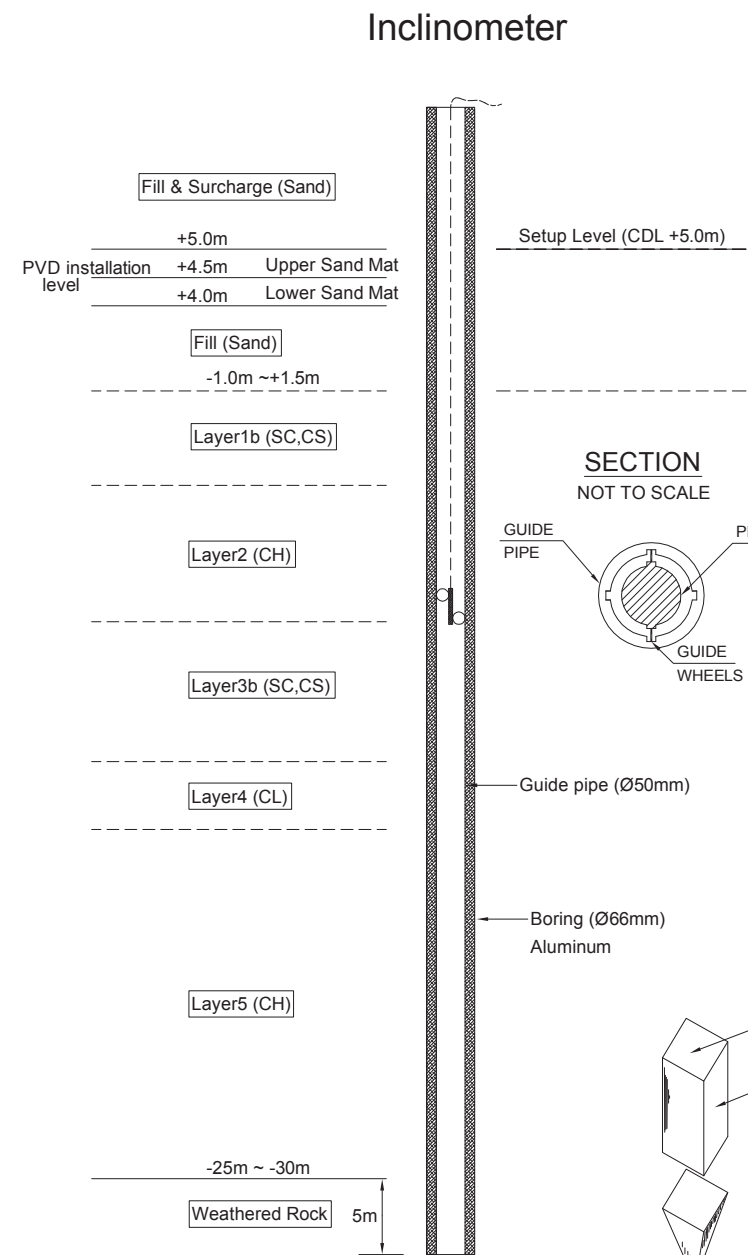
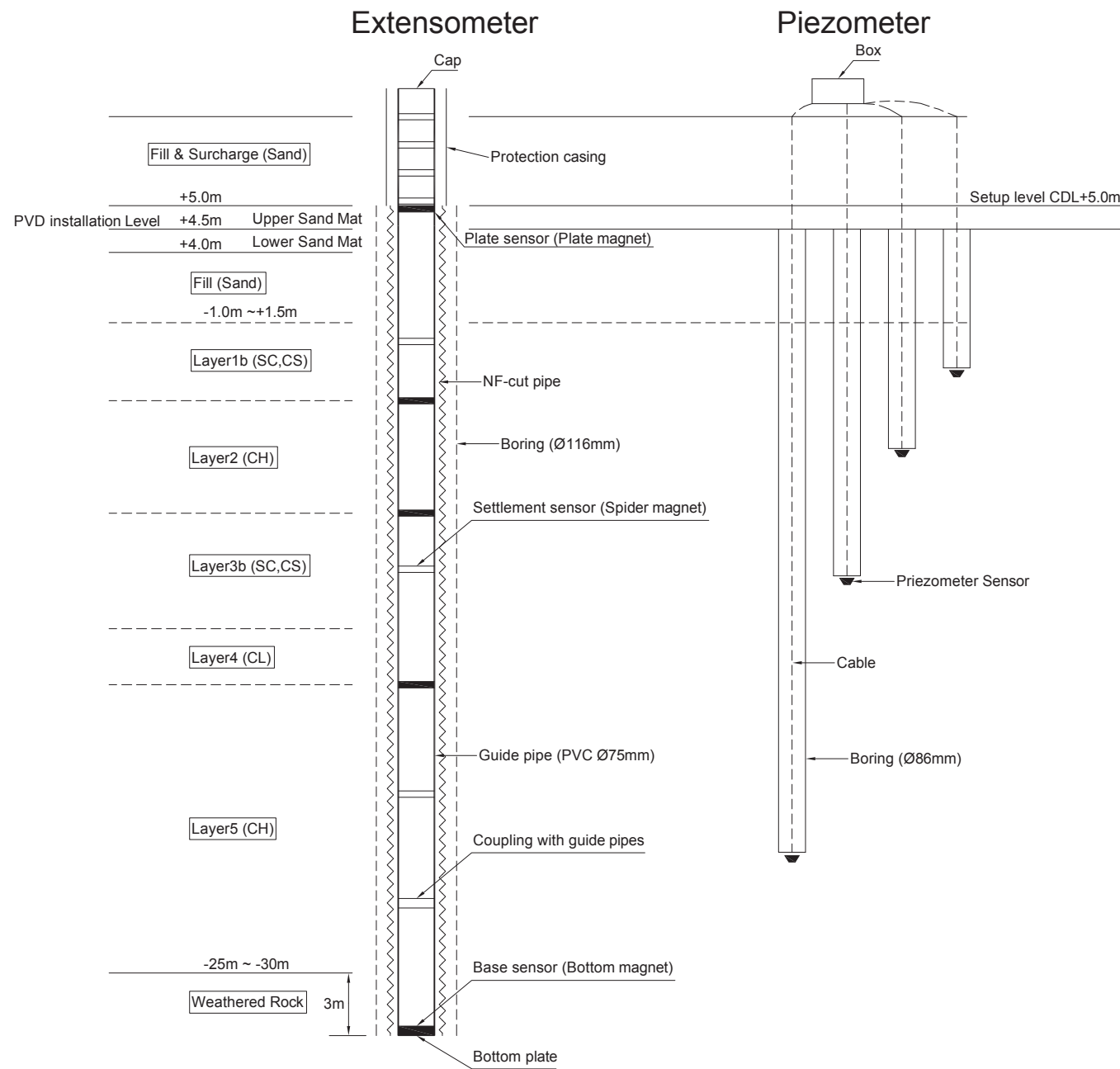
### Quantities of Monitoring Instruments

Legend	Monitoring Instruments	Units	Quantities	Remarks
△	Settlement Plate	Nos.	71	
●	Magnet Extensometer	Nos.	8	
□	Inclinometer	Nos.	16	
⊙	Stand Pipe	Nos.	16	
◇	Electric Piezometer	Nos.	32	8 Locations x 4 layers

### Notes:

- Dimensions are in (m), Elevations are in (m) - "Hon Dau" Datum
- Monitoring plans as shown are for reference only, Actual locations and Nos, shall be decided at site
- Detail and Installation procedure of Monitoring instruments are shown in drawing No: IM-29
- Coordinates of control points are shown in drawing No: IM-24
- Piezometer elevations show the Monitoring points of pore water pressure
- Settlement plate shall be placed on top of a layer of filling materials which is 0.5m thickness

INSTALLATION DETAILS  
FOR MONITORING INSTRUMENTS



- Notes:**
- Dimensions are in (m), Elevations are in (m) - "Hon Dau" Datum
  - Monitoring plans of Reclamation Area are show in drawing No: IM-28
  - Settlement plate shall be placed on top of a layer of filling materials which is 0.5m thickness
  - Each Piezometer shall be installed in each one borehole

Phụ lục 16-2

Tính toán khối neo (đoạn góc)

## Design of Anchor Block (L-Wall)

### 1. Type of Anchor Block

The design of the ordinary slab anchorage shall satisfy the following based on the Technical Standards and Commentaries for Port and Harbour Facilities in Japan.

(a) Stability of concrete wall anchorage

The height and placing depth of concrete wall anchorage may be determined to satisfy equation (5.7.1), on the assumption that the tie rod tension and the active earth pressure behind the concrete wall anchorage are resisted by the passive earth pressure in front of the concrete wall anchorage (see Fig. T- 5.7.4).

$$F = E_p / (A_p + E_A) \quad (5.7.1)$$

where

$F$  : safety factor

$A_p$  : reaction at the tie rod setting point according to 5.3.3 Bending Moment of Sheet Pile and Reaction at Tie Rod Setting Point (N/m)

$E_A$  : resultant active earth pressure acting on concrete wall anchorage (N/m)

$E_p$  : resultant passive earth pressure acting on concrete wall anchorage (N/m)

In the calculation of the earth pressure acting on concrete wall anchorage, the surcharge should be assumed to act as shown in Fig. T- 5.7.4 (i.e., the surcharge is only considered in the calculation of active earth pressure, and not of passive earth pressure). In this calculation, the safety factors should be 2.5 or larger in ordinary conditions and 2.0 or larger in extraordinary conditions.

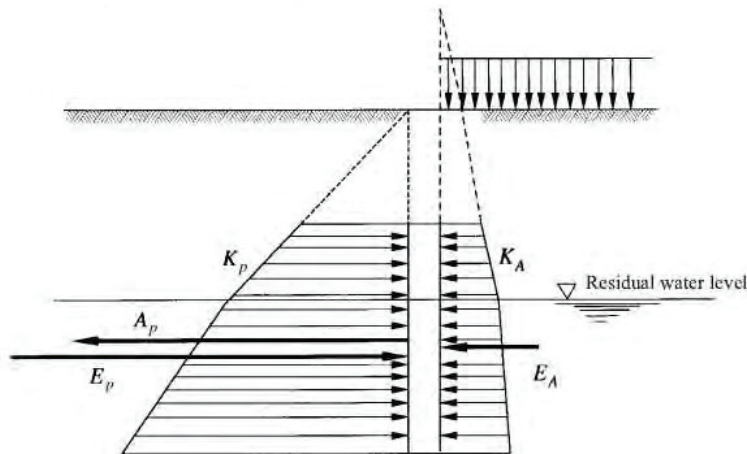


Fig. T- 5.7.4 External Forces Acting on Concrete Wall Anchorage

The above design method is applied in the case that the tie setting point is located at around the centroid of the resultant earth pressure.

If the simple slab anchorage is used, the tie setting point will be around +1.6 mCDL as shown in the trial calculation shown in the next page.

However, for the case of this project, the water level behind the retaining wall during the construction is estimated to be +2.5 mCDL ~ +3.55 mCDL and tie setting point 1.0m~2.0m below the water level would be difficult to be executed within the limited time frame.

For this reason, L-wall type anchor block with tie setting point at +3.0 mCDL is recommended.

## [Traial calculation for ordinary slab anchorage]

### After Completion

Elev (mCD)	h (m)	Passive Earth Pressure				Active Earth Pressure				Resultant Force		
		$\gamma h$ (kPa)	$K_p$	$p_p$ (kPa)	$P_p$ (kN/m)	$\gamma h$ (kPa)	$K_a$	$p_a$ (kPa)	$P_a$ (kN/m)	$P_p - P_a$ (kN/m)	$y$ (m)	$M$ (kNm/m)
5.50	1.20	0.00	4.806			30.00	0.291					
4.30		21.60	4.806			51.60	0.291					
4.30	0.75	21.60	4.806	103.81	38.9	51.60	0.291	15.02	5.6	33.3	4.25	141.5
3.55		35.10	4.806	168.69	63.3	65.10	0.291	18.94	7.1	56.2	4.00	224.6
3.55	3.75	35.10	4.806	168.69	316.3	65.10	0.291	18.94	35.5	280.8	2.50	701.9
-0.20		72.60	4.806	348.92	654.2	102.60	0.291	29.86	56.0	598.2	1.25	747.8
					1,072.7				104.2	968.5		1,815.9

Tie tension force (Block-1)  $T = 319.3$  kN/m  
 Factor of Safety  $FOS = P_p / (T + P_a) = 2.53 > 2.5 \dots OK$   
 Center of earth pressure Elev = 1.675 mCD

### During Construction

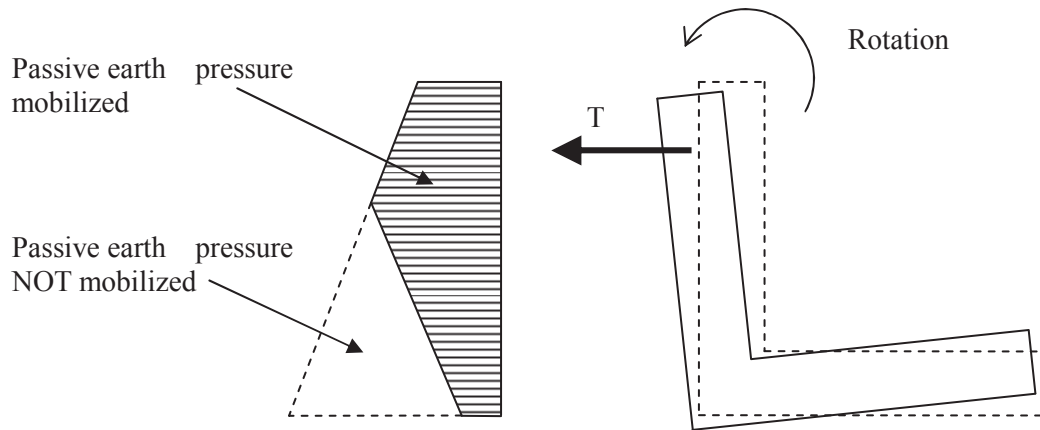
Elev (mCD)	h (m)	Passive Earth Pressure				Active Earth Pressure				Resultant Force		
		$\gamma h$ (kPa)	$K_p$	$p_p$ (kPa)	$P_p$ (kN/m)	$\gamma h$ (kPa)	$K_a$	$p_a$ (kPa)	$P_a$ (kN/m)	$P_p - P_a$ (kN/m)	$y$ (m)	$M$ (kNm/m)
4.50	0.20	0.00	4.806			20.00	0.291					
4.30		3.60	4.806			23.60	0.291					
4.30	0.75	3.60	4.806	17.30	6.5	23.60	0.291	6.87	2.6	3.9	4.05	15.8
3.55		17.10	4.806	82.18	30.8	37.10	0.291	10.80	4.0	26.8	3.80	101.7
3.55	3.55	17.10	4.806	82.18	145.9	37.10	0.291	10.80	19.2	126.7	2.37	299.9
0.00		52.60	4.806	252.80	448.7	72.60	0.291	21.13	37.5	411.2	1.18	486.6
					631.9				63.3	568.6		904.1

Tie tension force (Block-1)  $T = 246.72$  kN/m  
 Factor of Safety  $FOS = P_p / (T + P_a) = 2.04 > 2.0 \dots OK$   
 Center of earth pressure Elev = 1.590 mCD



## 2. Method of Verification

For the case of the L-wall type anchor block with tie setting point at +3.0 mCDL, a rotation of the L-wall would be an issue since the centroid of the passive earth pressure will be much lower than the tie setting point. Due to the rotation, a mobilization of the passive earth pressure at the lower portion of the wall would be insufficient.



In order to design the anchor block to meet the requirements of the Technical Standards and Commentaries for Port and Harbour Facilities in Japan, following design method is adopted.

- Rotation of the L-wall is considered in the design. The passive earth pressure and the ground reaction are obtained by the elasto-plastic method. For this purpose, linear soil springs are modeled on the front and bottom surfaces of the L-wall but the passive resistance is limited to the passive earth pressure.
- Following two cases are studied:
  - Working load case  
This analysis is based on the unfactored loads.  
The purpose of this analysis is to check the serviceability of the L-wall (displacement/rotation) as well as the structural details of the L-wall.
  - Ultimate load case  
This analysis is based on the factored tie tension and active earth pressure.  
The factors to be imposed are 2.5 for the permanent condition and 2.0 for the temporary condition (i.e., during construction).  
The purpose of this analysis is to check the ultimate capacity of the passive resistance of the L-wall in the case of the insufficient mobilization of the passive earth pressure due to the rotation of the L-wall.
- Following criteria are set for the acceptance of the design.
  - Rotation angle  $\theta$  shall not exceed 0.02 rad in both of the working and ultimate load cases.
  - Lateral displacement at the tie setting point shall not exceed 30mm in working load case.

- Soil spring are set as follows:
  - SPT-N value of 5 is assumed.
  - Coefficient of ground reaction :  $K = 2000 \times (\text{SPT-N}) \text{ (kN/m}^3\text{)}$
  
- Superimposed loads of 30kPa (after completion) or 20 kPa (during construction) are considered for the calculation of the active earth pressure. During surcharging work for the PVD improvement behind the DMM, the surcharge soil will be filled just behind the L-wall. However, this effect will not be considered since such surcharge fill will not be loaded within the active failure surface behind the L-wall and any minor effects in relation to the surcharge soil (or surcharging work) is considered to be covered by the superimposed load of 20kPa.

### 3. Section of L-wall and Results of Analysis

Dimensions of the L-wall sections are as follows.

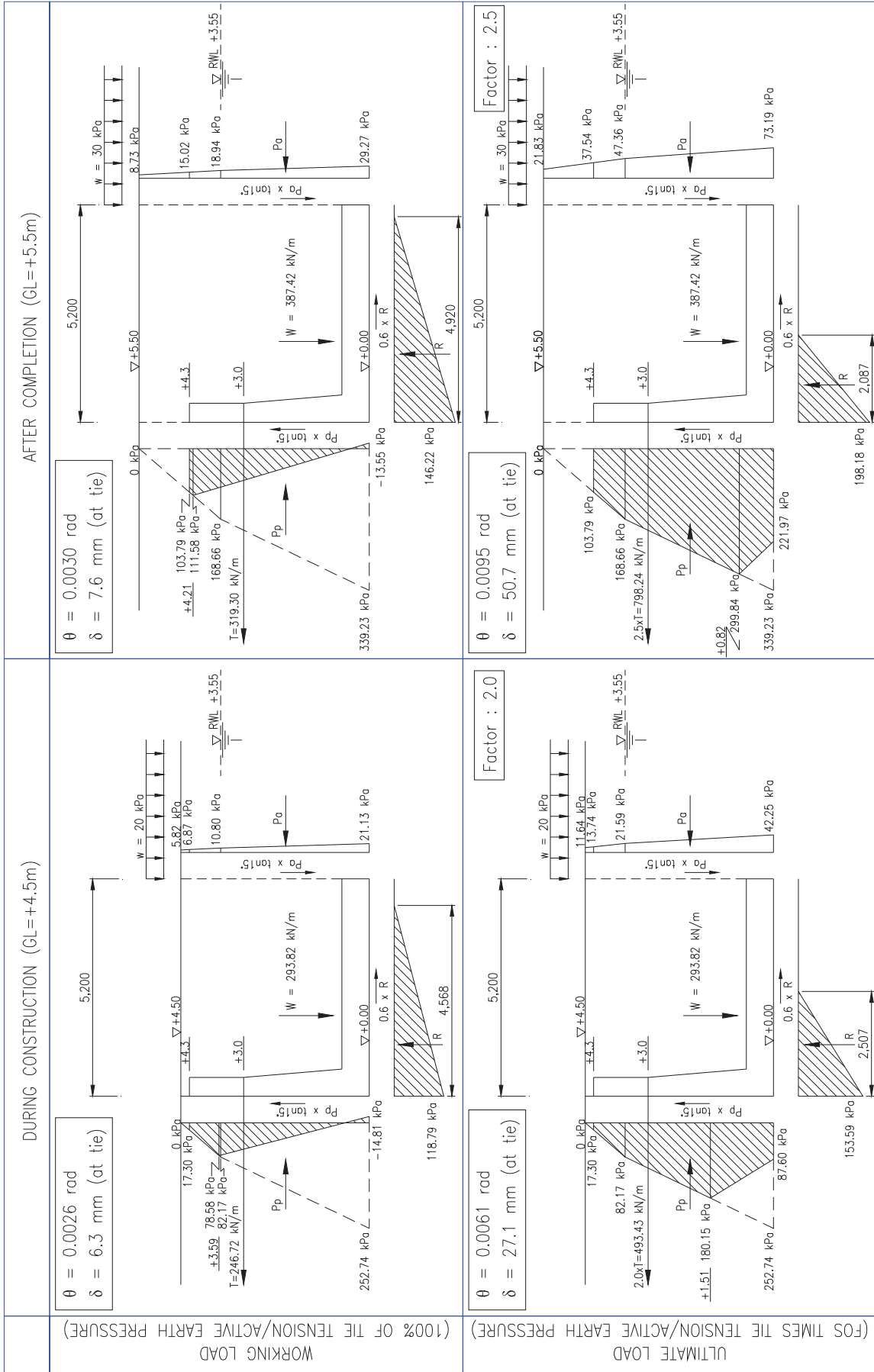
	Top Level	Bottom Level	Width	Thickness	
				Wall	Base Slab
Block-1	+4.3 mCD	+0.00 mCD	5.20 m	0.45 m	0.65 m
Block-a	+4.3 mCD	+0.30 mCD	4.00 m	0.45 m	0.65 m
Block-b	+4.3 mCD	+0.30 mCD	4.20 m	0.45 m	0.65 m
Block-c	+4.3 mCD	+0.30 mCD	4.20 m	0.45 m	0.65 m
Block-d	+4.3 mCD	+0.30 mCD	4.50 m	0.45 m	0.65 m

The design check was carried out along with the above procedure by using Microsoft Excel spreadsheet. The analysis results are shown in the following pages and summarized in the following table.

		Block-1		Block-a		Block-b		Block-c		Block-d		
		During Construction	After Completion	During Construction	After Completion	During Construction	After Completion	During Construction	After Completion	During Construction	After Completion	
Working Load	Rotation angle $\theta$	0.0026 rad	0.0030 rad	0.0030 rad	0.0034 rad	0.0029 rad	0.0033 rad	0.0029 rad	0.0033 rad	0.0027 rad	0.0031 rad	
	Acceptance Criteria	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	
	Check	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	
	Horizontal Deflection $\delta$	6.3 mm	7.6 mm	5.1 mm	6.3 mm	5.2 mm	6.4 mm	5.1 mm	6.3 mm	4.9 mm	6.1 mm	
	Acceptance Criteria	$\delta \leq 30\text{mm}$	$\delta \leq 30\text{mm}$	$\delta \leq 30\text{mm}$	$\delta \leq 30\text{mm}$	$\delta \leq 30\text{mm}$	$\delta \leq 30\text{mm}$	$\delta \leq 30\text{mm}$	$\delta \leq 30\text{mm}$	$\delta \leq 30\text{mm}$	$\delta \leq 30\text{mm}$	$\delta \leq 30\text{mm}$
	Check	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	
Ultimate Load (Factored Load)	Factor imposed on Working Load	$\gamma_f = 2.0$	$\gamma_f = 2.5$	$\gamma_f = 2.0$	$\gamma_f = 2.5$	$\gamma_f = 2.0$	$\gamma_f = 2.5$	$\gamma_f = 2.0$	$\gamma_f = 2.5$	$\gamma_f = 2.0$	$\gamma_f = 2.5$	
	Rotation angle $\theta$	0.0061 rad	0.0095 rad	0.0072 rad	0.0101 rad	0.0077 rad	0.0140 rad	0.0069 rad	0.0101 rad	0.0047 rad	0.0052 rad	
	Acceptance Criteria	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	$\theta \leq 0.02\text{rad}$	
	Check	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	
	Horizontal Deflection $\delta$	27.1 mm	50.7 mm	22.2 mm	38.8 mm	24.5 mm	49.5 mm	22.9 mm	41.1 mm	19.4 mm	32.8 mm	
	Acceptance Criteria	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	
Check	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A		

Summary of the results is also included in **Attachment-6**.

# Summary of Analysis (Block-1)



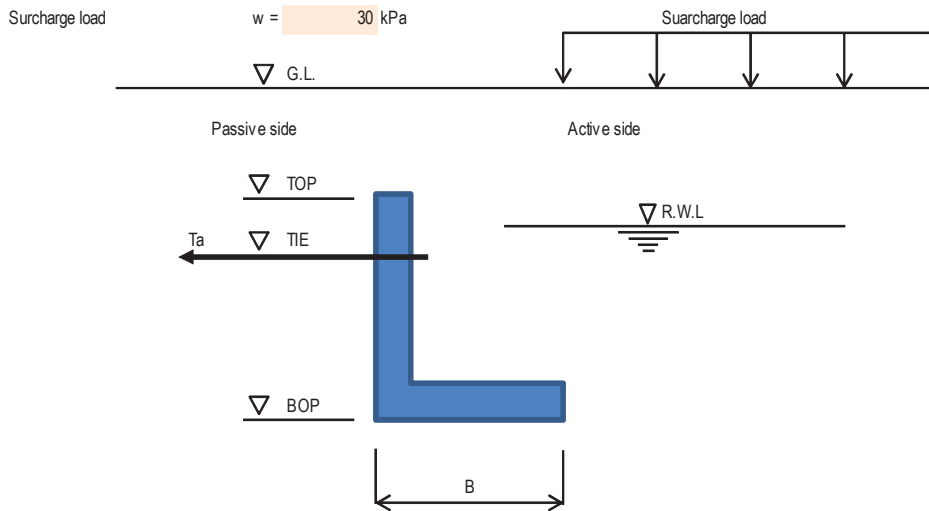
**Design of Anchor Plate (Elast-plastic method)**

**Block Block-1**

**Case : During Operation - - - Tie tension FOS = 2.5**

**1. Design Conditions**

Ground level	GL	5.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.00 m
Width of anchor plate	B	5.20 m
Tie elevation	TIE	3.00 m
Tie tension force	Ta	319.30 kN/m
Factor of safety	FOS	2.5
	FOS×Ta =	798.2375 kN/m
	M =	2394.7125 kNm/m



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cosδ
	Top	Bottom				
layer-1	5.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.00	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cosδ
	Top	Bottom				
layer-1	5.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.00	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cosδ	pp (kPa)	Pp (kN/m)
5.50	1.20	0.00	4.805	0.00	0.00
4.30		21.60	4.805	103.79	62.27
4.30	0.75	21.60	4.805	103.79	38.92
3.55		35.10	4.805	168.66	63.25
3.55	3.55	35.10	4.805	168.66	299.36
0.00		70.60	4.805	339.23	602.14
					1,065.94

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	pa (kPa)	Pa (kN/m)	y (m)	Ma (kNm/m)
5.50	1.20	30.00	0.291	8.73	5.24	5.10	26.71
4.30		51.60	0.291	15.02	9.01	4.70	42.34
4.30	0.75	51.60	0.291	15.02	5.63	4.05	22.80
3.55		65.10	0.291	18.94	7.10	3.80	27.00
3.55	3.55	65.10	0.291	18.94	33.63	2.37	79.58
0.00		100.60	0.291	29.27	51.96	1.18	61.49
					112.57		259.93

Active earth pressure	Horiz. force	281.43 kN/m	(= Pa * FOS)
	Vertical force	75.41 kN/m	(= Pa * tan $\delta$ * FOS)
	Moment	453.76 kNm/m	(= (Ma - V * B/2) * FOS)

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma' =$	13.90 kN/m <sup>3</sup>

		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	Wx (kNm/m)
Base	underwater	5.20	0.65	13.90	2.60	46.98	122.15
Wall 1	underwater	0.45	2.90	13.90	0.23	18.14	4.08
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	4.75	2.90	10.00	2.83	137.75	389.14
Soil 2	above water	4.75	0.75	18.00	2.83	64.13	181.15
Soil 3	above water	5.20	1.20	18.00	2.60	112.32	292.03
						387.42	990.39

Anchor plate	Vertical force	387.42 kN/m	(= W)
	Moment	16.90 kNm/m	(= W * B/2 - Wx)

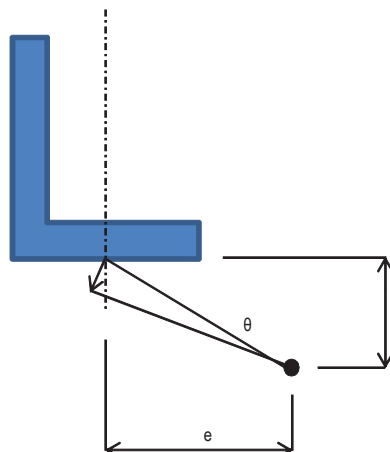
### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	KH =	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	KV =	10,000 kN/m <sup>3</sup> ... = 2000 N

Center of rotation

Center of rotation and rotation angle are assumed as follows:

Vertical distance	y	2.337 m
Horizontal distance	e	-0.513 m
Rotation angle	$\theta$	0.009497 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	5.50	0.00	7.84	0.00	744.33	0.00	0.00	5.50	0.00
	5.50		7.84	0.00	744.33	0.00	0.00	5.50	0.00
Plastic	5.50	1.20	7.84	0.00	744.33	0.00	0.00	5.10	0.00
	4.30		6.64	103.79	630.36	103.79	62.27	4.70	292.68
Plastic	4.30	0.00	6.64	103.79	630.36	103.79	0.00	4.30	0.00
	4.30		6.64	103.79	630.36	103.79	0.00	4.30	0.00
Plastic	4.30	0.75	6.64	103.79	630.36	103.79	38.92	4.05	157.63
	3.55		5.89	168.66	559.13	168.66	63.25	3.80	240.33
Plastic	3.55	2.73	5.89	168.66	559.13	168.66	230.23	2.64	607.79
	0.82		3.16	299.84	299.84	299.84	409.30	1.73	708.05
Elastic	0.82	0.82	3.16	299.84	299.84	299.84	122.91	0.55	67.18
	0.00		2.34	339.23	221.97	221.97	90.99	0.27	24.87
Below the top of Anchor Wall only							955.60		1,805.85

Passive side pressure	Horiz. force	955.60 kN/m	(= Pp')
	Vertical force	256.05 kN/m	(= Pp' * tan $\delta$ )
	Moment	2,471.59 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	198.18 kPa	P1	206.77 kN/m	M1	393.8 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				206.77		393.8
Effective contact area	A (A')	2.08668998 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	124.06 kN/m	(= $\mu * (P_1+P_2)$ )			

Base reaction force	Horiz. force	124.06 kN/m	(= Q)
	Vertical force	206.77 kN/m	(= P1+P2)
	Moment	393.78 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force (+) ←	Vert force (+) ↓	Moment (+) ↻
Active earth pressure	281.43	75.41	453.76
Weight of Plate	0.00	387.42	16.90
Tie tension	798.24	0.00	2,394.71
Passive soil pressure	-955.60	-256.05	-2,471.59
Base reaction	-124.06	-206.77	-393.78
Total	0.00	0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	2.337 m		
Horizontal distance	e	-0.513 m		
Rotation angle	$\theta$	0.0095 rad	<	Allowable rotation angle 0.02 rad OK
Horizontal deflection at tie	$\delta$	50.69 mm	--- no limitation	

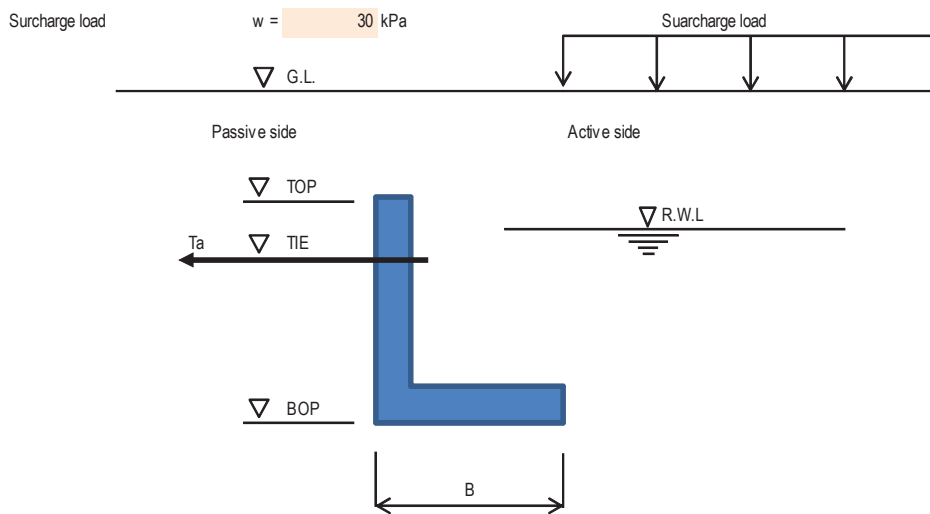
**Design of Anchor Plate (Elast-plastic method)**

**Block Block-1**

**Case : During Operation --- Tie tension FOS = 1.0 (Working tension load)**

**1. Design Conditions**

Ground level	GL	5.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.00 m
Width of anchor plate	B	5.20 m
Tie elevation	TIE	3.00 m
Tie tension force	Ta	319.30 kN/m
Factor of safety	FOS	1.0
	FOS×Ta =	319.295 kN/m
	M =	957.885 kNm/m



Soil conditions  
Passive side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_p \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.00	10	30	-15	4.805

Active side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_a \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.00	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_p \cos \delta$	pp (kPa)	Pp (kN/m)
5.50	1.20	0.00	4.805	0.00	0.00
4.30		21.60	4.805	103.79	62.27
4.30	0.75	21.60	4.805	103.79	38.92
3.55		35.10	4.805	168.66	63.25
3.55	3.55	35.10	4.805	168.66	299.36
0.00		70.60	4.805	339.23	602.14
					1,065.94

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.



Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	$M_a$ (kNm/m)
5.50	1.20	30.00	0.291	8.73	5.24	5.10	26.71
4.30		51.60	0.291	15.02	9.01	4.70	42.34
4.30	0.75	51.60	0.291	15.02	5.63	4.05	22.80
3.55		65.10	0.291	18.94	7.10	3.80	27.00
3.55	3.55	65.10	0.291	18.94	33.63	2.37	79.58
0.00		100.60	0.291	29.27	51.96	1.18	61.49
					112.57		259.93

Active earth pressure	Horiz. force	112.57 kN/m	(= $P_a * FOS$ )
	Vertical force	30.16 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	181.50 kNm/m	(= $(M_a - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

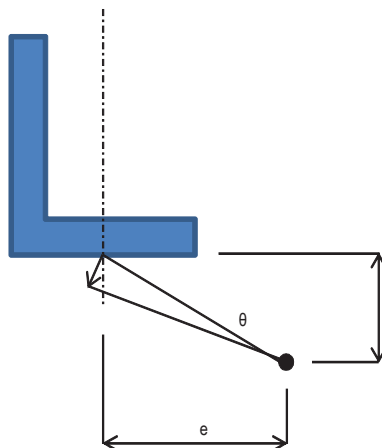
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	5.20	0.65	13.90	2.60	46.98	122.15
Wall 1	underwater	0.45	2.90	13.90	0.23	18.14	4.08
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	4.75	2.90	10.00	2.83	137.75	389.14
Soil 2	above water	4.75	0.75	18.00	2.83	64.13	181.15
Soil 3	above water	5.20	1.20	18.00	2.60	112.32	292.03
						387.42	990.39

Anchor plate	Vertical force	387.42 kN/m	(= W)
	Moment	16.90 kNm/m	(= $W * B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	-0.456 m
Horizontal distance	e	2.320 m
Rotation anglr	$\theta$	0.002972 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	5.50	0.00	5.04	0.00	149.92	0.00	0.00	5.50	0.00
	5.50		5.04	0.00	149.92	0.00	0.00	5.50	0.00
Plastic	5.50	1.20	5.04	0.00	149.92	0.00	0.00	5.10	0.00
	4.30		3.84	103.79	114.26	103.79	62.27	4.70	292.68
Plastic	4.30	0.09	3.84	103.79	114.26	103.79	4.67	4.27	19.96
	4.21		3.75	111.58	111.58	111.58	5.03	4.24	21.31
Elastic	4.21	0.66	3.75	111.58	111.58	111.58	36.82	3.99	146.90
	3.55		3.09	168.66	91.96	91.96	30.34	3.77	114.40
Elastic	3.55	0.00	3.09	168.66	91.96	91.96	0.00	3.55	0.00
	3.55		3.09	168.66	91.96	91.96	0.00	3.55	0.00
Elastic	3.55	3.55	3.09	168.66	91.96	91.96	163.24	2.37	386.33
	0.00		-0.46	339.23	-13.55	-13.55	-24.05	1.18	-28.45
Below the top of Anchor Wall only							216.05		660.44

Passive side pressure	Horiz. force	216.05 kN/m	(= Pp')
	Vertical force	57.89 kN/m	(= Pp' * tan $\delta$ )
	Moment	810.95 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	146.22 kPa	P1	359.69 kN/m	M1	345.3 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				359.69		345.3
Effective contact area	A (A')	4.919746208 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	215.81 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	215.81 kN/m	(= Q)
	Vertical force	359.69 kN/m	(= P1+P2)
	Moment	345.33 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force (+) ←	Vert force (+) ↓	Moment (+) ↻
Active earth pressure	112.57	30.16	181.50
Weight of Plate	0.00	387.42	16.90
Tie tension	319.30	0.00	957.89
Passive soil pressure	-216.05	-57.89	-810.95
Base reaction	-215.81	-359.69	-345.33
Total	-0.00	0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	-0.456 m			
Horizontal distance	e	2.320 m			
Rotation angle	$\theta$	0.0030 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	7.56 mm	<	Allowable deflection	30 mm OK

**Design of Anchor Plate (Elast-plastic method)**

**Block Block-1**

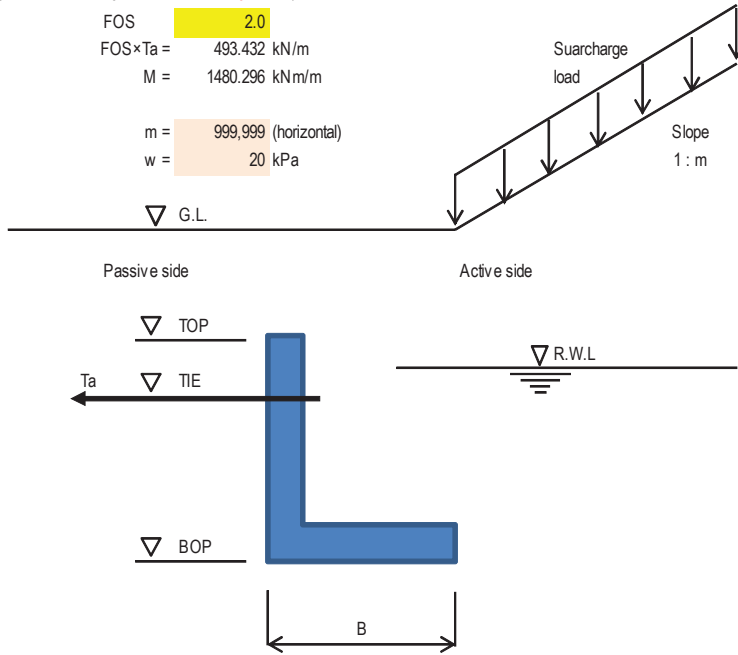
**Case : During Construction --- Tie tension FOS = 2.0**

**1. Design Conditions**

Ground level	GL	4.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.00 m
Width of anchor plate	B	5.20 m

Tie elevation	TIE	3.00 m
Tie tension force	Ta	246.72 kN/m
Factor of safety	FOS	2.0
	FOS × Ta =	493.432 kN/m
	M =	1480.296 kNm/m

Ground slope	m =	999,999 (horizontal)
Surcharge load	w =	20 kPa



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.00	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.00	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cosδ	pp (kPa)	Pp (kN/m)
4.50		0.00	4.805	0.00	0.00
4.30	0.20	3.60	4.805	17.30	1.73
4.30	0.75	3.60	4.805	17.30	6.49
3.55		17.10	4.805	82.17	30.81
3.55	3.55	17.10	4.805	82.17	145.84
0.00		52.60	4.805	252.74	448.62
					633.49

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	$M_a$ (kNm/m)
4.50	0.20	20.00	0.291	5.82	0.58	4.43	2.58
4.30		23.60	0.291	6.87	0.69	4.37	3.00
4.30	0.75	23.60	0.291	6.87	2.58	4.05	10.43
3.55		37.10	0.291	10.80	4.05	3.80	15.38
3.55	3.55	37.10	0.291	10.80	19.16	2.37	45.35
0.00		72.60	0.291	21.13	37.50	1.18	44.37
					64.56		121.12

Active earth pressure	Horiz. force	129.11 kN/m	(= $P_a * FOS$ )
	Vertical force	34.60 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	152.29 kNm/m	(= $(M_a - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

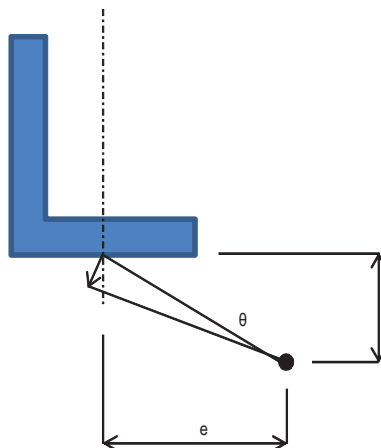
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	5.20	0.65	13.90	2.60	46.98	122.15
Wall 1	underwater	0.45	2.90	13.90	0.23	18.14	4.08
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	4.75	2.90	10.00	2.83	137.75	389.14
Soil 2	above water	4.75	0.75	18.00	2.83	64.13	181.15
Soil 3	above water	5.20	0.20	18.00	2.60	18.72	48.67
						293.82	747.03

Anchor plate	Vertical force	293.82 kN/m	(= W)
	Moment	16.90 kNm/m	(= $W * B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	1.430 m
Horizontal distance	e	-0.093 m
Rotation anglr	$\theta$	0.006125 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	4.50	0.00	5.93	0.00	363.25	0.00	0.00	4.50	0.00
	4.50		5.93	0.00	363.25	0.00	0.00	4.50	0.00
Plastic	4.50	0.20	5.93	0.00	363.25	0.00	0.00	4.43	0.00
	4.30		5.73	17.30	351.00	17.30	1.73	4.37	7.55
Plastic	4.30	0.00	5.73	17.30	351.00	17.30	0.00	4.30	0.00
	4.30		5.73	17.30	351.00	17.30	0.00	4.30	0.00
Plastic	4.30	0.75	5.73	17.30	351.00	17.30	6.49	4.05	26.27
	3.55		4.98	82.17	305.05	82.17	30.81	3.80	117.09
Plastic	3.55	2.04	4.98	82.17	305.05	82.17	83.77	2.87	240.45
	1.51		2.94	180.15	180.15	180.15	183.67	2.19	402.35
Elastic	1.51	1.51	2.94	180.15	180.15	180.15	136.09	1.01	137.07
	0.00		1.43	252.74	87.60	87.60	66.17	0.50	33.33
Below the top of Anchor Wall only							507.01		956.56

Passive side pressure	Horiz. force	507.01 kN/m	(= Pp')
	Vertical force	135.85 kN/m	(= Pp' * tan $\delta$ )
	Moment	1,309.77 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	153.59 kPa	P1	192.56 kN/m	M1	339.7 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				192.56		339.7
Effective contact area	A (A')	2.507421032 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	115.54 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	115.54 kN/m	(= Q)
	Vertical force	192.56 kN/m	(= P1+P2)
	Moment	339.71 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	129.11	34.60	152.29
Weight of Plate	0.00	293.82	16.90
Tie tension	493.43	0.00	1,480.30
Passive soil pressure	-507.01	-135.85	-1,309.77
Base reaction	-115.54	-192.56	-339.71
Total	-0.00	-0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	1.430 m			
Horizontal distance	e	-0.093 m			
Rotation angle	$\theta$	0.0061 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	27.14 mm	---	no limitation	

**Design of Anchor Plate (Elast-plastic method)**

**Block Block-1**

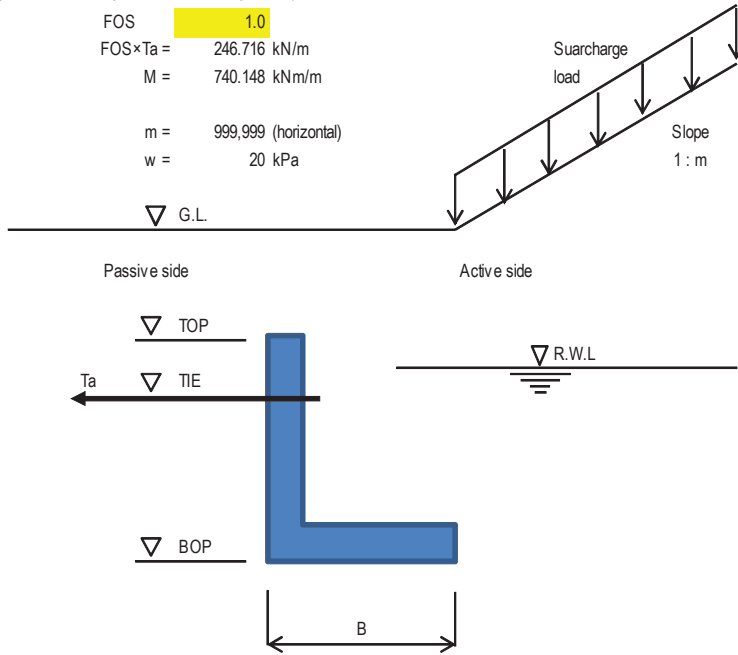
**Case : During Construction --- Tie tension FOS = 1.0 (Working tension load)**

**1. Design Conditions**

Ground level	GL	4.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.00 m
Width of anchor plate	B	5.20 m

Tie elevation	TIE	3.00 m
Tie tension force	Ta	246.72 kN/m
Factor of safety	FOS	1.0
	FOS × Ta =	246.716 kN/m
	M =	740.148 kNm/m

Ground slope	m =	999,999 (horizontal)
Surcharge load	w =	20 kPa



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.00	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.00	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cosδ	pp (kPa)	Pp (kN/m)
4.50		0.00	4.805	0.00	0.00
4.30	0.20	3.60	4.805	17.30	1.73
4.30	0.75	3.60	4.805	17.30	6.49
3.55		17.10	4.805	82.17	30.81
3.55	3.55	17.10	4.805	82.17	145.84
0.00		52.60	4.805	252.74	448.62
					633.49

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	$M_a$ (kNm/m)
4.50	0.20	20.00	0.291	5.82	0.58	4.43	2.58
4.30		23.60	0.291	6.87	0.69	4.37	3.00
4.30	0.75	23.60	0.291	6.87	2.58	4.05	10.43
3.55		37.10	0.291	10.80	4.05	3.80	15.38
3.55	3.55	37.10	0.291	10.80	19.16	2.37	45.35
0.00		72.60	0.291	21.13	37.50	1.18	44.37
					64.56		121.12

Active earth pressure	Horiz. force	64.56 kN/m	(= $P_a * FOS$ )
	Vertical force	17.30 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	76.15 kNm/m	(= $(M_a - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

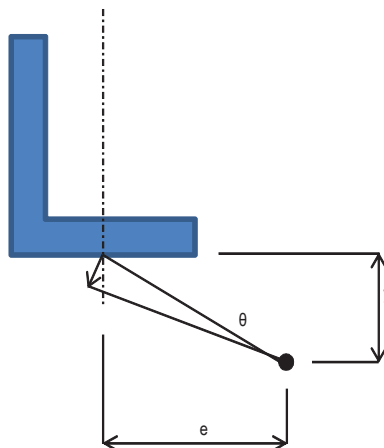
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	5.20	0.65	13.90	2.60	46.98	122.15
Wall 1	underwater	0.45	2.90	13.90	0.23	18.14	4.08
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	4.75	2.90	10.00	2.83	137.75	389.14
Soil 2	above water	4.75	0.75	18.00	2.83	64.13	181.15
Soil 3	above water	5.20	0.20	18.00	2.60	18.72	48.67
						293.82	747.03

Anchor plate	Vertical force	293.82 kN/m	(= W)
	Moment	16.90 kNm/m	(= $W * B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	-0.570 m
Horizontal distance	e	1.968 m
Rotation anglr	$\theta$	0.002600 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	4.50	0.00	3.93	0.00	102.20	0.00	0.00	4.50	0.00
	4.50		3.93	0.00	102.20	0.00	0.00	4.50	0.00
Plastic	4.50	0.20	3.93	0.00	102.20	0.00	0.00	4.43	0.00
	4.30		3.73	17.30	97.00	17.30	1.73	4.37	7.55
Plastic	4.30	0.71	3.73	17.30	97.00	17.30	6.13	4.06	24.90
	3.59		3.02	78.58	78.58	78.58	27.84	3.83	106.55
Elastic	3.59	0.04	3.02	78.58	78.58	78.58	1.63	3.58	5.83
	3.55		2.98	82.17	77.50	77.50	1.61	3.56	5.73
Elastic	3.55	0.00	2.98	82.17	77.50	77.50	0.00	3.55	0.00
	3.55		2.98	82.17	77.50	77.50	0.00	3.55	0.00
Elastic	3.55	3.55	2.98	82.17	77.50	77.50	137.56	2.37	325.56
	0.00		-0.57	252.74	-14.81	-14.81	-26.29	1.18	-31.11
Below the top of Anchor Wall only							148.47		437.46

Passive side pressure	Horiz. force	148.47 kN/m	(= Pp')
	Vertical force	39.78 kN/m	(= Pp' * tan $\delta$ )
	Moment	540.90 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	118.79 kPa	P1	271.33 kN/m	M1	292.3 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				271.33		292.3
Effective contact area	A (A')	4.568235997 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	162.80 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	162.80 kN/m	(= Q)
	Vertical force	271.33 kN/m	(= P1+P2)
	Moment	292.29 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	64.56	17.30	76.15
Weight of Plate	0.00	293.82	16.90
Tie tension	246.72	0.00	740.15
Passive soil pressure	-148.47	-39.78	-540.90
Base reaction	-162.80	-271.33	-292.29
Total	0.00	0.00	-0.00

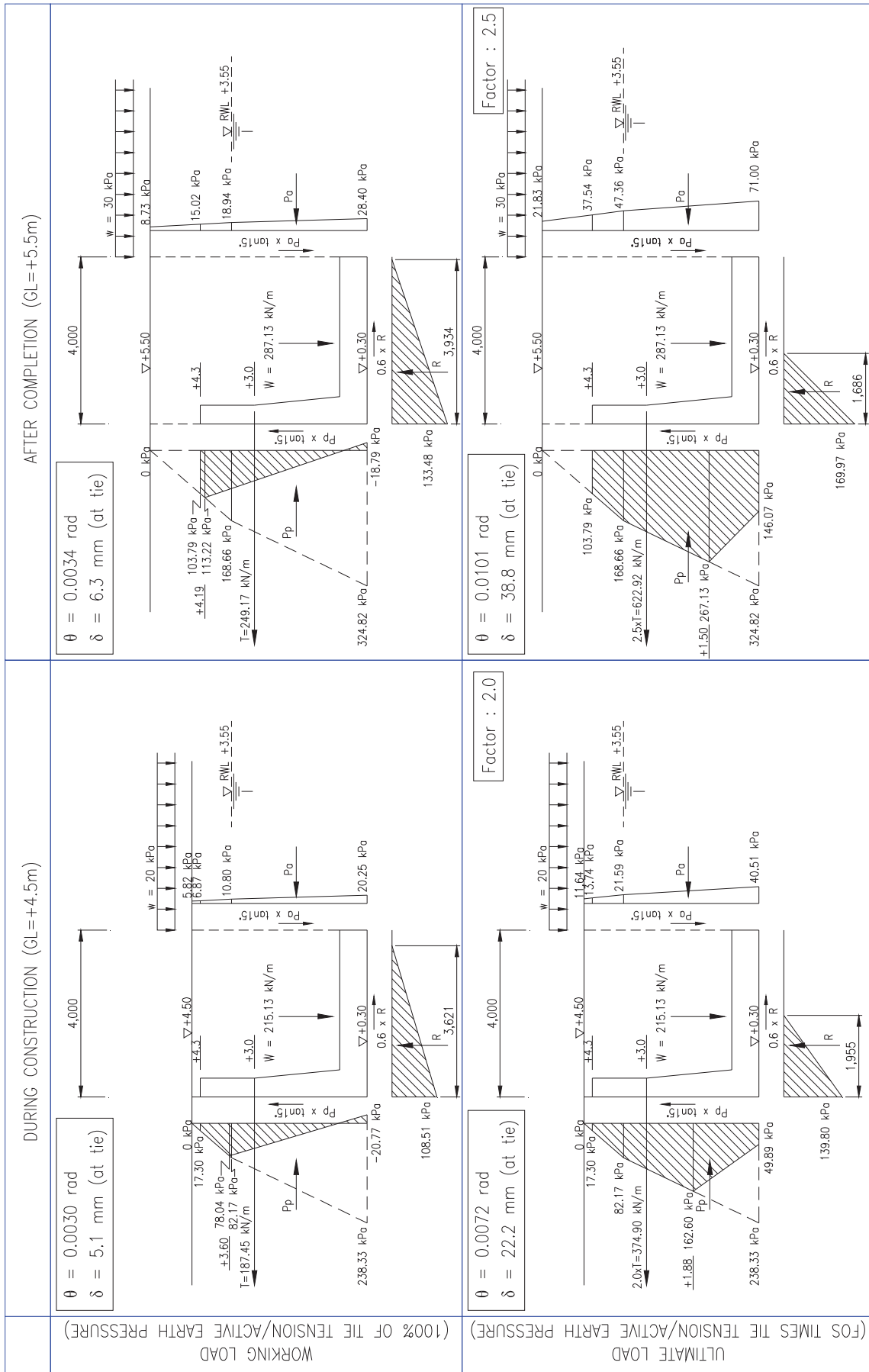
<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	-0.570 m			
Horizontal distance	e	1.968 m			
Rotation angle	$\theta$	0.0026 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	6.32 mm	<	Allowable deflection	30 mm OK



# Summary of Analysis (Block-a)



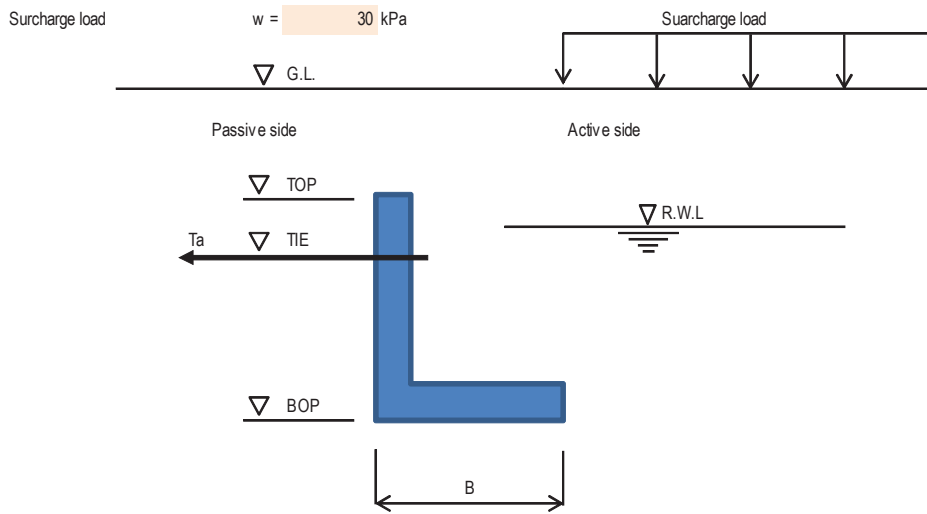
**Design of Anchor Plate (Elast-plastic method)**

**Block Block-a**

**Case : During Operation --- Tie tension FOS = 2.5**

**1. Design Conditions**

Ground level	GL	5.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.00 m
Tie elevation	TIE	3.00 m
Tie tension force	Ta	249.17 kN/m
Factor of safety	FOS	2.5
	FOS×Ta =	622.915 kN/m
	M =	1681.8705 kNm/m



Soil conditions

Passive side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_p \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_a \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_p \cos \delta$	pp (kPa)	Pp (kN/m)
5.50		0.00	4.805	0.00	0.00
4.30	1.20	21.60	4.805	103.79	62.27
4.30	0.75	21.60	4.805	103.79	38.92
3.55		35.10	4.805	168.66	63.25
3.55	3.25	35.10	4.805	168.66	274.07
0.30		67.60	4.805	324.82	527.83
					966.33

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	$y$ (m)	$Ma$ (kNm/m)
5.50	1.20	30.00	0.291	8.73	5.24	4.80	25.14
4.30		51.60	0.291	15.02	9.01	4.40	39.64
4.30	0.75	51.60	0.291	15.02	5.63	3.75	21.12
3.55		65.10	0.291	18.94	7.10	3.50	24.86
3.55	3.25	65.10	0.291	18.94	30.78	2.17	66.70
0.30		97.60	0.291	28.40	46.15	1.08	50.00
					103.92		227.46

Active earth pressure	Horiz. force	259.80 kN/m	(= $P_a * FOS$ )
	Vertical force	69.61 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	429.43 kNm/m	(= $(Ma - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

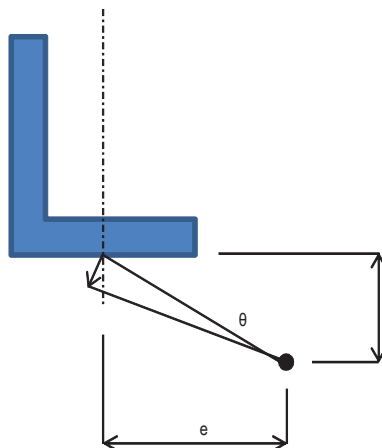
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	4.00	0.65	13.90	2.00	36.14	72.28
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.55	2.60	10.00	2.23	92.30	205.37
Soil 2	above water	3.55	0.75	18.00	2.23	47.93	106.63
Soil 3	above water	4.00	1.20	18.00	2.00	86.40	172.80
						287.13	562.56

Anchor plate	Vertical force	287.13 kN/m	(= W)
	Moment	11.69 kNm/m	(= $W * B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	1.149 m
Horizontal distance	e	-0.314 m
Rotation anglr	$\theta$	0.010084 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	5.50	0.00	6.65	0.00	670.43	0.00	0.00	5.20	0.00
	5.50		6.65	0.00	670.43	0.00	0.00	5.20	0.00
Plastic	5.50	1.20	6.65	0.00	670.43	0.00	0.00	4.80	0.00
	4.30		5.45	103.79	549.43	103.79	62.27	4.40	274.00
Plastic	4.30	0.00	5.45	103.79	549.43	103.79	0.00	4.00	0.00
	4.30		5.45	103.79	549.43	103.79	0.00	4.00	0.00
Plastic	4.30	0.75	5.45	103.79	549.43	103.79	38.92	3.75	145.95
	3.55		4.70	168.66	473.80	168.66	63.25	3.50	221.36
Plastic	3.55	2.05	4.70	168.66	473.80	168.66	172.83	2.57	443.62
	1.50		2.65	267.13	267.13	267.13	273.74	1.88	515.64
Elastic	1.50	1.20	2.65	267.13	267.13	267.13	160.35	0.80	128.34
	0.30		1.45	324.82	146.07	146.07	87.68	0.40	35.09
Below the top of Anchor Wall only							796.76		1,490.00

Passive side pressure	Horiz. force	796.76 kN/m	(= Pp')
	Vertical force	213.49 kN/m	(= Pp' * tan $\delta$ )
	Moment	1,916.98 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	169.97 kPa	P1	143.25 kN/m	M1	206.0 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				143.25		206.0
Effective contact area	A (A')	1.685558347 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	85.95 kN/m	(= $\mu * (P_1+P_2)$ )			

Base reaction force	Horiz. force	85.95 kN/m	(= Q)
	Vertical force	143.25 kN/m	(= P1+P2)
	Moment	206.01 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	259.80	69.61	429.43
Weight of Plate	0.00	287.13	11.69
Tie tension	622.92	0.00	1,681.87
Passive soil pressure	-796.76	-213.49	-1,916.98
Base reaction	-85.95	-143.25	-206.01
Total	0.00	0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	1.149 m		
Horizontal distance	e	-0.314 m		
Rotation angle	$\theta$	0.0101 rad	<	Allowable rotation angle 0.02 rad OK
Horizontal deflection at tie	$\delta$	38.81 mm	---	no limitation

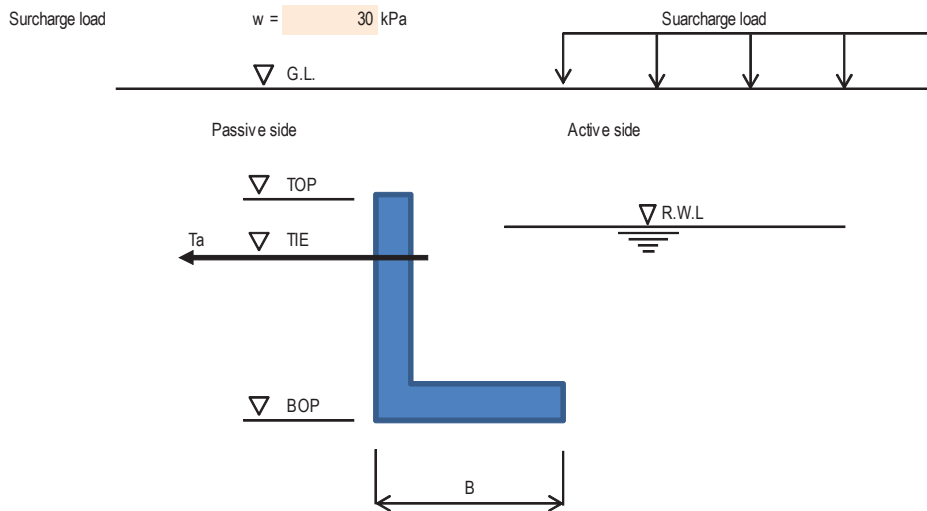
**Design of Anchor Plate (Elast-plastic method)**

**Block Block-a**

**Case : During Operation --- Tie tension FOS = 1.0 (Working tension load)**

**1. Design Conditions**

Ground level	GL	5.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.00 m
Tie elevation	TIE	3.00 m
Tie tension force	Ta	249.17 kN/m
Factor of safety	FOS	1.0
	FOS×Ta =	249.166 kN/m
	M =	672.7482 kNm/m



Soil conditions

Passive side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_p \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_a \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_p \cos \delta$	pp (kPa)	Pp (kN/m)
5.50		0.00	4.805	0.00	0.00
4.30	1.20	21.60	4.805	103.79	62.27
4.30	0.75	21.60	4.805	103.79	38.92
3.55		35.10	4.805	168.66	63.25
3.55	3.25	35.10	4.805	168.66	274.07
0.30		67.60	4.805	324.82	527.83
					966.33

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	$M_a$ (kNm/m)
5.50	1.20	30.00	0.291	8.73	5.24	4.80	25.14
4.30		51.60	0.291	15.02	9.01	4.40	39.64
4.30	0.75	51.60	0.291	15.02	5.63	3.75	21.12
3.55		65.10	0.291	18.94	7.10	3.50	24.86
3.55	3.25	65.10	0.291	18.94	30.78	2.17	66.70
0.30		97.60	0.291	28.40	46.15	1.08	50.00
					103.92		227.46

Active earth pressure	Horiz. force	103.92 kN/m	(= $P_a \cdot FOS$ )
	Vertical force	27.85 kN/m	(= $P_a \cdot \tan \delta \cdot FOS$ )
	Moment	171.77 kNm/m	(= $(M_a - V \cdot B/2) \cdot FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

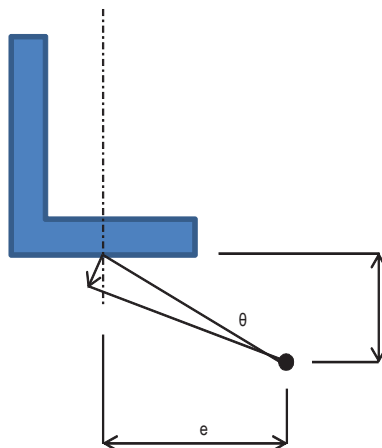
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	4.00	0.65	13.90	2.00	36.14	72.28
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.55	2.60	10.00	2.23	92.30	205.37
Soil 2	above water	3.55	0.75	18.00	2.23	47.93	106.63
Soil 3	above water	4.00	1.20	18.00	2.00	86.40	172.80
						287.13	562.56

Anchor plate	Vertical force	287.13 kN/m	(= W)
	Moment	11.69 kNm/m	(= $W \cdot B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	-0.854 m
Horizontal distance	e	1.934 m
Rotation anglr	$\theta$	0.003393 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	5.50	0.00	4.65	0.00	157.63	0.00	0.00	5.20	0.00
	5.50		4.65	0.00	157.63	0.00	0.00	5.20	0.00
Plastic	5.50	1.20	4.65	0.00	157.63	0.00	0.00	4.80	0.00
	4.30		3.45	103.79	116.92	103.79	62.27	4.40	274.00
Plastic	4.30	0.11	3.45	103.79	116.92	103.79	5.66	3.96	22.43
	4.19		3.34	113.22	113.22	113.22	6.17	3.93	24.24
Elastic	4.19	0.64	3.34	113.22	113.22	113.22	36.28	3.68	133.43
	3.55		2.70	168.66	91.47	91.47	29.31	3.46	101.54
Elastic	3.55	0.00	2.70	168.66	91.47	91.47	0.00	3.25	0.00
	3.55		2.70	168.66	91.47	91.47	0.00	3.25	0.00
Elastic	3.55	3.25	2.70	168.66	91.47	91.47	148.64	2.17	322.06
	0.30		-0.55	324.82	-18.79	-18.79	-30.54	1.08	-33.08
Below the top of Anchor Wall only							195.54		570.62

Passive side pressure	Horiz. force	195.54 kN/m	(= Pp')
	Vertical force	52.39 kN/m	(= Pp' * tan $\delta$ )
	Moment	675.41 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	133.48 kPa	P1	262.58 kN/m	M1	180.8 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				262.58		180.8
Effective contact area	A (A')	3.934273112 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	157.55 kN/m	(= $\mu * (P_1+P_2)$ )			

Base reaction force	Horiz. force	157.55 kN/m	(= Q)
	Vertical force	262.58 kN/m	(= P1+P2)
	Moment	180.81 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	103.92	27.85	171.77
Weight of Plate	0.00	287.13	11.69
Tie tension	249.17	0.00	672.75
Passive soil pressure	-195.54	-52.39	-675.41
Base reaction	-157.55	-262.58	-180.81
Total	0.00	0.00	0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	-0.854 m			
Horizontal distance	e	1.934 m			
Rotation angle	$\theta$	0.0034 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	6.26 mm	<	Allowable deflection	30 mm OK

**Design of Anchor Plate (Elast-plastic method)**

**Block Block-a**

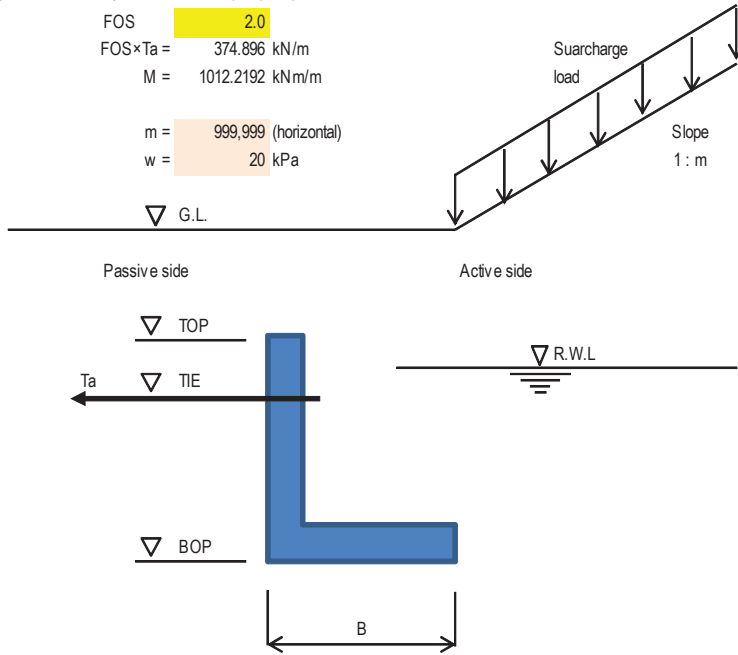
**Case : During Construction --- Tie tension FOS = 2.0**

**1. Design Conditions**

Ground level	GL	4.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.00 m

Tie elevation	TIE	3.00 m
Tie tension force	Ta	187.45 kN/m
Factor of safety	FOS	2.0
	FOS × Ta =	374.896 kN/m
	M =	1012.2192 kNm/m

Ground slope	m =	999,999 (horizontal)
Surcharge load	w =	20 kPa



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cosδ	pp (kPa)	Pp (kN/m)
4.50		0.00	4.805	0.00	0.00
4.30	0.20	3.60	4.805	17.30	1.73
4.30	0.75	3.60	4.805	17.30	6.49
3.55		17.10	4.805	82.17	30.81
3.55	3.25	17.10	4.805	82.17	133.52
0.30		49.60	4.805	238.33	387.28
					559.83

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.



Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	Ma (kNm/m)
4.50	0.20	20.00	0.291	5.82	0.58	4.13	2.41
4.30		23.60	0.291	6.87	0.69	4.07	2.79
4.30	0.75	23.60	0.291	6.87	2.58	3.75	9.66
3.55		37.10	0.291	10.80	4.05	3.50	14.17
3.55	3.25	37.10	0.291	10.80	17.54	2.17	38.01
0.30		69.60	0.291	20.25	32.91	1.08	35.65
					58.35		102.69

Active earth pressure	Horiz. force	116.70 kN/m	(= $P_a * FOS$ )
	Vertical force	31.27 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	142.85 kNm/m	(= $(Ma - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

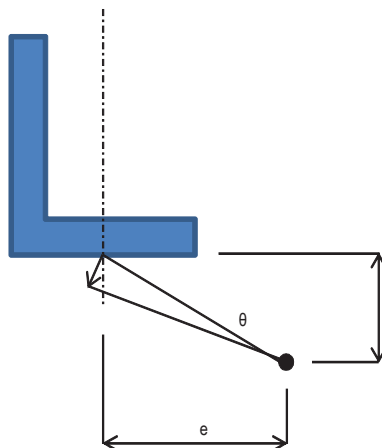
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	Wx (kNm/m)
Base	underwater	4.00	0.65	13.90	2.00	36.14	72.28
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.55	2.60	10.00	2.23	92.30	205.37
Soil 2	above water	3.55	0.75	18.00	2.23	47.93	106.63
Soil 3	above water	4.00	0.20	18.00	2.00	14.40	28.80
						215.13	418.56

Anchor plate	Vertical force	215.13 kN/m	(= W)
	Moment	11.69 kNm/m	(= $W * B/2 - Wx$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	KH =	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	KV =	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	0.398 m
Horizontal distance	e	-0.045 m
Rotation anglr	$\theta$	0.007151 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	4.50	0.00	4.90	0.00	350.24	0.00	0.00	4.20	0.00
	4.50		4.90	0.00	350.24	0.00	0.00	4.20	0.00
Plastic	4.50	0.20	4.90	0.00	350.24	0.00	0.00	4.13	0.00
	4.30		4.70	17.30	335.94	17.30	1.73	4.07	7.03
Plastic	4.30	0.00	4.70	17.30	335.94	17.30	0.00	4.00	0.00
	4.30		4.70	17.30	335.94	17.30	0.00	4.00	0.00
Plastic	4.30	0.75	4.70	17.30	335.94	17.30	6.49	3.75	24.33
	3.55		3.95	82.17	282.31	82.17	30.81	3.50	107.84
Plastic	3.55	1.67	3.95	82.17	282.31	82.17	68.77	2.69	185.13
	1.88		2.27	162.60	162.60	162.60	136.09	2.13	290.42
Elastic	1.88	1.58	2.27	162.60	162.60	162.60	128.13	1.05	134.63
	0.30		0.70	238.33	49.89	49.89	39.32	0.53	20.65
Below the top of Anchor Wall only							409.61		763.00

Passive side pressure	Horiz. force	409.61 kN/m	(= Pp')
	Vertical force	109.75 kN/m	(= Pp' * tan $\delta$ )
	Moment	982.51 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	139.80 kPa	P1	136.64 kN/m	M1	184.2 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				136.64		184.2
Effective contact area	A (A')	1.954867447 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	81.99 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	81.99 kN/m	(= Q)
	Vertical force	136.64 kN/m	(= P1+P2)
	Moment	184.25 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force (+) ←	Vert force (+) ↓	Moment (+) ↻
Active earth pressure	116.70	31.27	142.85
Weight of Plate	0.00	215.13	11.69
Tie tension	374.90	0.00	1,012.22
Passive soil pressure	-409.61	-109.75	-982.51
Base reaction	-81.99	-136.64	-184.25
Total	-0.00	-0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	0.398 m		
Horizontal distance	e	-0.045 m		
Rotation angle	$\theta$	0.0072 rad	<	Allowable rotation angle 0.02 rad OK
Horizontal deflection at tie	$\delta$	22.15 mm	--- no limitation	

**Design of Anchor Plate (Elast-plastic method)**

**Block Block-a**

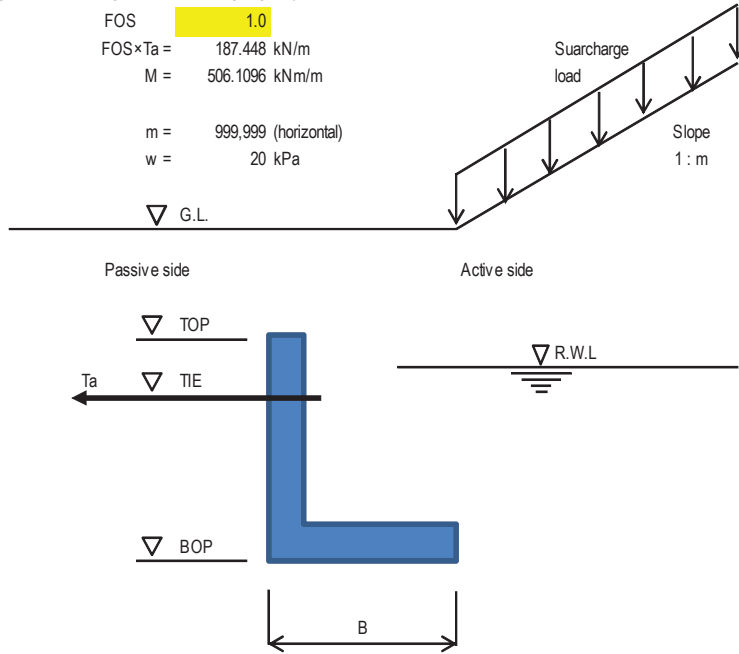
**Case : During Construction --- Tie tension FOS = 1.0 (Working tension load)**

**1. Design Conditions**

Ground level	GL	4.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.00 m

Tie elevation	TIE	3.00 m
Tie tension force	Ta	187.45 kN/m
Factor of safety	FOS	<b>1.0</b>
	FOS×Ta =	187.448 kN/m
	M =	506.1096 kNm/m

Ground slope	m =	999,999 (horizontal)
Surcharge load	w =	20 kPa



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cosδ	pp (kPa)	Pp (kN/m)
4.50	0.20	0.00	4.805	0.00	0.00
4.30		3.60	4.805	17.30	1.73
4.30	0.75	3.60	4.805	17.30	6.49
3.55		17.10	4.805	82.17	30.81
3.55	3.25	17.10	4.805	82.17	133.52
0.30		49.60	4.805	238.33	387.28
					559.83

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	$y$ (m)	$M_a$ (kNm/m)
4.50	0.20	20.00	0.291	5.82	0.58	4.13	2.41
4.30		23.60	0.291	6.87	0.69	4.07	2.79
4.30	0.75	23.60	0.291	6.87	2.58	3.75	9.66
3.55		37.10	0.291	10.80	4.05	3.50	14.17
3.55	3.25	37.10	0.291	10.80	17.54	2.17	38.01
0.30		69.60	0.291	20.25	32.91	1.08	35.65
					58.35		102.69

Active earth pressure	Horiz. force	58.35 kN/m	(= $P_a * FOS$ )
	Vertical force	15.63 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	71.42 kNm/m	(= $(M_a - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

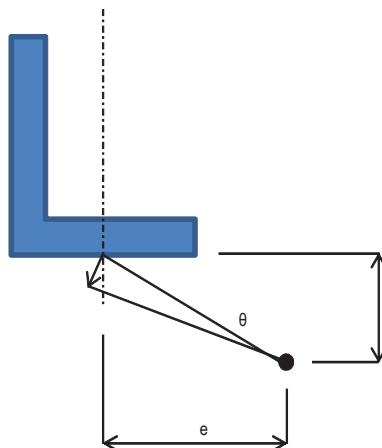
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	4.00	0.65	13.90	2.00	36.14	72.28
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.55	2.60	10.00	2.23	92.30	205.37
Soil 2	above water	3.55	0.75	18.00	2.23	47.93	106.63
Soil 3	above water	4.00	0.20	18.00	2.00	14.40	28.80
						215.13	418.56

Anchor plate	Vertical force	215.13 kN/m	(= W)
	Moment	11.69 kNm/m	(= $W * B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	-0.993 m
Horizontal distance	e	1.621 m
Rotation anglr	$\theta$	0.002996 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	4.50	0.00	3.51	0.00	105.08	0.00	0.00	4.20	0.00
	4.50		3.51	0.00	105.08	0.00	0.00	4.20	0.00
Plastic	4.30	0.20	3.31	17.30	99.08	17.30	1.73	4.07	7.03
	4.30		3.31	17.30	99.08	17.30	6.07	3.77	22.87
Plastic	3.60	0.70	2.60	78.04	78.04	78.04	27.40	3.53	96.78
	3.60		2.60	78.04	78.04	78.04	1.86	3.28	6.11
Elastic	3.55	0.05	2.56	82.17	76.61	76.61	1.83	3.27	5.97
Elastic	3.55	0.00	2.56	82.17	76.61	76.61	0.00	3.25	0.00
	3.55		2.56	82.17	76.61	76.61	0.00	3.25	0.00
Elastic	3.55	3.25	2.56	82.17	76.61	76.61	124.49	2.17	269.73
	0.30		-0.69	238.33	-20.77	-20.77	-33.76	1.08	-36.57
Below the top of Anchor Wall only							127.90		364.90

Passive side pressure	Horiz. force	127.90 kN/m	(= Pp')
	Vertical force	34.27 kN/m	(= Pp' * tan $\delta$ )
	Moment	433.44 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	108.51 kPa	P1	196.49 kN/m	M1	155.8 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				196.49		155.8
Effective contact area	A (A')	3.621468697 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	117.89 kN/m	(= $\mu * (P_1+P_2)$ )			

Base reaction force	Horiz. force	117.89 kN/m	(= Q)
	Vertical force	196.49 kN/m	(= P1+P2)
	Moment	155.79 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force (+) ←	Vert force (+) ↓	Moment (+) ↻
Active earth pressure	58.35	15.63	71.42
Weight of Plate	0.00	215.13	11.69
Tie tension	187.45	0.00	506.11
Passive soil pressure	-127.90	-34.27	-433.44
Base reaction	-117.89	-196.49	-155.79
Total	0.00	0.00	0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	-0.993 m			
Horizontal distance	e	1.621 m			
Rotation angle	$\theta$	0.0030 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	5.11 mm	<	Allowable deflection	30 mm OK

# Summary of Analysis (Block-b)



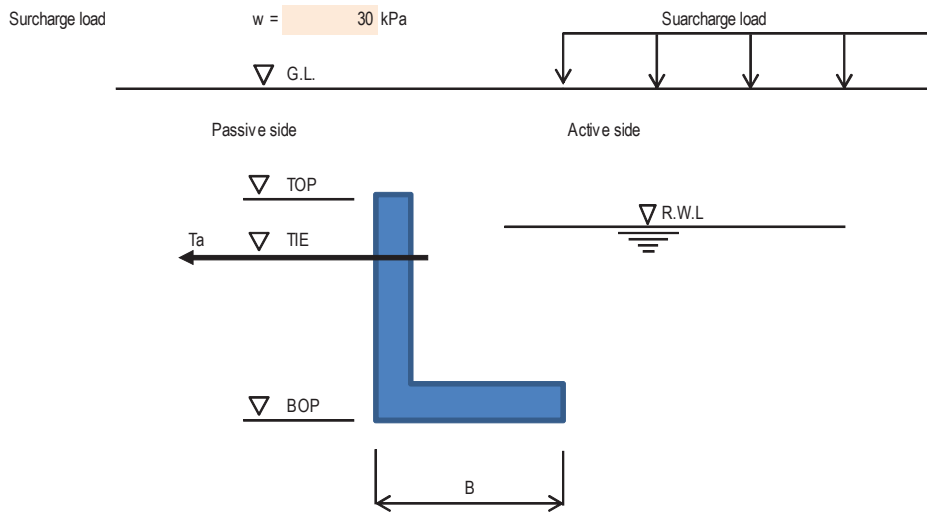
**Design of Anchor Plate (Elast-plastic method)**

**Block Block-b**

**Case : During Operation --- Tie tension FOS = 2.5**

**1. Design Conditions**

Ground level	GL	5.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.20 m
Tie elevation	TIE	3.00 m
Tie tension force	Ta	264.27 kN/m
Factor of safety	FOS	2.5
	FOS×Ta =	660.665 kN/m
	M =	1783.7955 kNm/m



Soil conditions

Passive side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_p \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_a \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_p \cos \delta$	pp (kPa)	Pp (kN/m)
5.50		0.00	4.805	0.00	0.00
4.30	1.20	21.60	4.805	103.79	62.27
4.30	0.75	21.60	4.805	103.79	38.92
3.55		35.10	4.805	168.66	63.25
3.55	3.25	35.10	4.805	168.66	274.07
0.30		67.60	4.805	324.82	527.83
					966.33

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	$y$ (m)	$Ma$ (kNm/m)
5.50	1.20	30.00	0.291	8.73	5.24	4.80	25.14
4.30		51.60	0.291	15.02	9.01	4.40	39.64
4.30	0.75	51.60	0.291	15.02	5.63	3.75	21.12
3.55		65.10	0.291	18.94	7.10	3.50	24.86
3.55	3.25	65.10	0.291	18.94	30.78	2.17	66.70
0.30		97.60	0.291	28.40	46.15	1.08	50.00
					103.92		227.46

Active earth pressure	Horiz. force	259.80 kN/m	(= $P_a * FOS$ )
	Vertical force	69.61 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	422.47 kNm/m	(= $(Ma - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

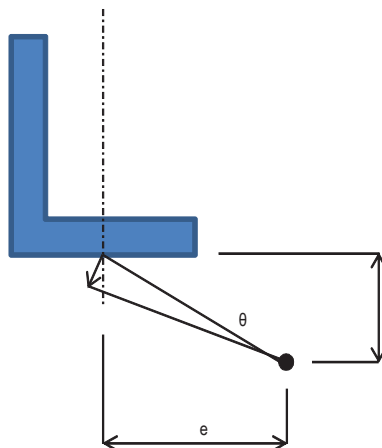
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	4.20	0.65	13.90	2.10	37.95	79.69
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.75	2.60	10.00	2.33	97.50	226.69
Soil 2	above water	3.75	0.75	18.00	2.33	50.63	117.70
Soil 3	above water	4.20	1.20	18.00	2.10	90.72	190.51
						301.16	620.07

Anchor plate	Vertical force	301.16 kN/m	(= $W$ )
	Moment	12.35 kNm/m	(= $W * B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	0.846 m
Horizontal distance	e	-0.645 m
Rotation anglr	$\theta$	0.013965 rad





## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	5.50	0.00	6.35	0.00	886.18	0.00	0.00	5.20	0.00
	5.50		6.35	0.00	886.18	0.00	0.00	5.20	0.00
Plastic	5.50	1.20	6.35	0.00	886.18	0.00	0.00	4.80	0.00
	4.30		5.15	103.79	718.60	103.79	62.27	4.40	274.00
Plastic	4.30	0.00	5.15	103.79	718.60	103.79	0.00	4.00	0.00
	4.30		5.15	103.79	718.60	103.79	0.00	4.00	0.00
Plastic	4.30	0.75	5.15	103.79	718.60	103.79	38.92	3.75	145.95
	3.55		4.40	168.66	613.87	168.66	63.25	3.50	221.36
Plastic	3.55	2.37	4.40	168.66	613.87	168.66	200.03	2.46	491.93
	1.18		2.02	282.63	282.63	282.63	335.20	1.67	559.34
Elastic	1.18	0.88	2.02	282.63	282.63	282.63	124.07	0.59	72.62
	0.30		1.15	324.82	160.02	160.02	70.25	0.29	20.56
Below the top of Anchor Wall only							831.72		1,511.76

Passive side pressure	Horiz. force	831.72 kN/m	(= Pp')
	Vertical force	222.86 kN/m	(= Pp' * tan $\delta$ )
	Moment	1,979.76 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	203.25 kPa	P1	147.91 kN/m	M1	238.9 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				147.91		238.9
Effective contact area	A (A')	1.455455519 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	88.75 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	88.75 kN/m	(= Q)
	Vertical force	147.91 kN/m	(= P1+P2)
	Moment	238.85 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	259.80	69.61	422.47
Weight of Plate	0.00	301.16	12.35
Tie tension	660.67	0.00	1,783.80
Passive soil pressure	-831.72	-222.86	-1,979.76
Base reaction	-88.75	-147.91	-238.85
Total	0.00	0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	0.846 m		
Horizontal distance	e	-0.645 m		
Rotation angle	$\theta$	0.0140 rad	<	Allowable rotation angle 0.02 rad OK
Horizontal deflection at tie	$\delta$	49.52 mm	--- no limitation	

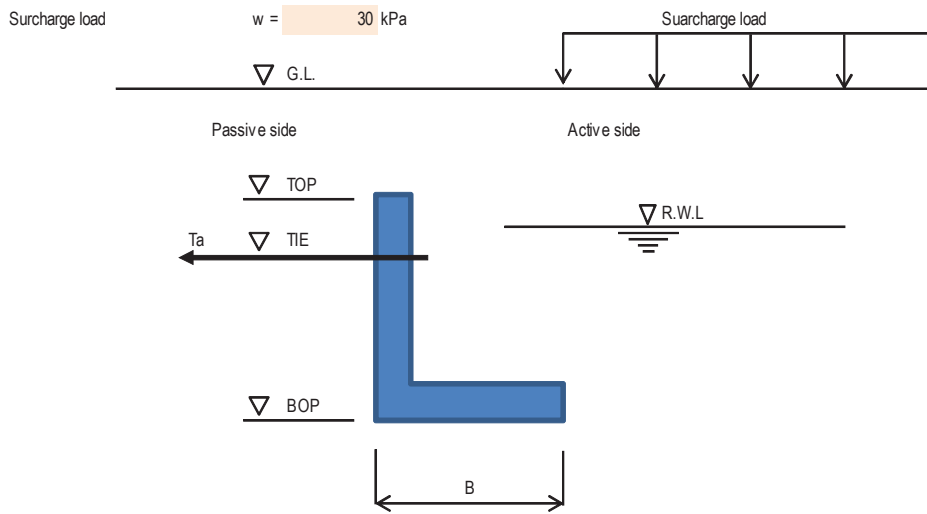
**Design of Anchor Plate (Elast-plastic method)**

**Block Block-b**

**Case : During Operation --- Tie tension FOS = 1.0 (Working tension load)**

**1. Design Conditions**

Ground level	GL	5.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.20 m
Tie elevation	TIE	3.00 m
Tie tension force	Ta	264.27 kN/m
Factor of safety	FOS	1.0
	FOS × Ta =	264.266 kN/m
	M =	713.5182 kNm/m



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cosδ
	Top	Bottom				
layer-1	5.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cosδ
	Top	Bottom				
layer-1	5.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cosδ	pp (kPa)	Pp (kN/m)
5.50		0.00	4.805	0.00	0.00
4.30	1.20	21.60	4.805	103.79	62.27
4.30	0.75	21.60	4.805	103.79	38.92
3.55		35.10	4.805	168.66	63.25
3.55	3.25	35.10	4.805	168.66	274.07
0.30		67.60	4.805	324.82	527.83
					966.33

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	$M_a$ (kNm/m)
5.50	1.20	30.00	0.291	8.73	5.24	4.80	25.14
4.30		51.60	0.291	15.02	9.01	4.40	39.64
4.30	0.75	51.60	0.291	15.02	5.63	3.75	21.12
3.55		65.10	0.291	18.94	7.10	3.50	24.86
3.55	3.25	65.10	0.291	18.94	30.78	2.17	66.70
0.30		97.60	0.291	28.40	46.15	1.08	50.00
					103.92		227.46

Active earth pressure	Horiz. force	103.92 kN/m	(= $P_a * FOS$ )
	Vertical force	27.85 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	168.99 kNm/m	(= $(M_a - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

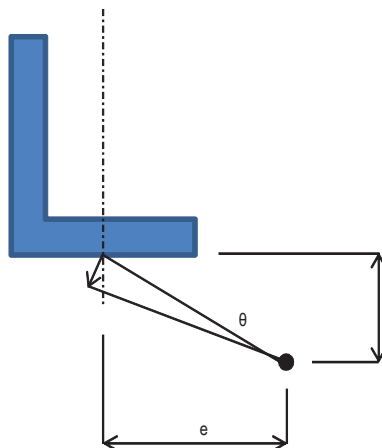
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	4.20	0.65	13.90	2.10	37.95	79.69
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.75	2.60	10.00	2.33	97.50	226.69
Soil 2	above water	3.75	0.75	18.00	2.33	50.63	117.70
Soil 3	above water	4.20	1.20	18.00	2.10	90.72	190.51
						301.16	620.07

Anchor plate	Vertical force	301.16 kN/m	(= W)
	Moment	12.35 kNm/m	(= $W * B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	-0.734 m
Horizontal distance	e	2.005 m
Rotation anglr	$\theta$	0.003258 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	5.50	0.00	4.77	0.00	155.30	0.00	0.00	5.20	0.00
	5.50		4.77	0.00	155.30	0.00	0.00	5.20	0.00
Plastic	5.50	1.20	4.77	0.00	155.30	0.00	0.00	4.80	0.00
	4.30		3.57	103.79	116.20	103.79	62.27	4.40	274.00
Plastic	4.30	0.10	3.57	103.79	116.20	103.79	5.41	3.97	21.45
	4.20		3.46	112.80	112.80	112.80	5.88	3.93	23.11
Elastic	4.20	0.65	3.46	112.80	112.80	112.80	36.42	3.68	134.05
	3.55		2.82	168.66	91.76	91.76	29.63	3.47	102.67
Elastic	3.55	0.00	2.82	168.66	91.76	91.76	0.00	3.25	0.00
	3.55		2.82	168.66	91.76	91.76	0.00	3.25	0.00
Elastic	3.55	3.25	2.82	168.66	91.76	91.76	149.12	2.17	323.08
	0.30		-0.43	324.82	-14.13	-14.13	-22.95	1.08	-24.87
Below the top of Anchor Wall only							203.50		579.50

Passive side pressure	Horiz. force	203.50 kN/m	(= Pp')
	Vertical force	54.53 kN/m	(= Pp' * tan $\delta$ )
	Moment	694.01 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	133.74 kPa	P1	274.47 kN/m	M1	200.8 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				274.47		200.8
Effective contact area	A (A')	4.104694112 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	164.68 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	164.68 kN/m	(= Q)
	Vertical force	274.47 kN/m	(= P1+P2)
	Moment	200.85 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	103.92	27.85	168.99
Weight of Plate	0.00	301.16	12.35
Tie tension	264.27	0.00	713.52
Passive soil pressure	-203.50	-54.53	-694.01
Base reaction	-164.68	-274.47	-200.85
Total	0.00	0.00	0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	-0.734 m			
Horizontal distance	e	2.005 m			
Rotation angle	$\theta$	0.0033 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	6.41 mm	<	Allowable deflection	30 mm OK

**Design of Anchor Plate (Elast-plastic method)**

**Block Block-b**

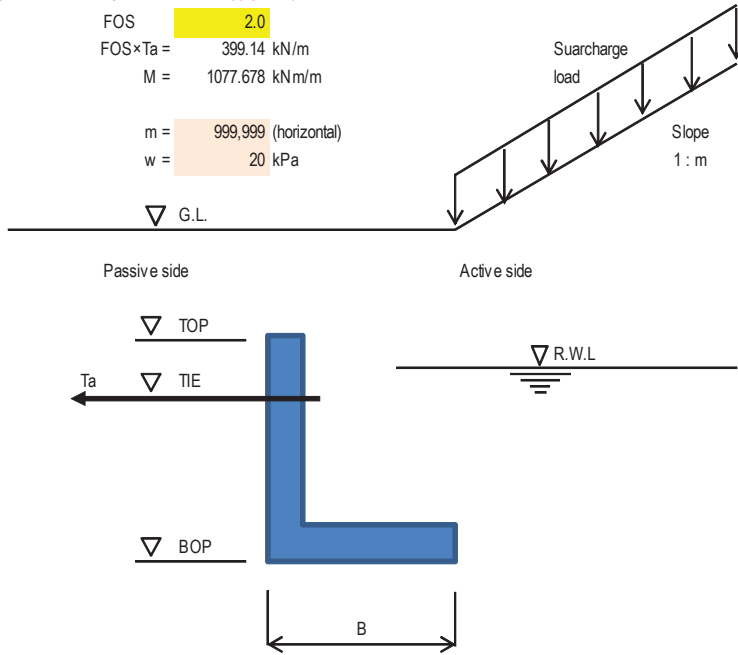
**Case : During Construction --- Tie tension FOS = 2.0**

**1. Design Conditions**

Ground level	GL	4.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.20 m

Tie elevation	TIE	3.00 m
Tie tension force	Ta	199.57 kN/m
Factor of safety	FOS	2.0
	FOS × Ta =	399.14 kN/m
	M =	1077.678 kNm/m

Ground slope	m =	999,999 (horizontal)
Surcharge load	w =	20 kPa



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cosδ	pp (kPa)	Pp (kN/m)
4.50		0.00	4.805	0.00	0.00
4.30	0.20	3.60	4.805	17.30	1.73
4.30	0.75	3.60	4.805	17.30	6.49
3.55		17.10	4.805	82.17	30.81
3.55	3.25	17.10	4.805	82.17	133.52
0.30		49.60	4.805	238.33	387.28
					559.83

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	$M_a$ (kNm/m)
4.50	0.20	20.00	0.291	5.82	0.58	4.13	2.41
4.30		23.60	0.291	6.87	0.69	4.07	2.79
4.30	0.75	23.60	0.291	6.87	2.58	3.75	9.66
3.55		37.10	0.291	10.80	4.05	3.50	14.17
3.55	3.25	37.10	0.291	10.80	17.54	2.17	38.01
0.30		69.60	0.291	20.25	32.91	1.08	35.65
					58.35		102.69

Active earth pressure	Horiz. force	116.70 kN/m	(= $P_a * FOS$ )
	Vertical force	31.27 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	139.72 kNm/m	(= $(M_a - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

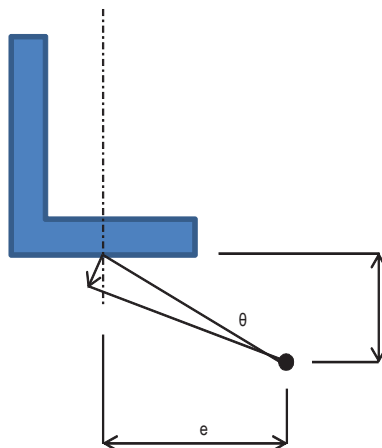
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	4.20	0.65	13.90	2.10	37.95	79.69
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.75	2.60	10.00	2.33	97.50	226.69
Soil 2	above water	3.75	0.75	18.00	2.33	50.63	117.70
Soil 3	above water	4.20	0.20	18.00	2.10	15.12	31.75
						225.56	461.31

Anchor plate	Vertical force	225.56 kN/m	(= W)
	Moment	12.35 kNm/m	(= $W * B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	0.479 m
Horizontal distance	e	-0.185 m
Rotation anglr	$\theta$	0.007705 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	4.50	0.00	4.98	0.00	383.62	0.00	0.00	4.20	0.00
	4.50		4.98	0.00	383.62	0.00	0.00	4.20	0.00
Plastic	4.50	0.20	4.98	0.00	383.62	0.00	0.00	4.13	0.00
	4.30		4.78	17.30	368.21	17.30	1.73	4.07	7.03
Plastic	4.30	0.00	4.78	17.30	368.21	17.30	0.00	4.00	0.00
	4.30		4.78	17.30	368.21	17.30	0.00	4.00	0.00
Plastic	4.30	0.75	4.78	17.30	368.21	17.30	6.49	3.75	24.33
	3.55		4.03	82.17	310.43	82.17	30.81	3.50	107.84
Plastic	3.55	1.82	4.03	82.17	310.43	82.17	74.96	2.64	198.04
	1.73		2.20	169.84	169.84	169.84	154.96	2.03	315.11
Elastic	1.73	1.43	2.20	169.84	169.84	169.84	121.04	0.95	115.01
	0.30		0.78	238.33	60.03	60.03	42.78	0.48	20.33
Below the top of Anchor Wall only							431.04		780.65

Passive side pressure	Horiz. force	431.04 kN/m	(= Pp')
	Vertical force	115.50 kN/m	(= Pp' * tan $\delta$ )
	Moment	1,023.19 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	147.57 kPa	P1	141.33 kN/m	M1	206.6 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
			141.33		206.6	
Effective contact area	A (A')	1.915384235 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	84.80 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	84.80 kN/m	(= Q)
	Vertical force	141.33 kN/m	(= P1+P2)
	Moment	206.56 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	116.70	31.27	139.72
Weight of Plate	0.00	225.56	12.35
Tie tension	399.14	0.00	1,077.68
Passive soil pressure	-431.04	-115.50	-1,023.19
Base reaction	-84.80	-141.33	-206.56
Total	-0.00	-0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	0.479 m		
Horizontal distance	e	-0.185 m		
Rotation angle	$\theta$	0.0077 rad	<	Allowable rotation angle 0.02 rad OK
Horizontal deflection at tie	$\delta$	24.49 mm	--- no limitation	

**Design of Anchor Plate (Elast-plastic method)**

**Block Block-b**

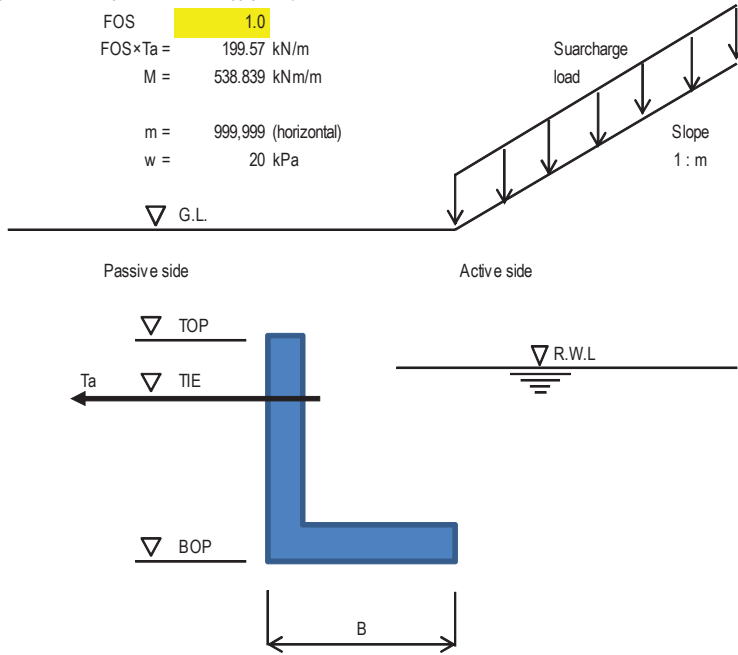
**Case : During Construction --- Tie tension FOS = 1.0 (Working tension load)**

**1. Design Conditions**

Ground level	GL	4.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.20 m

Tie elevation	TIE	3.00 m
Tie tension force	Ta	199.57 kN/m
Factor of safety	FOS	1.0
	FOS × Ta =	199.57 kN/m
	M =	538.839 kNm/m

Ground slope	m =	999,999 (horizontal)
Surcharge load	w =	20 kPa



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cosδ	pp (kPa)	Pp (kN/m)
4.50		0.00	4.805	0.00	0.00
4.30	0.20	3.60	4.805	17.30	1.73
4.30	0.75	3.60	4.805	17.30	6.49
3.55		17.10	4.805	82.17	30.81
3.55	3.25	17.10	4.805	82.17	133.52
0.30		49.60	4.805	238.33	387.28
					559.83

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.



Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	Ma (kNm/m)
4.50	0.20	20.00	0.291	5.82	0.58	4.13	2.41
4.30		23.60	0.291	6.87	0.69	4.07	2.79
4.30	0.75	23.60	0.291	6.87	2.58	3.75	9.66
3.55		37.10	0.291	10.80	4.05	3.50	14.17
3.55	3.25	37.10	0.291	10.80	17.54	2.17	38.01
0.30		69.60	0.291	20.25	32.91	1.08	35.65
					58.35		102.69

Active earth pressure	Horiz. force	58.35 kN/m	(= $P_a \cdot FOS$ )
	Vertical force	15.63 kN/m	(= $P_a \cdot \tan \delta \cdot FOS$ )
	Moment	69.86 kNm/m	(= $(Ma - V \cdot B/2) \cdot FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

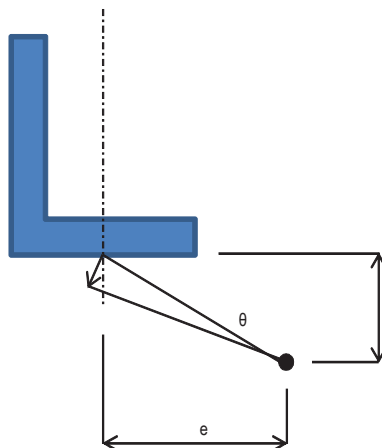
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	Wx (kNm/m)
Base	underwater	4.20	0.65	13.90	2.10	37.95	79.69
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.75	2.60	10.00	2.33	97.50	226.69
Soil 2	above water	3.75	0.75	18.00	2.33	50.63	117.70
Soil 3	above water	4.20	0.20	18.00	2.10	15.12	31.75
						225.56	461.31

Anchor plate	Vertical force	225.56 kN/m	(= W)
	Moment	12.35 kNm/m	(= $W \cdot B/2 - Wx$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	KH =	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	KV =	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	-0.879 m
Horizontal distance	e	1.678 m
Rotation anglr	$\theta$	0.002874 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	4.50	0.00	3.62	0.00	104.06	0.00	0.00	4.20	0.00
	4.50		3.62	0.00	104.06	0.00	0.00	4.20	0.00
Plastic	4.50	0.20	3.62	0.00	104.06	0.00	0.00	4.13	0.00
	4.30		3.42	17.30	98.32	17.30	1.73	4.07	7.03
Plastic	4.30	0.70	3.42	17.30	98.32	17.30	6.08	3.77	22.90
	3.60		2.72	78.11	78.11	78.11	27.46	3.53	96.97
Elastic	3.60	0.05	2.72	78.11	78.11	78.11	1.83	3.28	6.01
	3.55		2.67	82.17	76.76	76.76	1.80	3.27	5.88
Elastic	3.55	0.00	2.67	82.17	76.76	76.76	0.00	3.25	0.00
	3.55		2.67	82.17	76.76	76.76	0.00	3.25	0.00
Elastic	3.55	3.25	2.67	82.17	76.76	76.76	124.74	2.17	270.27
	0.30		-0.58	238.33	-16.63	-16.63	-27.02	1.08	-29.27
Below the top of Anchor Wall only							134.89		372.75

Passive side pressure	Horiz. force	134.89 kN/m	(= Pp')
	Vertical force	36.14 kN/m	(= Pp' * tan $\delta$ )
	Moment	448.66 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	108.56 kPa	P1	205.05 kN/m	M1	172.4 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				205.05		172.4
Effective contact area	A (A')	3.777701481 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	123.03 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	123.03 kN/m	(= Q)
	Vertical force	205.05 kN/m	(= P1+P2)
	Moment	172.40 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	58.35	15.63	69.86
Weight of Plate	0.00	225.56	12.35
Tie tension	199.57	0.00	538.84
Passive soil pressure	-134.89	-36.14	-448.66
Base reaction	-123.03	-205.05	-172.40
Total	-0.00	-0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	-0.879 m			
Horizontal distance	e	1.678 m			
Rotation angle	$\theta$	0.0029 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	5.23 mm	<	Allowable deflection	30 mm OK

# Summary of Analysis (Block-c)

AFTER COMPLETION (GL=+5.5m)	DURING CONSTRUCTION (GL=+4.5m)	WORKING LOAD (100% OF THE TENSION/ACTIVE EARTH PRESSURE)
<p><math>\theta = 0.0033</math> rad <math>\delta = 6.3</math> mm (at tie)</p>	<p><math>\theta = 0.0029</math> rad <math>\delta = 5.1</math> mm (at tie)</p>	<p><math>\theta = 0.0069</math> rad <math>\delta = 22.9</math> mm (at tie)</p>
<p><math>\theta = 0.0101</math> rad <math>\delta = 41.1</math> mm (at tie)</p>	<p><math>\theta = 0.0069</math> rad <math>\delta = 22.9</math> mm (at tie)</p>	<p>ULTIMATE LOAD (FOS TIMES THE TENSION/ACTIVE EARTH PRESSURE)</p>

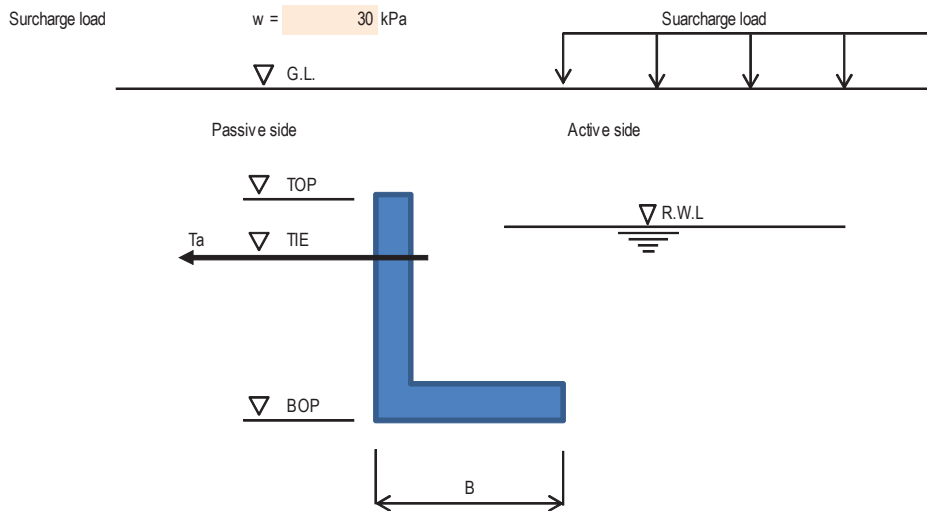
**Design of Anchor Plate (Elast-plastic method)**

**Block Block-c**

**Case : During Operation --- Tie tension FOS = 2.5**

**1. Design Conditions**

Ground level	GL	5.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.20 m
Tie elevation	TIE	3.00 m
Tie tension force	Ta	260.93 kN/m
Factor of safety	FOS	2.5
	FOS × Ta =	652.3275 kN/m
	M =	1761.28425 kNm/m



Soil conditions

Passive side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_p \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_a \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_p \cos \delta$	pp (kPa)	Pp (kN/m)
5.50		0.00	4.805	0.00	0.00
4.30	1.20	21.60	4.805	103.79	62.27
4.30	0.75	21.60	4.805	103.79	38.92
3.55		35.10	4.805	168.66	63.25
3.55	3.25	35.10	4.805	168.66	274.07
0.30		67.60	4.805	324.82	527.83
					966.33

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	$y$ (m)	$Ma$ (kNm/m)
5.50	1.20	30.00	0.291	8.73	5.24	4.80	25.14
4.30		51.60	0.291	15.02	9.01	4.40	39.64
4.30	0.75	51.60	0.291	15.02	5.63	3.75	21.12
3.55		65.10	0.291	18.94	7.10	3.50	24.86
3.55	3.25	65.10	0.291	18.94	30.78	2.17	66.70
0.30		97.60	0.291	28.40	46.15	1.08	50.00
					103.92		227.46

Active earth pressure	Horiz. force	259.80 kN/m	(= $P_a * FOS$ )
	Vertical force	69.61 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	422.47 kNm/m	(= $(Ma - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

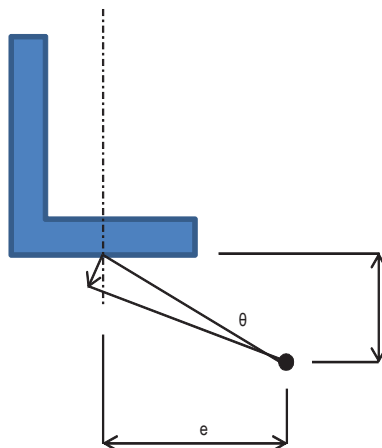
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	4.20	0.65	13.90	2.10	37.95	79.69
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.75	2.60	10.00	2.33	97.50	226.69
Soil 2	above water	3.75	0.75	18.00	2.33	50.63	117.70
Soil 3	above water	4.20	1.20	18.00	2.10	90.72	190.51
						301.16	620.07

Anchor plate	Vertical force	301.16 kN/m	(= W)
	Moment	12.35 kNm/m	(= $W * B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	1.364 m
Horizontal distance	e	-0.374 m
Rotation anglr	$\theta$	0.010108 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	5.50	0.00	6.86	0.00	693.77	0.00	0.00	5.20	0.00
	5.50		6.86	0.00	693.77	0.00	0.00	5.20	0.00
Plastic	5.50	1.20	6.86	0.00	693.77	0.00	0.00	4.80	0.00
	4.30		5.66	103.79	572.48	103.79	62.27	4.40	274.00
Plastic	4.30	0.00	5.66	103.79	572.48	103.79	0.00	4.00	0.00
	4.30		5.66	103.79	572.48	103.79	0.00	4.00	0.00
Plastic	4.30	0.75	5.66	103.79	572.48	103.79	38.92	3.75	145.95
	3.55		4.91	168.66	496.67	168.66	63.25	3.50	221.36
Plastic	3.55	2.20	4.91	168.66	496.67	168.66	185.48	2.52	466.83
	1.35		2.71	274.34	274.34	274.34	301.71	1.78	538.15
Elastic	1.35	1.05	2.71	274.34	274.34	274.34	144.09	0.70	100.91
	0.30		1.66	324.82	168.16	168.16	88.33	0.35	30.93
Below the top of Anchor Wall only							821.78		1,504.12

Passive side pressure	Horiz. force	821.78 kN/m	(= Pp')
	Vertical force	220.20 kN/m	(= Pp' * tan $\delta$ )
	Moment	1,966.54 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	174.47 kPa	P1	150.57 kN/m	M1	229.6 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				150.57		229.6
Effective contact area	A (A')	1.726066166 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	90.34 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	90.34 kN/m	(= Q)
	Vertical force	150.57 kN/m	(= P1+P2)
	Moment	229.57 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	259.80	69.61	422.47
Weight of Plate	0.00	301.16	12.35
Tie tension	652.33	0.00	1,761.28
Passive soil pressure	-821.78	-220.20	-1,966.54
Base reaction	-90.34	-150.57	-229.57
Total	0.00	0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	1.364 m			
Horizontal distance	e	-0.374 m			
Rotation angle	$\theta$	0.0101 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	41.08 mm	---	no limitation	

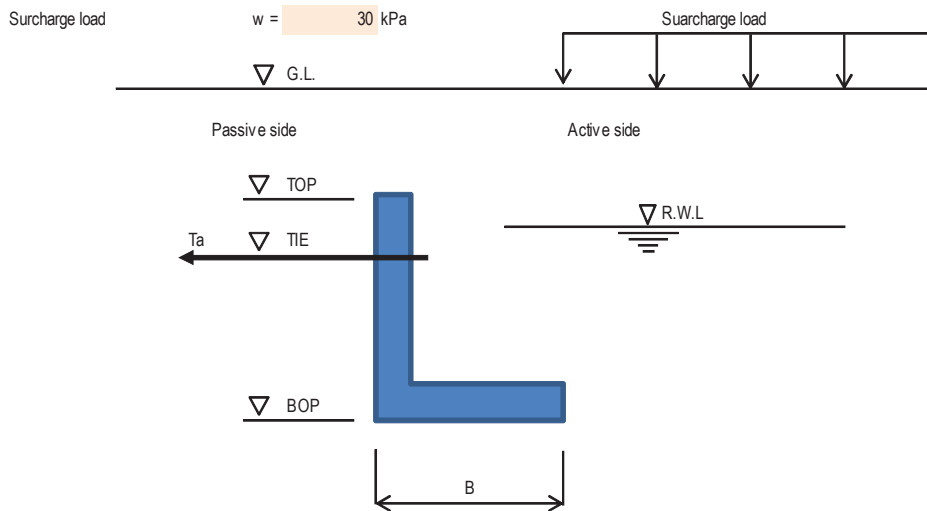
**Design of Anchor Plate (Elast-plastic method)**

**Block Block-c**

**Case : During Operation --- Tie tension FOS = 1.0 (Working tension load)**

**1. Design Conditions**

Ground level	GL	5.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.20 m
Tie elevation	TIE	3.00 m
Tie tension force	Ta	260.93 kN/m
Factor of safety	FOS	1.0
	FOS×Ta =	260.931 kN/m
	M =	704.5137 kNm/m



Soil conditions

Passive side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_p \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_a \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_p \cos \delta$	pp (kPa)	Pp (kN/m)
5.50		0.00	4.805	0.00	0.00
4.30	1.20	21.60	4.805	103.79	62.27
4.30	0.75	21.60	4.805	103.79	38.92
3.55		35.10	4.805	168.66	63.25
3.55	3.25	35.10	4.805	168.66	274.07
0.30		67.60	4.805	324.82	527.83
					966.33

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	$y$ (m)	$M_a$ (kNm/m)
5.50	1.20	30.00	0.291	8.73	5.24	4.80	25.14
4.30		51.60	0.291	15.02	9.01	4.40	39.64
4.30	0.75	51.60	0.291	15.02	5.63	3.75	21.12
3.55		65.10	0.291	18.94	7.10	3.50	24.86
3.55	3.25	65.10	0.291	18.94	30.78	2.17	66.70
0.30		97.60	0.291	28.40	46.15	1.08	50.00
					103.92		227.46

Active earth pressure	Horiz. force	103.92 kN/m	(= $P_a \cdot FOS$ )
	Vertical force	27.85 kN/m	(= $P_a \cdot \tan \delta \cdot FOS$ )
	Moment	168.99 kNm/m	(= $(M_a - V \cdot B/2) \cdot FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

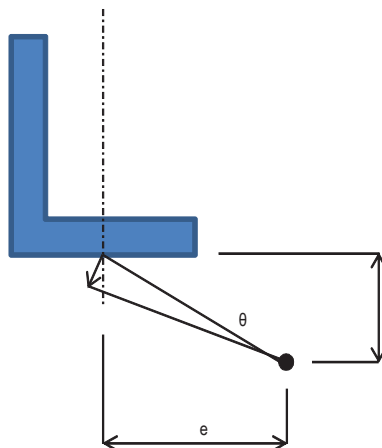
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	4.20	0.65	13.90	2.10	37.95	79.69
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.75	2.60	10.00	2.33	97.50	226.69
Soil 2	above water	3.75	0.75	18.00	2.33	50.63	117.70
Soil 3	above water	4.20	1.20	18.00	2.10	90.72	190.51
						301.16	620.07

Anchor plate	Vertical force	301.16 kN/m	(= W)
	Moment	12.35 kNm/m	(= $W \cdot B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	-0.769 m
Horizontal distance	e	2.007 m
Rotation anglr	$\theta$	0.003267 rad





## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	5.50	0.00	4.73	0.00	154.56	0.00	0.00	5.20	0.00
	5.50		4.73	0.00	154.56	0.00	0.00	5.20	0.00
Plastic	5.50	1.20	4.73	0.00	154.56	0.00	0.00	4.80	0.00
	4.30		3.53	103.79	115.36	103.79	62.27	4.40	274.00
Plastic	4.30	0.10	3.53	103.79	115.36	103.79	5.04	3.97	19.99
	4.20		3.43	112.19	112.19	112.19	5.45	3.94	21.43
Elastic	4.20	0.65	3.43	112.19	112.19	112.19	36.62	3.69	134.97
	3.55		2.78	168.66	90.86	90.86	29.66	3.47	102.85
Elastic	3.55	0.00	2.78	168.66	90.86	90.86	0.00	3.25	0.00
	3.55		2.78	168.66	90.86	90.86	0.00	3.25	0.00
Elastic	3.55	3.25	2.78	168.66	90.86	90.86	147.64	2.17	319.89
	0.30		-0.47	324.82	-15.31	-15.31	-24.88	1.08	-26.96
Below the top of Anchor Wall only							199.53		572.18

Passive side pressure	Horiz. force	199.53 kN/m	(= Pp')
	Vertical force	53.46 kN/m	(= Pp' * tan $\delta$ )
	Moment	684.45 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	134.17 kPa	P1	275.54 kN/m	M1	201.4 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				275.54		201.4
Effective contact area	A (A')	4.10717768 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	165.32 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	165.32 kN/m	(= Q)
	Vertical force	275.54 kN/m	(= P1+P2)
	Moment	201.40 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force (+) ←	Vert force (+) ↓	Moment (+) ↻
Active earth pressure	103.92	27.85	168.99
Weight of Plate	0.00	301.16	12.35
Tie tension	260.93	0.00	704.51
Passive soil pressure	-199.53	-53.46	-684.45
Base reaction	-165.32	-275.54	-201.40
Total	0.00	0.00	0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	-0.769 m			
Horizontal distance	e	2.007 m			
Rotation angle	$\theta$	0.0033 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	6.31 mm	<	Allowable deflection	30 mm OK

**Design of Anchor Plate (Elast-plastic method)**

**Block Block-c**

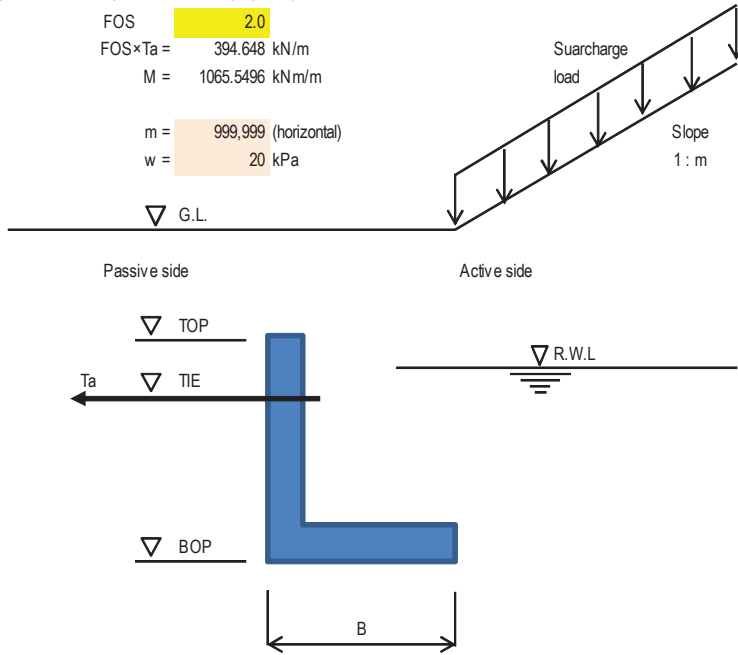
**Case : During Construction --- Tie tension FOS = 2.0**

**1. Design Conditions**

Ground level	GL	4.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.20 m

Tie elevation	TIE	3.00 m
Tie tension force	Ta	197.32 kN/m
Factor of safety	FOS	2.0
	FOS × Ta =	394.648 kN/m
	M =	1065.5496 kNm/m

Ground slope	m =	999,999 (horizontal)
Surcharge load	w =	20 kPa



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cosδ	pp (kPa)	Pp (kN/m)
4.50		0.00	4.805	0.00	0.00
4.30	0.20	3.60	4.805	17.30	1.73
4.30	0.75	3.60	4.805	17.30	6.49
3.55		17.10	4.805	82.17	30.81
3.55	3.25	17.10	4.805	82.17	133.52
0.30		49.60	4.805	238.33	387.28
					559.83

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	Ma (kNm/m)
4.50	0.20	20.00	0.291	5.82	0.58	4.13	2.41
4.30		23.60	0.291	6.87	0.69	4.07	2.79
4.30	0.75	23.60	0.291	6.87	2.58	3.75	9.66
3.55		37.10	0.291	10.80	4.05	3.50	14.17
3.55	3.25	37.10	0.291	10.80	17.54	2.17	38.01
0.30		69.60	0.291	20.25	32.91	1.08	35.65
					58.35		102.69

Active earth pressure	Horiz. force	116.70 kN/m	(= $P_a * FOS$ )
	Vertical force	31.27 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	139.72 kNm/m	(= $(Ma - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

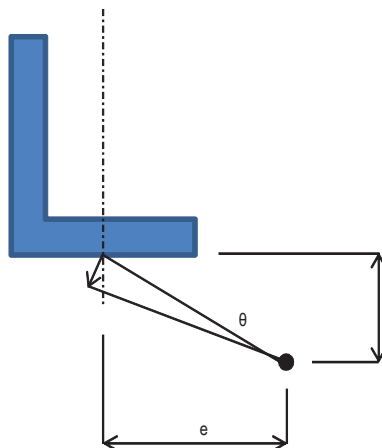
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	Wx (kNm/m)
Base	underwater	4.20	0.65	13.90	2.10	37.95	79.69
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.75	2.60	10.00	2.33	97.50	226.69
Soil 2	above water	3.75	0.75	18.00	2.33	50.63	117.70
Soil 3	above water	4.20	0.20	18.00	2.10	15.12	31.75
						225.56	461.31

Anchor plate	Vertical force	225.56 kN/m	(= W)
	Moment	12.35 kNm/m	(= $W * B/2 - Wx$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	KH =	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	KV =	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	0.591 m
Horizontal distance	e	-0.073 m
Rotation anglr	$\theta$	0.006948 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	4.50	0.00	5.09	0.00	353.71	0.00	0.00	4.20	0.00
	4.50		5.09	0.00	353.71	0.00	0.00	4.20	0.00
Plastic	4.50	0.20	5.09	0.00	353.71	0.00	0.00	4.13	0.00
	4.30		4.89	17.30	339.81	17.30	1.73	4.07	7.03
Plastic	4.30	0.00	4.89	17.30	339.81	17.30	0.00	4.00	0.00
	4.30		4.89	17.30	339.81	17.30	0.00	4.00	0.00
Plastic	4.30	0.75	4.89	17.30	339.81	17.30	6.49	3.75	24.33
	3.55		4.14	82.17	287.71	82.17	30.81	3.50	107.84
Plastic	3.55	1.75	4.14	82.17	287.71	82.17	71.85	2.67	191.62
	1.80		2.39	166.20	166.20	166.20	145.33	2.08	302.88
Elastic	1.80	1.50	2.39	166.20	166.20	166.20	124.74	1.00	124.83
	0.30		0.89	238.33	61.91	61.91	46.47	0.50	23.25
Below the top of Anchor Wall only							425.69		774.76

Passive side pressure	Horiz. force	425.69 kN/m	(= Pp')
	Vertical force	114.06 kN/m	(= Pp' * tan $\delta$ )
	Moment	1,014.29 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	140.84 kPa	P1	142.76 kN/m	M1	203.3 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
			142.76		203.3	
Effective contact area	A (A')	2.027229109 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	85.66 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	85.66 kN/m	(= Q)
	Vertical force	142.76 kN/m	(= P1+P2)
	Moment	203.33 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	116.70	31.27	139.72
Weight of Plate	0.00	225.56	12.35
Tie tension	394.65	0.00	1,065.55
Passive soil pressure	-425.69	-114.06	-1,014.29
Base reaction	-85.66	-142.76	-203.33
<b>Total</b>	<b>-0.00</b>	<b>-0.00</b>	<b>-0.00</b>

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	0.591 m		
Horizontal distance	e	-0.073 m		
Rotation angle	$\theta$	0.0069 rad	<	Allowable rotation angle 0.02 rad OK
Horizontal deflection at tie	$\delta$	22.87 mm	--- no limitation	

**Design of Anchor Plate (Elast-plastic method)**

**Block Block-c**

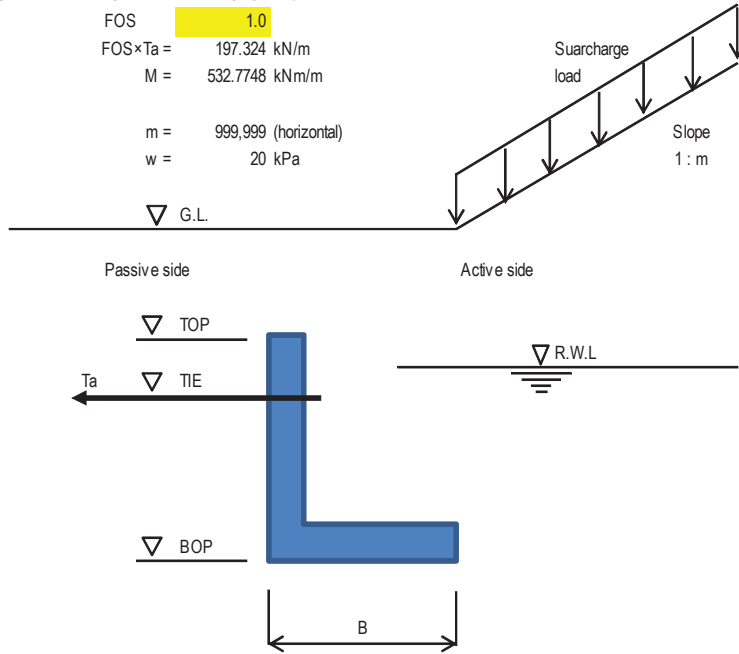
**Case : During Construction --- Tie tension FOS = 1.0 (Working tension load)**

**1. Design Conditions**

Ground level	GL	4.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.20 m

Tie elevation	TIE	3.00 m
Tie tension force	Ta	197.32 kN/m
Factor of safety	FOS	<b>1.0</b>
	FOS × Ta =	197.324 kN/m
	M =	532.7748 kNm/m

Ground slope	m =	999,999 (horizontal)
Surcharge load	w =	20 kPa



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cos δ
	Top	Bottom				
layer-1	4.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cos δ
	Top	Bottom				
layer-1	4.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cos δ	pp (kPa)	Pp (kN/m)
4.50	0.20	0.00	4.805	0.00	0.00
4.30		3.60	4.805	17.30	1.73
4.30	0.75	3.60	4.805	17.30	6.49
3.55		17.10	4.805	82.17	30.81
3.55	3.25	17.10	4.805	82.17	133.52
0.30		49.60	4.805	238.33	387.28
					559.83

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	Ma (kNm/m)
4.50	0.20	20.00	0.291	5.82	0.58	4.13	2.41
4.30		23.60	0.291	6.87	0.69	4.07	2.79
4.30	0.75	23.60	0.291	6.87	2.58	3.75	9.66
3.55		37.10	0.291	10.80	4.05	3.50	14.17
3.55	3.25	37.10	0.291	10.80	17.54	2.17	38.01
0.30		69.60	0.291	20.25	32.91	1.08	35.65
					58.35		102.69

Active earth pressure	Horiz. force	58.35 kN/m	(= $P_a * FOS$ )
	Vertical force	15.63 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	69.86 kNm/m	(= $(Ma - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	Wx (kNm/m)
Base	underwater	4.20	0.65	13.90	2.10	37.95	79.69
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	3.75	2.60	10.00	2.33	97.50	226.69
Soil 2	above water	3.75	0.75	18.00	2.33	50.63	117.70
Soil 3	above water	4.20	0.20	18.00	2.10	15.12	31.75
						225.56	461.31

Anchor plate	Vertical force	225.56 kN/m	(= W)
	Moment	12.35 kNm/m	(= $W * B/2 - Wx$ )

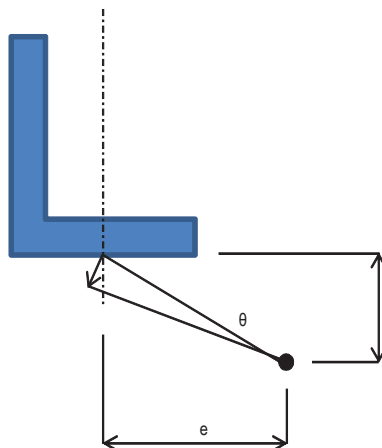
### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	KH =	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	KV =	10,000 kN/m <sup>3</sup> ... = 2000 N

Center of rotation

Center of rotation and rotation angle are assumed as follows:

Vertical distance	y	-0.905 m
Horizontal distance	e	1.687 m
Rotation anglr	$\theta$	0.002870 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	4.50	0.00	3.59	0.00	103.15	0.00	0.00	4.20	0.00
	4.50		3.59	0.00	103.15	0.00	0.00	4.20	0.00
Plastic	4.50	0.20	3.59	0.00	103.15	0.00	0.00	4.13	0.00
	4.30		3.39	17.30	97.41	17.30	1.73	4.07	7.03
Plastic	4.30	0.70	3.39	17.30	97.41	17.30	6.02	3.77	22.67
	3.60		2.70	77.45	77.45	77.45	26.94	3.54	95.25
Elastic	3.60	0.05	2.70	77.45	77.45	77.45	2.11	3.29	6.93
	3.55		2.64	82.17	75.89	75.89	2.07	3.27	6.76
Elastic	3.55	0.00	2.64	82.17	75.89	75.89	0.00	3.25	0.00
	3.55		2.64	82.17	75.89	75.89	0.00	3.25	0.00
Elastic	3.55	3.25	2.64	82.17	75.89	75.89	123.32	2.17	267.20
	0.30		-0.61	238.33	-17.38	-17.38	-28.24	1.08	-30.59
Below the top of Anchor Wall only							132.21		368.22

Passive side pressure	Horiz. force	132.21 kN/m	(= Pp')
	Vertical force	35.43 kN/m	(= Pp' * tan $\delta$ )
	Moment	442.61 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	108.67 kPa	P1	205.76 kN/m	M1	172.4 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				205.76		172.4
Effective contact area	A (A')	3.786835623 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	123.46 kN/m	(= $\mu * (P_1+P_2)$ )			

Base reaction force	Horiz. force	123.46 kN/m	(= Q)
	Vertical force	205.76 kN/m	(= P1+P2)
	Moment	172.37 kNm/m	(= M1+M2)

### Balance of forces

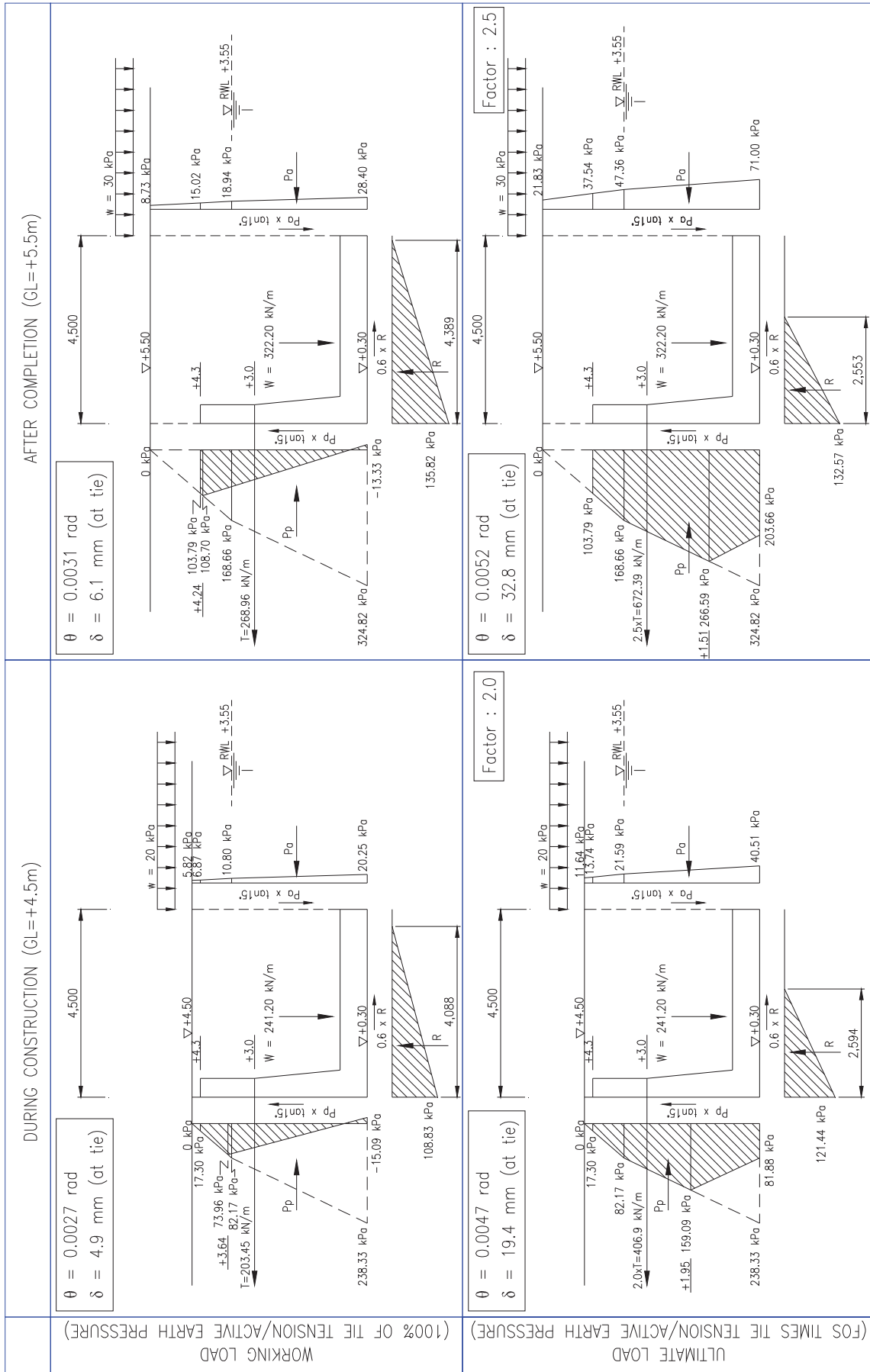
	Horiz force (+) ←	Vert force (+) ↓	Moment (+) ↻
Active earth pressure	58.35	15.63	69.86
Weight of Plate	0.00	225.56	12.35
Tie tension	197.32	0.00	532.77
Passive soil pressure	-132.21	-35.43	-442.61
Base reaction	-123.46	-205.76	-172.37
Total	0.00	0.00	0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	-0.905 m			
Horizontal distance	e	1.687 m			
Rotation angle	$\theta$	0.0029 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	5.15 mm	<	Allowable deflection	30 mm OK

# Summary of Analysis (Block-d)





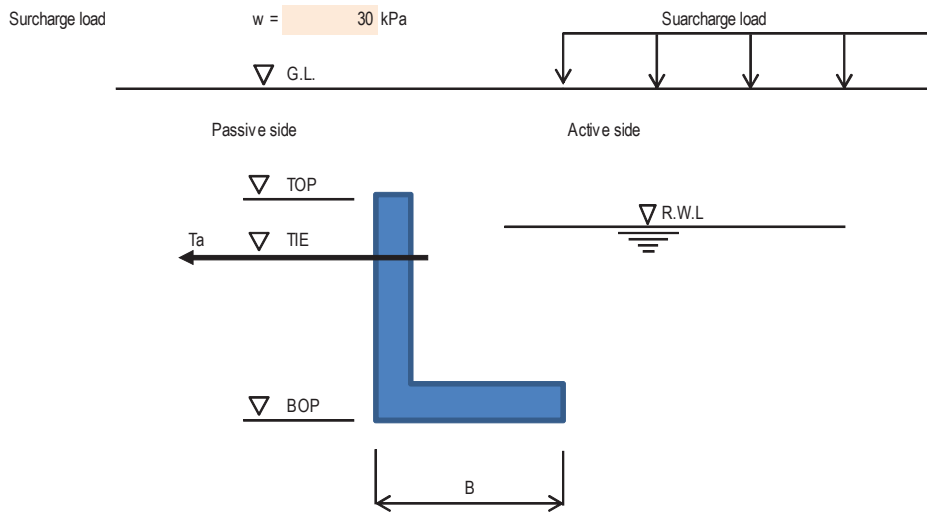
**Design of Anchor Plate (Elast-plastic method)**

**Block Block-d**

**Case : During Operation --- Tie tension FOS = 2.5**

**1. Design Conditions**

Ground level	GL	5.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.50 m
Tie elevation	TIE	3.00 m
Tie tension force	Ta	268.96 kN/m
Factor of safety	FOS	2.5
	FOS×Ta =	672.39 kN/m
	M =	1815.453 kNm/m



Soil conditions

Passive side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_p \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_a \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_p \cos \delta$	pp (kPa)	Pp (kN/m)
5.50		0.00	4.805	0.00	0.00
4.30	1.20	21.60	4.805	103.79	62.27
4.30	0.75	21.60	4.805	103.79	38.92
3.55		35.10	4.805	168.66	63.25
3.55	3.25	35.10	4.805	168.66	274.07
0.30		67.60	4.805	324.82	527.83
					966.33

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	$y$ (m)	$Ma$ (kNm/m)
5.50	1.20	30.00	0.291	8.73	5.24	4.80	25.14
4.30		51.60	0.291	15.02	9.01	4.40	39.64
4.30	0.75	51.60	0.291	15.02	5.63	3.75	21.12
3.55		65.10	0.291	18.94	7.10	3.50	24.86
3.55	3.25	65.10	0.291	18.94	30.78	2.17	66.70
0.30		97.60	0.291	28.40	46.15	1.08	50.00
					103.92		227.46

Active earth pressure	Horiz. force	259.80 kN/m	(= $P_a * FOS$ )
	Vertical force	69.61 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	412.02 kNm/m	(= $(Ma - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

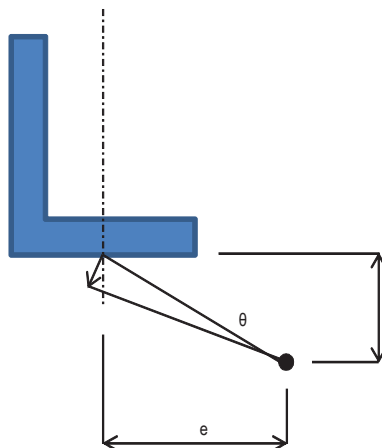
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	4.50	0.65	13.90	2.25	40.66	91.48
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	4.05	2.60	10.00	2.48	105.30	260.62
Soil 2	above water	4.05	0.75	18.00	2.48	54.68	135.32
Soil 3	above water	4.50	1.20	18.00	2.25	97.20	218.70
						322.20	711.60

Anchor plate	Vertical force	322.20 kN/m	(= W)
	Moment	13.34 kNm/m	(= $W * B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	3.622 m
Horizontal distance	e	0.303 m
Rotation anglr	$\theta$	0.005192 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	5.50	0.00	9.12	0.00	473.67	0.00	0.00	5.20	0.00
	5.50		9.12	0.00	473.67	0.00	0.00	5.20	0.00
Plastic	5.50	1.20	9.12	0.00	473.67	0.00	0.00	4.80	0.00
	4.30		7.92	103.79	411.36	103.79	62.27	4.40	274.00
Plastic	4.30	0.00	7.92	103.79	411.36	103.79	0.00	4.00	0.00
	4.30		7.92	103.79	411.36	103.79	0.00	4.00	0.00
Plastic	4.30	0.75	7.92	103.79	411.36	103.79	38.92	3.75	145.95
	3.55		7.17	168.66	372.42	168.66	63.25	3.50	221.36
Plastic	3.55	2.04	7.17	168.66	372.42	168.66	171.87	2.57	441.81
	1.51		5.13	266.59	266.59	266.59	271.67	1.89	513.79
Elastic	1.51	1.21	5.13	266.59	266.59	266.59	161.54	0.81	130.51
	0.30		3.92	324.82	203.66	203.66	123.41	0.40	49.85
Below the top of Anchor Wall only							830.65		1,503.28

Passive side pressure	Horiz. force	830.65 kN/m	(= Pp')
	Vertical force	222.57 kN/m	(= Pp' * tan $\delta$ )
	Moment	2,004.06 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	132.57 kPa	P1	169.24 kN/m	M1	236.8 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
				169.24		236.8
Effective contact area	A (A')	2.553143148 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	101.54 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	101.54 kN/m	(= Q)
	Vertical force	169.24 kN/m	(= P1+P2)
	Moment	236.76 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	259.80	69.61	412.02
Weight of Plate	0.00	322.20	13.34
Tie tension	672.39	0.00	1,815.45
Passive soil pressure	-830.65	-222.57	-2,004.06
Base reaction	-101.54	-169.24	-236.76
Total	-0.00	-0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	3.622 m		
Horizontal distance	e	0.303 m		
Rotation angle	$\theta$	0.0052 rad	<	Allowable rotation angle 0.02 rad OK
Horizontal deflection at tie	$\delta$	32.83 mm	--- no limitation	

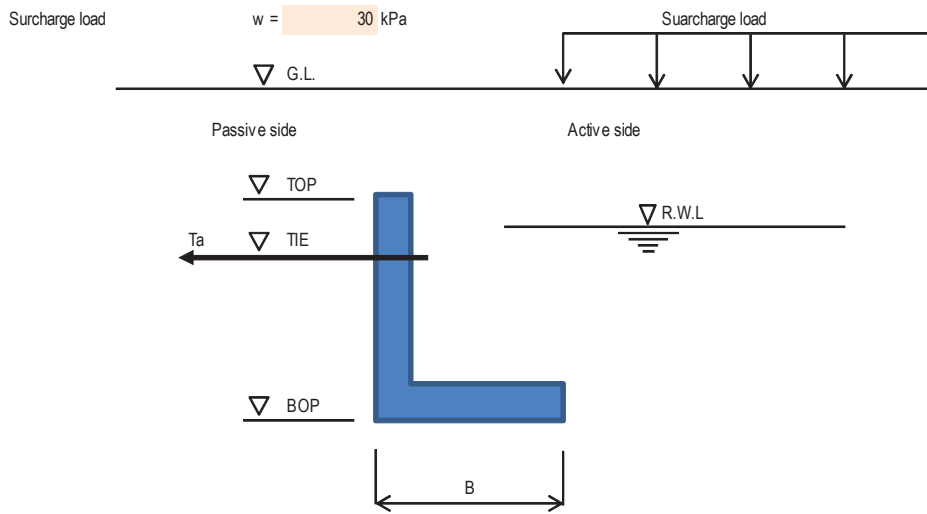
**Design of Anchor Plate (Elast-plastic method)**

**Block Block-d**

**Case : During Operation --- Tie tension FOS = 1.0 (Working tension load)**

**1. Design Conditions**

Ground level	GL	5.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.50 m
Tie elevation	TIE	3.00 m
Tie tension force	Ta	268.96 kN/m
Factor of safety	FOS	1.0
	FOS × Ta =	268.956 kN/m
	M =	726.1812 kNm/m



Soil conditions

Passive side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_p \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (deg)	$\delta$ (deg)	$K_a \cos \delta$
	Top	Bottom				
layer-1	5.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_p \cos \delta$	pp (kPa)	Pp (kN/m)
5.50		0.00	4.805	0.00	0.00
4.30	1.20	21.60	4.805	103.79	62.27
4.30	0.75	21.60	4.805	103.79	38.92
3.55		35.10	4.805	168.66	63.25
3.55	3.25	35.10	4.805	168.66	274.07
0.30		67.60	4.805	324.82	527.83
					966.33

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	$M_a$ (kNm/m)
5.50	1.20	30.00	0.291	8.73	5.24	4.80	25.14
4.30		51.60	0.291	15.02	9.01	4.40	39.64
4.30	0.75	51.60	0.291	15.02	5.63	3.75	21.12
3.55		65.10	0.291	18.94	7.10	3.50	24.86
3.55	3.25	65.10	0.291	18.94	30.78	2.17	66.70
0.30		97.60	0.291	28.40	46.15	1.08	50.00
					103.92		227.46

Active earth pressure	Horiz. force	103.92 kN/m	(= $P_a \cdot FOS$ )
	Vertical force	27.85 kN/m	(= $P_a \cdot \tan \delta \cdot FOS$ )
	Moment	164.81 kNm/m	(= $(M_a - V \cdot B/2) \cdot FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

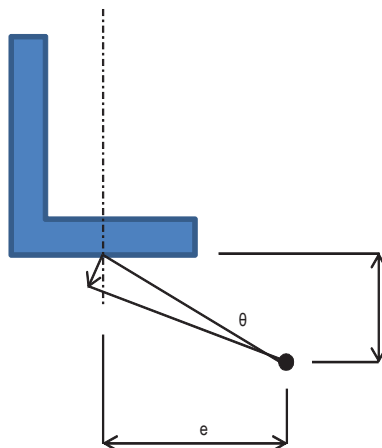
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	$W_x$ (kNm/m)
Base	underwater	4.50	0.65	13.90	2.25	40.66	91.48
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	4.05	2.60	10.00	2.48	105.30	260.62
Soil 2	above water	4.05	0.75	18.00	2.48	54.68	135.32
Soil 3	above water	4.50	1.20	18.00	2.25	97.20	218.70
						322.20	711.60

Anchor plate	Vertical force	322.20 kN/m	(= W)
	Moment	13.34 kNm/m	(= $W \cdot B/2 - W_x$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	$K_H =$	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	$K_V =$	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	-0.731 m
Horizontal distance	e	2.139 m
Rotation anglr	$\theta$	0.003095 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	5.50	0.00	4.77	0.00	147.59	0.00	0.00	5.20	0.00
	5.50		4.77	0.00	147.59	0.00	0.00	5.20	0.00
Plastic	5.50	1.20	4.77	0.00	147.59	0.00	0.00	4.80	0.00
	4.30		3.57	103.79	110.45	103.79	62.27	4.40	274.00
Plastic	4.30	0.06	3.57	103.79	110.45	103.79	2.94	3.98	11.72
	4.24		3.51	108.70	108.70	108.70	3.08	3.96	12.22
Elastic	4.24	0.69	3.51	108.70	108.70	108.70	37.68	3.71	139.86
	3.55		2.82	168.66	87.24	87.24	30.24	3.48	105.27
Elastic	3.55	0.00	2.82	168.66	87.24	87.24	0.00	3.25	0.00
	3.55		2.82	168.66	87.24	87.24	0.00	3.25	0.00
Elastic	3.55	3.25	2.82	168.66	87.24	87.24	141.77	2.17	307.16
	0.30		-0.43	324.82	-13.33	-13.33	-21.67	1.08	-23.47
Below the top of Anchor Wall only							194.05		552.77

Passive side pressure	Horiz. force	194.05 kN/m	(= Pp')
	Vertical force	51.99 kN/m	(= Pp' * tan $\delta$ )
	Moment	669.76 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	135.82 kPa	P1	298.05 kN/m	M1	234.6 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
			298.05		234.6	
Effective contact area	A (A')	4.388866672 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	178.83 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	178.83 kN/m	(= Q)
	Vertical force	298.05 kN/m	(= P1+P2)
	Moment	234.58 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	103.92	27.85	164.81
Weight of Plate	0.00	322.20	13.34
Tie tension	268.96	0.00	726.18
Passive soil pressure	-194.05	-51.99	-669.76
Base reaction	-178.83	-298.05	-234.58
Total	-0.00	0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	-0.731 m			
Horizontal distance	e	2.139 m			
Rotation angle	$\theta$	0.0031 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	6.09 mm	<	Allowable deflection	30 mm OK

**Design of Anchor Plate (Elast-plastic method)**

**Block Block-d**

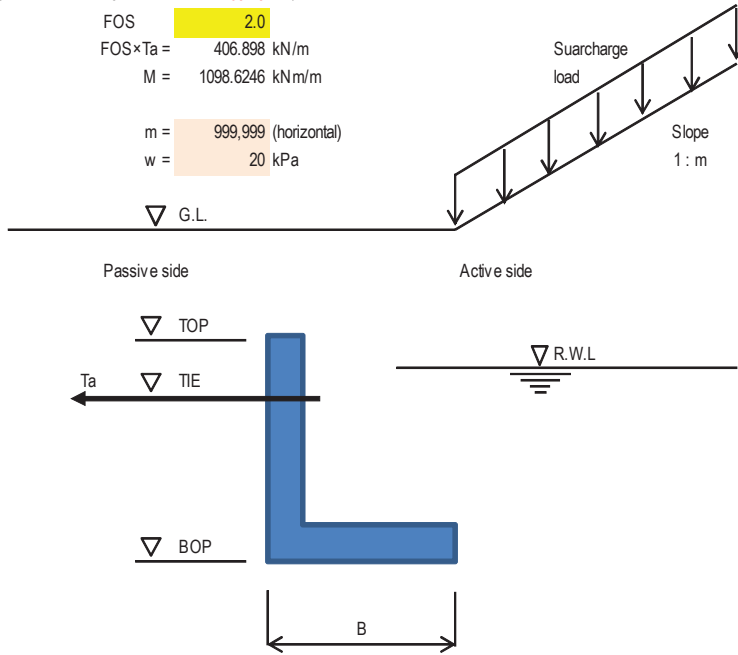
**Case : During Construction --- Tie tension FOS = 2.0**

**1. Design Conditions**

Ground level	GL	4.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.50 m

Tie elevation	TIE	3.00 m
Tie tension force	Ta	203.45 kN/m
Factor of safety	FOS	2.0
	FOS × Ta =	406.898 kN/m
	M =	1098.6246 kNm/m

Ground slope	m =	999,999 (horizontal)
Surcharge load	w =	20 kPa



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cosδ	pp (kPa)	Pp (kN/m)
4.50		0.00	4.805	0.00	0.00
4.30	0.20	3.60	4.805	17.30	1.73
4.30	0.75	3.60	4.805	17.30	6.49
3.55		17.10	4.805	82.17	30.81
3.55	3.25	17.10	4.805	82.17	133.52
0.30		49.60	4.805	238.33	387.28
					559.83

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	Ma (kNm/m)
4.50	0.20	20.00	0.291	5.82	0.58	4.13	2.41
4.30		23.60	0.291	6.87	0.69	4.07	2.79
4.30	0.75	23.60	0.291	6.87	2.58	3.75	9.66
3.55		37.10	0.291	10.80	4.05	3.50	14.17
3.55	3.25	37.10	0.291	10.80	17.54	2.17	38.01
0.30		69.60	0.291	20.25	32.91	1.08	35.65
					58.35		102.69

Active earth pressure	Horiz. force	116.70 kN/m	(= $P_a * FOS$ )
	Vertical force	31.27 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	135.03 kNm/m	(= $(Ma - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

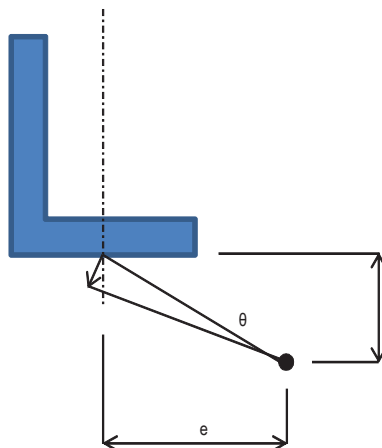
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	Wx (kNm/m)
Base	underwater	4.50	0.65	13.90	2.25	40.66	91.48
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	4.05	2.60	10.00	2.48	105.30	260.62
Soil 2	above water	4.05	0.75	18.00	2.48	54.68	135.32
Soil 3	above water	4.50	0.20	18.00	2.25	16.20	36.45
						241.20	529.35

Anchor plate	Vertical force	241.20 kN/m	(= W)
	Moment	13.34 kNm/m	(= $W * B/2 - Wx$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	KH =	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	KV =	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	1.449 m
Horizontal distance	e	0.344 m
Rotation anglr	$\theta$	0.004682 rad





## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	4.50	0.00	5.95	0.00	278.53	0.00	0.00	4.20	0.00
	4.50		5.95	0.00	278.53	0.00	0.00	4.20	0.00
Plastic	4.50	0.20	5.95	0.00	278.53	0.00	0.00	4.13	0.00
	4.30		5.75	17.30	269.16	17.30	1.73	4.07	7.03
Plastic	4.30	0.00	5.75	17.30	269.16	17.30	0.00	4.00	0.00
	4.30		5.75	17.30	269.16	17.30	0.00	4.00	0.00
Plastic	4.30	0.75	5.75	17.30	269.16	17.30	6.49	3.75	24.33
	3.55		5.00	82.17	234.05	82.17	30.81	3.50	107.84
Plastic	3.55	1.60	5.00	82.17	234.05	82.17	65.77	2.72	178.66
	1.95		3.40	159.09	159.09	159.09	127.35	2.18	277.96
Elastic	1.95	1.65	3.40	159.09	159.09	159.09	131.18	1.10	144.21
	0.30		1.75	238.33	81.88	81.88	67.51	0.55	37.11
Below the top of Anchor Wall only							429.10		770.11

Passive side pressure	Horiz. force	429.10 kN/m	(= Pp')
	Vertical force	114.98 kN/m	(= Pp' * tan $\delta$ )
	Moment	1,028.81 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	121.44 kPa	P1	157.49 kN/m	M1	218.2 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
			157.49		218.2	
Effective contact area	A (A')	2.593702963 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	94.49 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	94.49 kN/m	(= Q)
	Vertical force	157.49 kN/m	(= P1+P2)
	Moment	218.19 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force	Vert force	Moment
	(+) ←	(+) ↓	↺ (+)
Active earth pressure	116.70	31.27	135.03
Weight of Plate	0.00	241.20	13.34
Tie tension	406.90	0.00	1,098.62
Passive soil pressure	-429.10	-114.98	-1,028.81
Base reaction	-94.49	-157.49	-218.19
Total	-0.00	-0.00	-0.00

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	1.449 m		
Horizontal distance	e	0.344 m		
Rotation angle	$\theta$	0.0047 rad	<	Allowable rotation angle 0.02 rad OK
Horizontal deflection at tie	$\delta$	19.42 mm	--- no limitation	

**Design of Anchor Plate (Elast-plastic method)**

**Block Block-d**

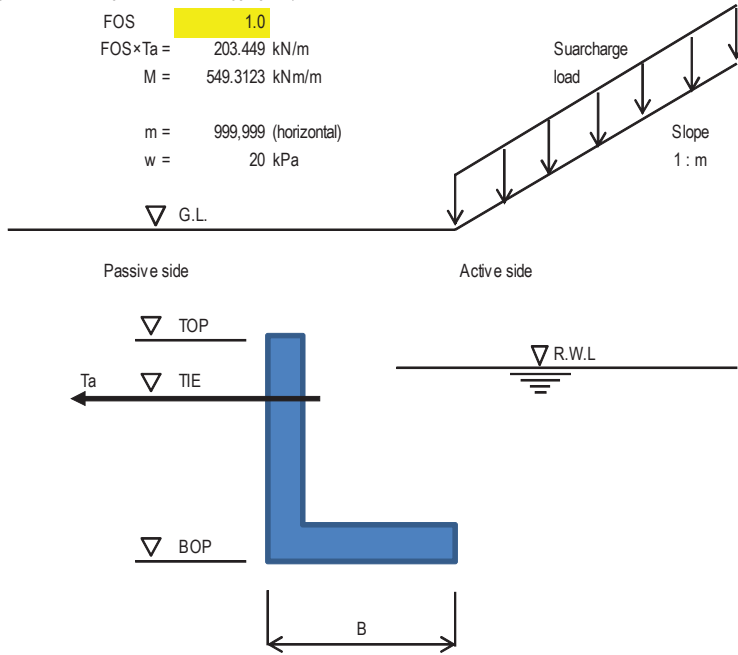
**Case : During Construction --- Tie tension FOS = 1.0 (Working tension load)**

**1. Design Conditions**

Ground level	GL	4.50 m
Residual water level	RWL	3.55 m
Top of anchor plate	TOP	4.30 m
Bottom of anchor plate	BOP	0.30 m
Width of anchor plate	B	4.50 m

Tie elevation	TIE	3.00 m
Tie tension force	Ta	203.45 kN/m
Factor of safety	FOS	1.0
	FOS × Ta =	203.449 kN/m
	M =	549.3123 kNm/m

Ground slope	m =	999,999 (horizontal)
Surcharge load	w =	20 kPa



Soil conditions  
Passive side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Kp cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	-15	4.805
layer-2	4.30	3.55	18	30	-15	4.805
layer-3	3.55	0.30	10	30	-15	4.805

Active side

	Elevation		γ (kN/m <sup>3</sup> )	φ (deg)	δ (deg)	Ka cosδ
	Top	Bottom				
layer-1	4.50	4.30	18	30	15	0.291
layer-2	4.30	3.55	18	30	15	0.291
layer-3	3.55	0.30	10	30	15	0.291

**2. Earth Pressure**

Passive earth pressure

Elev. (m)	h (m)	γh (kPa)	Kp cosδ	pp (kPa)	Pp (kN/m)
4.50		0.00	4.805	0.00	0.00
4.30	0.20	3.60	4.805	17.30	1.73
4.30	0.75	3.60	4.805	17.30	6.49
3.55		17.10	4.805	82.17	30.81
3.55	3.25	17.10	4.805	82.17	133.52
0.30		49.60	4.805	238.33	387.28
					559.83

\* Note that the above calculated values is the limit of the passive pressure which may not be fully mobilised depending on the soil strain based on the analysis.

Active earth pressure

Elev. (m)	h (m)	$\gamma h$ (kPa)	$K_a \cos \delta$	$p_a$ (kPa)	$P_a$ (kN/m)	y (m)	Ma (kNm/m)
4.50	0.20	20.00	0.291	5.82	0.58	4.13	2.41
4.30		23.60	0.291	6.87	0.69	4.07	2.79
4.30	0.75	23.60	0.291	6.87	2.58	3.75	9.66
3.55		37.10	0.291	10.80	4.05	3.50	14.17
3.55	3.25	37.10	0.291	10.80	17.54	2.17	38.01
0.30		69.60	0.291	20.25	32.91	1.08	35.65
					58.35		102.69

Active earth pressure	Horiz. force	58.35 kN/m	(= $P_a * FOS$ )
	Vertical force	15.63 kN/m	(= $P_a * \tan \delta * FOS$ )
	Moment	67.51 kNm/m	(= $(Ma - V * B/2) * FOS$ )

### 3. Weight of Anchor Plate (w/ soil)

Weight of anchor plate and soil

Wall thickness	b	0.45 m	Concrete	above water	$\gamma =$	24.00 kN/m <sup>3</sup>
Base slab thickness	t	0.65 m		underwater	$\gamma =$	13.90 kN/m <sup>3</sup>

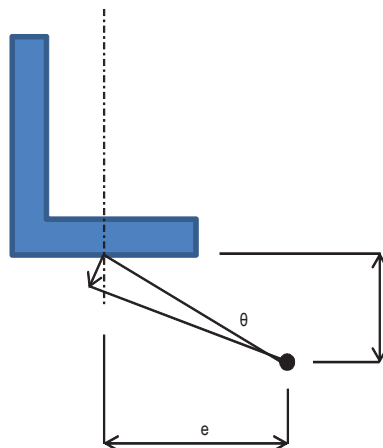
		b (m)	h (m)	$\gamma$ (kN/m <sup>3</sup> )	x (m)	W (kN/m)	Wx (kNm/m)
Base	underwater	4.50	0.65	13.90	2.25	40.66	91.48
Wall 1	underwater	0.45	2.60	13.90	0.23	16.26	3.66
Wall 2	above water	0.45	0.75	24.00	0.23	8.10	1.82
Soil 1	underwater	4.05	2.60	10.00	2.48	105.30	260.62
Soil 2	above water	4.05	0.75	18.00	2.48	54.68	135.32
Soil 3	above water	4.50	0.20	18.00	2.25	16.20	36.45
						241.20	529.35

Anchor plate	Vertical force	241.20 kN/m	(= W)
	Moment	13.34 kNm/m	(= $W * B/2 - Wx$ )

### 4. Soil Spring and Center of Rotation

SPT-N values	In front of the plate	N =	5 (sand)
	Below the plate	N =	5 (rock)
Soil Spring	Horizontal (Plate)	KH =	10,000 kN/m <sup>3</sup> ... = 2000 N
	Vertical (base reaction)	KV =	10,000 kN/m <sup>3</sup> ... = 2000 N
Center of rotation	Center of rotation and rotation angle are assumed as follows:		

Vertical distance	y	-0.867 m
Horizontal distance	e	1.838 m
Rotation anglr	$\theta$	0.002662 rad



## 5. Soil Reaction Forces

Passive side soil reaction	Elev. (m)	h (m)	y' (m)	pp * note 1 (kPa)	KH y' $\theta$ * note 2 (kPa)	pp' * note 3 (kPa)	Pp' (kN/m)	y (m)	Mp (kNm/m)
Plastic	4.50	0.00	3.63	0.00	96.72	0.00	0.00	4.20	0.00
	4.50		3.63	0.00	96.72	0.00	0.00	4.20	0.00
Plastic	4.50	0.20	3.63	0.00	96.72	0.00	0.00	4.13	0.00
	4.30		3.43	17.30	91.40	17.30	1.73	4.07	7.03
Plastic	4.30	0.66	3.43	17.30	91.40	17.30	5.67	3.78	21.43
	3.64		2.78	73.96	73.96	73.96	24.22	3.56	86.31
Elastic	3.64	0.09	2.78	73.96	73.96	73.96	3.51	3.31	11.63
	3.55		2.68	82.17	71.43	71.43	3.39	3.28	11.13
Elastic	3.55	0.00	2.68	82.17	71.43	71.43	0.00	3.25	0.00
	3.55		2.68	82.17	71.43	71.43	0.00	3.25	0.00
Elastic	3.55	3.25	2.68	82.17	71.43	71.43	116.07	2.17	251.49
	0.30		-0.57	238.33	-15.09	-15.09	-24.53	1.08	-26.57
Below the top of Anchor Wall only							128.33		355.41

Passive side pressure	Horiz. force	128.33 kN/m	(= Pp')
	Vertical force	34.39 kN/m	(= Pp' * tan $\delta$ )
	Moment	432.78 kNm/m	(= Mp + V * B/2)

\* note 1 : Passive earth pressure obtained in section 2.

\* note 2 : Soil reaction obtained based on soil strain.

\* note 3 : Smaller of pp and KH y'  $\theta$

Base reaction force	p1	108.83 kPa	P1	222.44 kN/m	M1	197.4 kNm/m
	p2	0.00 kPa	P2	0.00 kN/m	M2	0.0 kNm/m
			222.44		197.4	
Effective contact area	A (A')	4.08790852 m <sup>2</sup> /m				
Base friction coefficient	$\mu$	0.60				
Base shear force	Q	133.47 kN/m	(= $\mu * (Pa+P2)$ )			

Base reaction force	Horiz. force	133.47 kN/m	(= Q)
	Vertical force	222.44 kN/m	(= P1+P2)
	Moment	197.39 kNm/m	(= M1+M2)

### Balance of forces

	Horiz force (+) ←	Vert force (+) ↓	Moment (+) ↻
Active earth pressure	58.35	15.63	67.51
Weight of Plate	0.00	241.20	13.34
Tie tension	203.45	0.00	549.31
Passive soil pressure	-128.33	-34.39	-432.78
Base reaction	-133.47	-222.44	-197.39
<b>Total</b>	<b>-0.00</b>	<b>0.00</b>	<b>-0.00</b>

<== All values are nearly ZERO !!

Therefore, assumed center of rotation and rotation angle as shown below are correct !

Vertical distance	y	-0.867 m			
Horizontal distance	e	1.838 m			
Rotation angle	$\theta$	0.0027 rad	<	Allowable rotation angle	0.02 rad OK
Horizontal deflection at tie	$\delta$	4.88 mm	<	Allowable deflection	30 mm OK

## 4. Design of RC Section

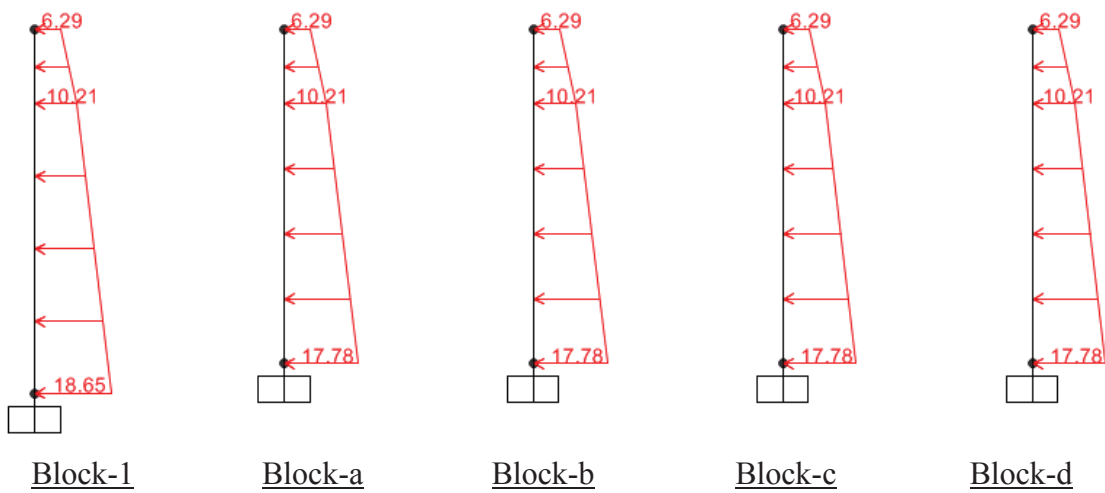
### 4.1 General

From section 3, it is obvious that the design forces under operation (after completion) are greater than that under construction. Therefore, the design of RC section uses the forces under operation.

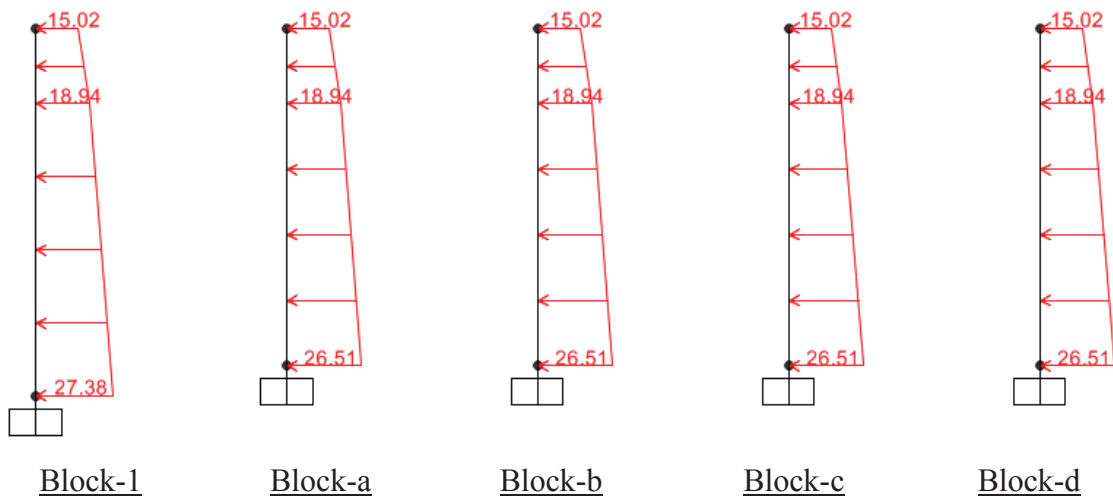
### 4.2 Design force on front wall

#### 1) Design load

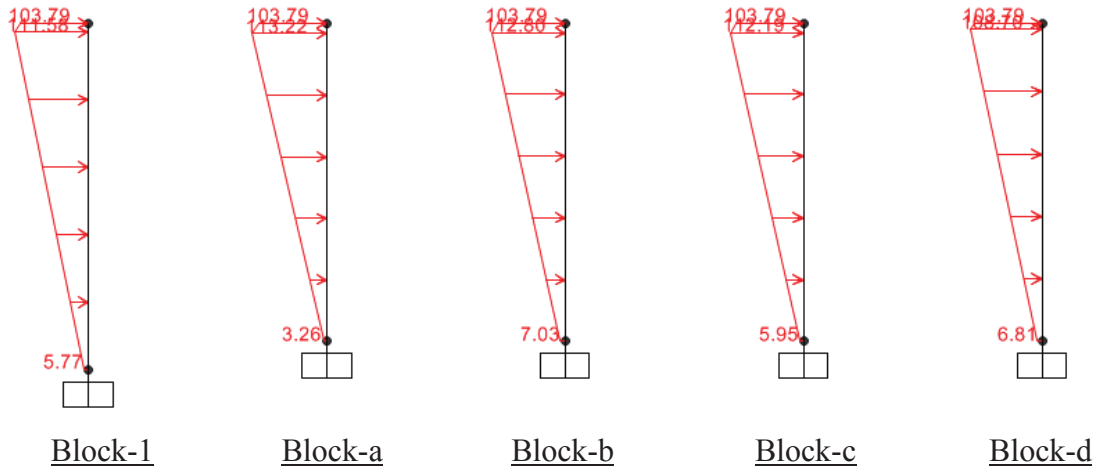
- Active earth pressure ( $w=0\text{kPa}$ ) : Pa0



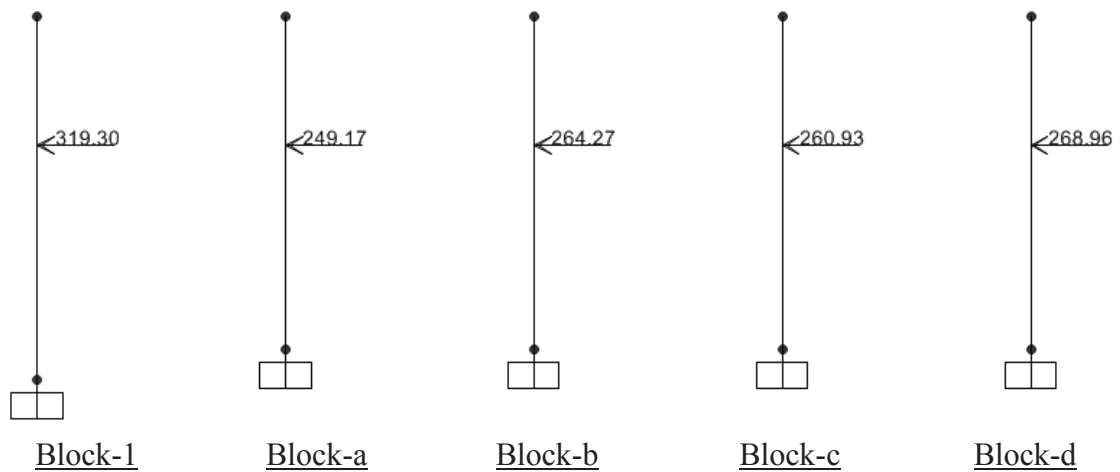
- Active earth pressure ( $w=30\text{kPa}$ ) : Pa30



- Passive earth pressure :  $P_p$



- Tie tension force :  $T$



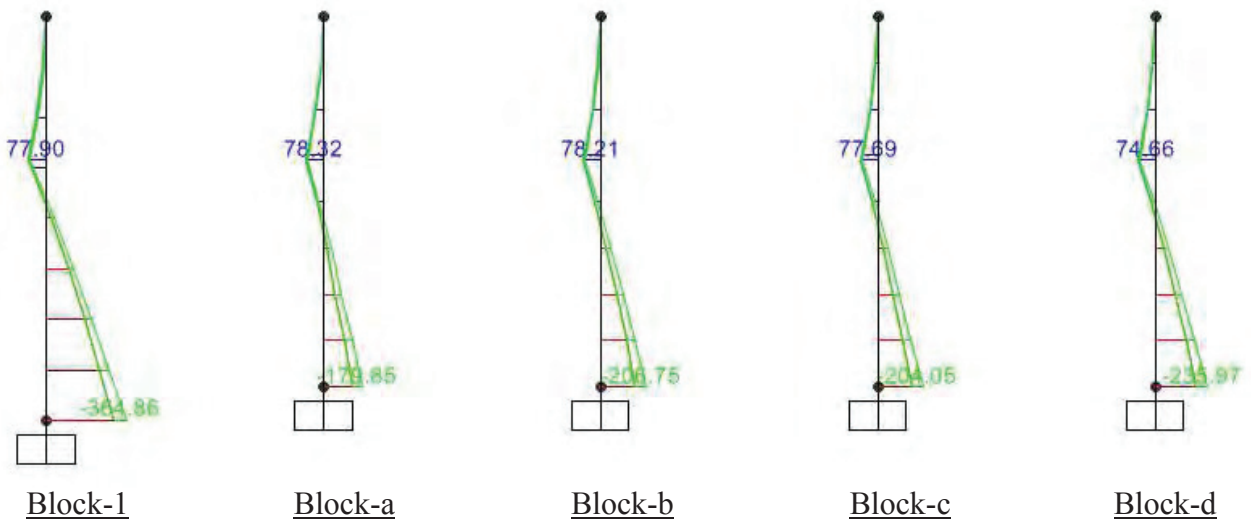
## 2) Analysis results

Analysis was made using SAP2000 for both of the following combinations:

- $Pa_0 + P_p + T$
- $Pa_{30} + P_p + T$

Enveloped results of the above two combinations are shown as follows.

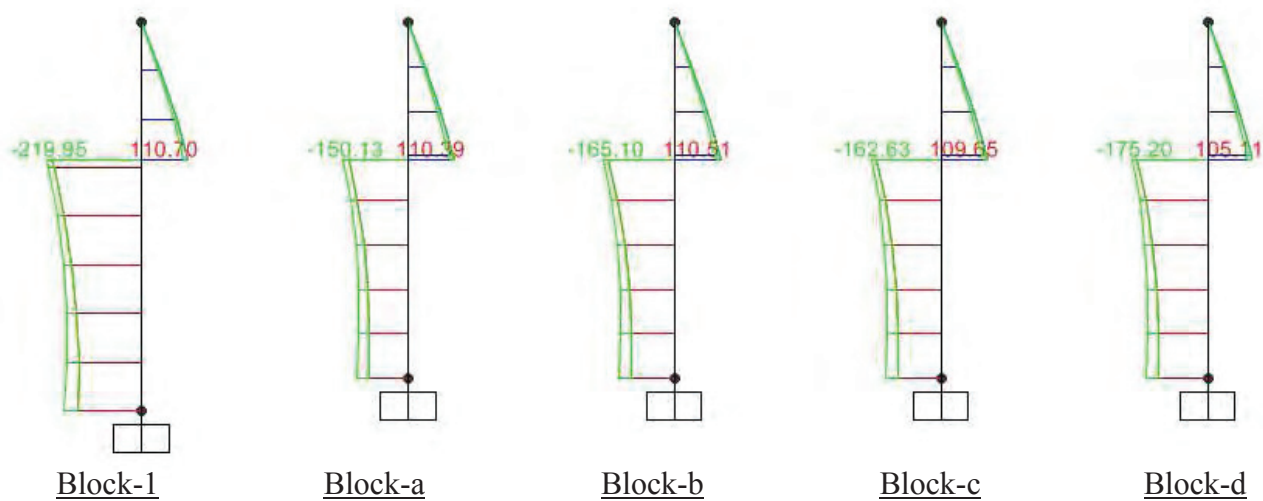
- Bending moment (per meter, envelope)



		Bending Moment kNm/m
Block-1	positive	77.90
	negative	-364.86
Block-a	positive	78.32
	negative	-179.85
Block-b	positive	78.21
	negative	-206.75
Block-c	positive	77.69
	negative	-204.05
Block-d	positive	74.66
	negative	-235.97

note : positive moment induces tensile stress in seaside rebar.  
negative moment induces tensile stress in landside rebar.

- Shear force (per meter, envelope)

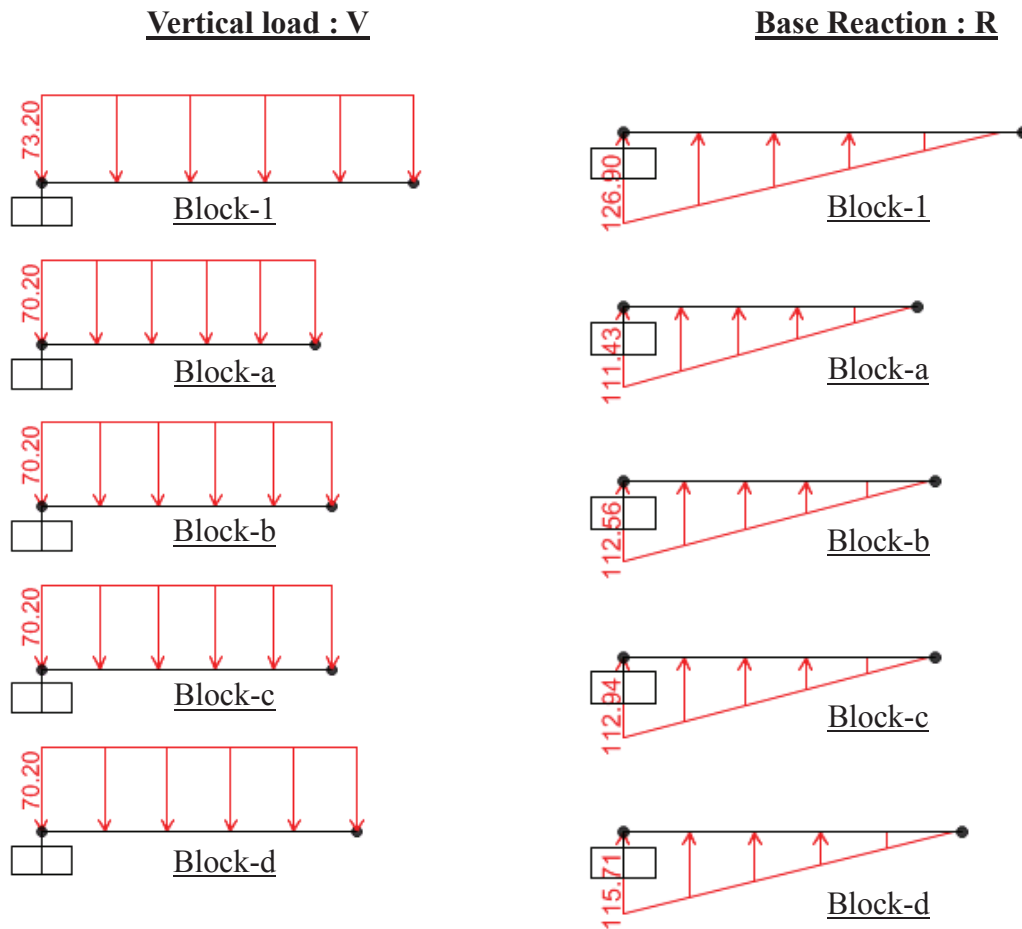


	Shear Force kN/m
Block-1	219.95
Block-a	150.13
Block-b	165.10
Block-c	162.63
Block-d	175.20



### 4.3 Design force on base slab

#### 1) Design load



\* Vertical load:

Block-1 :  $w = (5.5-3.55) \times 18 + (3.55-0.65) \times 10 + (0.65-0.0) \times (24-10) = 73.2 \text{ kPa}$

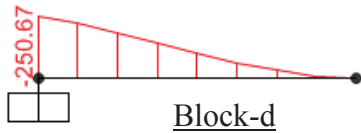
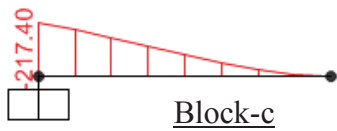
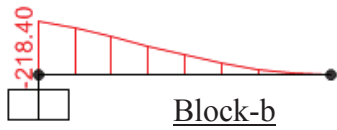
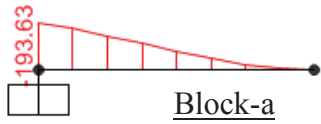
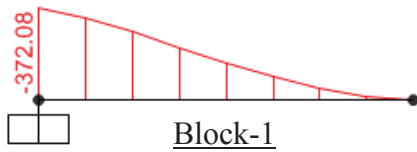
Block-a, b, c & d :  $w = (5.5-3.55) \times 18 + (3.55-0.95) \times 10 + (0.95-0.3) \times (24-10) = 70.2 \text{ kPa}$

#### 2) Analysis results

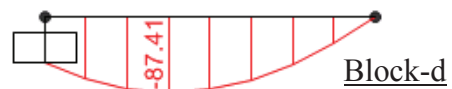
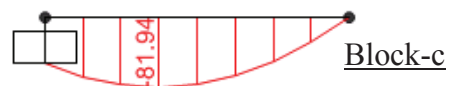
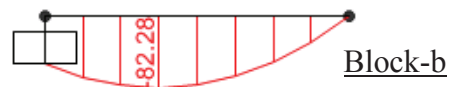
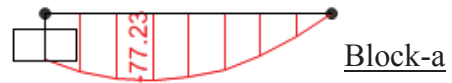
Analysis was made using SAP2000 for the following combination:

➤ V + R

**Bending Moment : M**



**Shear Force : V**

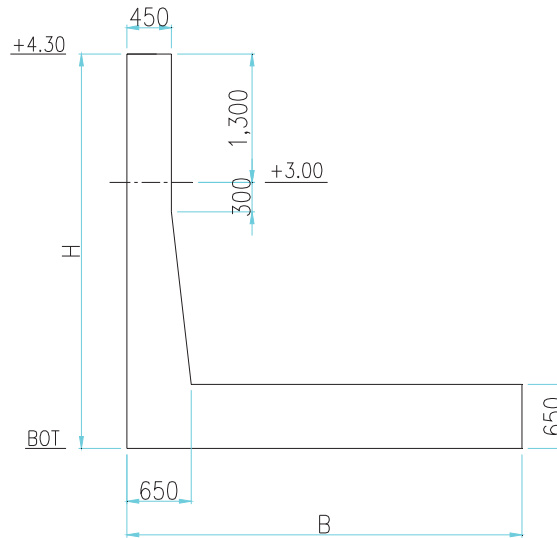


	Bending Moment (kNm/m)	Shear Force (kN/m)
Block-1	-372.08	110.49
Block-a	-193.63	77.23
Block-b	-218.40	82.28
Block-c	-217.40	81.94
Block-d	-250.67	87.41

note : negative moment induces tensile stress in upper rebar.

#### 4.4 RC section check

The dimensions of anchor block are shown below.



	B (mm)	H (mm)	BOT (mCDL)
Block-a	4,000	4,000	+0.30
Block-b	4,200	4,000	+0.30
Block-c	4,200	4,000	+0.30
Block-d	4,500	4,000	+0.30
Block-1	5,200	4,300	±0.00

The concrete sections were checked as following page and the results are all satisfactory.

- For the front wall (seaside rebar) and the base slab (lower rebar)

### Concrete Section Design by Allowable Stress Method

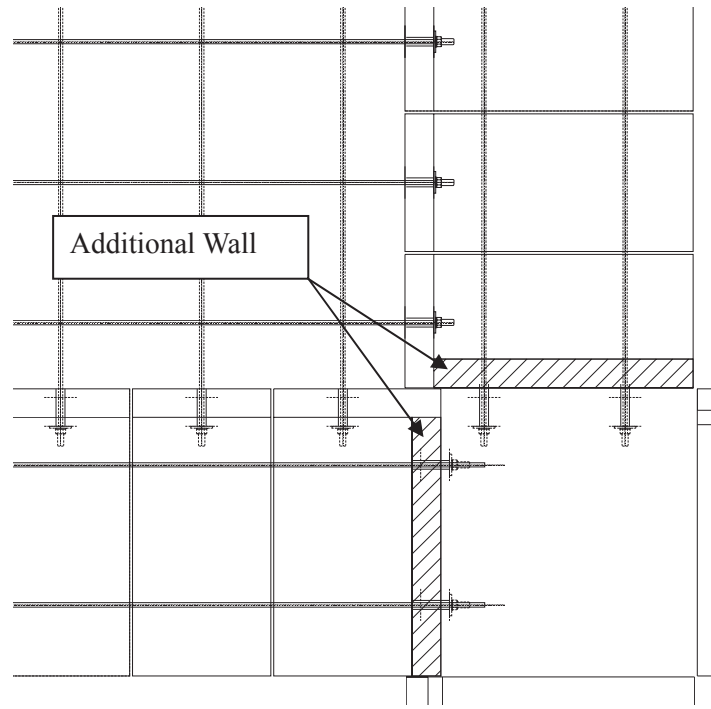
	Sym.	Unit	Block-1	Block-a	Block-b	Block-c	Block-d	
<b>Type of Member</b>								
			Slab	Slab	Slab	Slab	Slab	
<b>Section Property</b>								
Breadth of Section	b	mm	1,000	1,000	1,000	1,000	1,000	
Height of Section	h	mm	450	450	450	450	450	
Effective height for tensile bar	d	mm	364	364	364	364	364	
<b>Design Load</b>								
Bending Moment	M	kNm	77.90	78.32	78.21	77.69	74.66	
Shear Force	S	kN	219.95	150.13	165.10	162.63	175.20	
<b>Allowable Stress - Reinforcement</b>								
Material			SD295	SD295	SD295	SD295	SD295	
Allowable stress	$\sigma_{sa}$	MPa	180	180	180	180	180	
<b>Allowable Stress - Concrete</b>								
Standard Strength 28 days cylinder	$\sigma_{ck}$	MPa	30	30	30	30	30	
Allowable bending compressive stress	$\sigma_{ca}$	MPa	11.0	11.0	11.0	11.0	11.0	
Allowable shear stress without links	$\tau_a$	MPa	1.00	1.00	1.00	1.00	1.00	
Young's modulus ratio	n		7.14	7.14	7.14	7.14	7.14	
<b>Rebar Arrangement</b>								
<Tensile bar>	Bar-1		6.67 -D22	6.67 -D22	6.67 -D22	6.67 -D22	6.67 -D22	
	Bar-2		0 -D16	0 -D16	0 -D16	0 -D16	0 -D16	
	As	mm <sup>2</sup>	2,535	2,535	2,535	2,535	2,535	
	p		0.006966	0.006966	0.006966	0.006966	0.006966	
Cover	C	mm	75	75	75	75	75	
Spacing	Cs	mm	150	150	150	150	150	
<Link>			0 -D13	0 -D13	0 -D13	0 -D13	0 -D13	
	Area	mm <sup>2</sup>	0	0	0	0	0	
	Spacing	mm	200	200	200	200	200	
<b>Working Stress</b>								
Increment Factor for Allowable Stress			1.00	1.00	1.00	1.00	1.00	
Concrete Compressive Stress	Working stress	$\sigma$	MPa	4.79	4.82	4.81	4.78	4.59
	Allowable stress	$\sigma_{ca}$	MPa	11.00	11.00	11.00	11.00	11.00
	Check !!			OK	OK	OK	OK	OK
Steel Reinforcement	Working stress	$\sigma$	MPa	92.74	93.24	93.11	92.49	88.88
	Allowable stress	$\sigma_{sa}$	MPa	180.00	180.00	180.00	180.00	180.00
	Check !!			OK	OK	OK	OK	OK
Shear	Working stress	$\tau$	MPa	0.66	0.45	0.50	0.49	0.53
	Allowable stress	$\tau_a$	MPa	1.00	1.00	1.00	1.00	1.00
	Check !!			OK	OK	OK	OK	OK
<b>Crack Width</b>								
Flexural Crack	Calculated crack width	w	mm	0.263	0.264	0.264	0.262	0.255
	Allowable crack width	wa	mm	0.300	0.300	0.300	0.300	0.300
	Check !!			OK	OK	OK	OK	OK

- For the front wall (landside rebar) and the base slab (upper rebar)

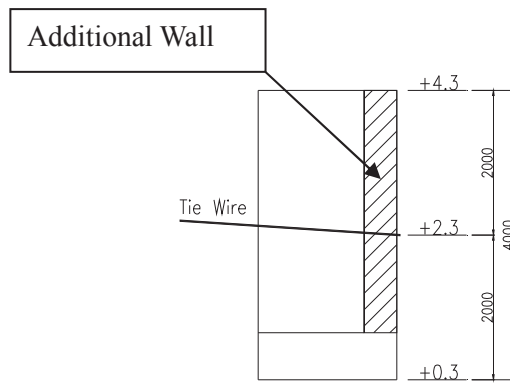
### Concrete Section Design by Allowable Stress Method

	Sym.	Unit	Block-1	Block-a	Block-b	Block-c	Block-d	
<b>Type of Member</b>								
			Slab	Slab	Slab	Slab	Slab	
<b>Section Property</b>								
Breadth of Section	b	mm	1,000	1,000	1,000	1,000	1,000	
Height of Section	h	mm	650	650	650	650	650	
Effective height for tensile bar	d	mm	559.0	564.0	564.0	564.0	562.5	
<b>Design Load</b>								
Bending Moment	M	kNm	372.08	193.63	218.40	217.40	250.67	
Shear Force	S	kN	219.95	150.13	165.10	162.63	175.20	
<b>Allowable Stress - Reinforcement</b>								
Material			SD295	SD295	SD295	SD295	SD295	
Allowable stress	$\sigma_{sa}$	MPa	180	180	180	180	180	
<b>Allowable Stress - Concrete</b>								
Standard Strength 28 days cylinder	$\sigma_{ck}$	MPa	30	30	30	30	30	
Allowable bending compressive stress	$\sigma_{ca}$	MPa	11.0	11.0	11.0	11.0	11.0	
Allowable shear stress without links	$\tau_a$	MPa	1.00	1.00	1.00	1.00	1.00	
Young's modulus ratio	n		7.14	7.14	7.14	7.14	7.14	
<b>Rebar Arrangement</b>								
<Tensile bar>	Bar-1		5 -D32	5 -D22	5 -D22	5 -D22	5 -D25	
	Bar-2		5 -D25	5 -D19	5 -D22	5 -D22	5 -D22	
	As	mm <sup>2</sup>	6,476	3,318	3,801	3,801	4,355	
	p		0.011584	0.005884	0.006740	0.006740	0.007742	
Cover	C	mm	75	75	75	75	75	
Spacing	Cs	mm	100	100	100	100	100	
<Link>			0 -D13	0 -D13	0 -D13	0 -D13	0 -D13	
	Area	mm <sup>2</sup>	0	0	0	0	0	
	Spacing	mm	150	150	150	150	150	
<b>Working Stress</b>								
Increment Factor for Allowable Stress			1.00	1.00	1.00	1.00	1.00	
Concrete Compressive Stress	Working stress	$\sigma$	MPa	8.06	5.29	5.67	5.64	6.20
	Allowable stress	$\sigma_{ca}$	MPa	11.00	11.00	11.00	11.00	11.00
	Check !!			OK	OK	OK	OK	OK
Steel Reinforcement	Working stress	$\sigma$	MPa	115.60	112.90	111.77	111.26	112.94
	Allowable stress	$\sigma_{sa}$	MPa	180.00	180.00	180.00	180.00	180.00
	Check !!			OK	OK	OK	OK	OK
Shear	Working stress	$\tau$	MPa	0.44	0.29	0.32	0.32	0.34
	Allowable stress	$\tau_a$	MPa	1.00	1.00	1.00	1.00	1.00
	Check !!			OK	OK	OK	OK	OK
<b>Crack Width</b>								
Flexural Crack	Calculated crack width	w	mm	0.278	0.279	0.277	0.276	0.277
	Allowable crack width	wa	mm	0.300	0.300	0.300	0.300	0.300
	Check !!			OK	OK	OK	OK	OK

4.5 Check for additional wall of corner location

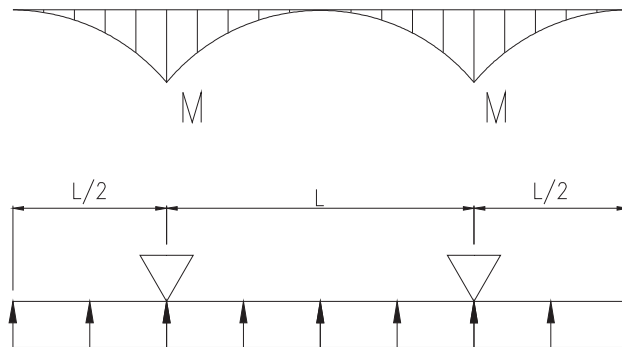


Plan View of Additional Wall



Side View of Additional Wall

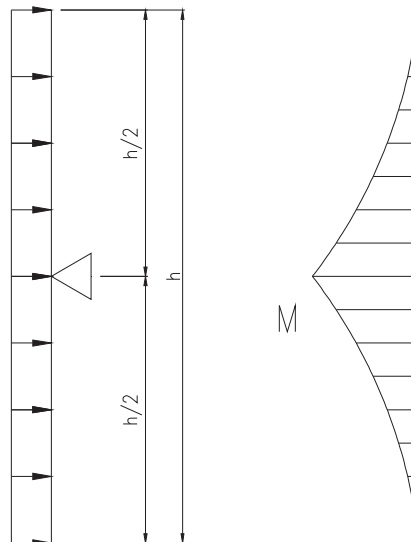
- Horizontal direction



$$M = (T \times L^2) / (8 \times h)$$

	T (kN/m)	L (m)	h (m)	M (kNm)
Block-a	249.17	1.96	4.0	29.91
Block-d	268.96	1.96	4.0	32.29

- Vertical direction



$$M = T \times h / 8$$

	T (kN/m)	h (m)	M (kNm)
Block-a	249.17	4.0	124.59
Block-d	268.96	4.0	134.48

### Concrete Section Design by Allowable Stress Method

	Sym.	Unit	Horizontal Direction	Vertical Direction	
<b>Type of Member</b>					
			<b>Slab</b>	<b>Slab</b>	
<b>Section Property</b>					
Breadth of Section	b	mm	<b>1,000</b>	<b>1,000</b>	
Height of Section	h	mm	<b>450</b>	<b>450</b>	
Effective height for tensile bar	d	mm	<b>345</b>	<b>364</b>	
<b>Design Load</b>					
Bending Moment	M	kNm	32.29	134.48	
Shear Force	S	kN			
<b>Allowable Stress - Reinforcement</b>					
Material			<b>SD295</b>	<b>SD295</b>	
Allowable stress	$\sigma_{sa}$	MPa	180	180	
<b>Allowable Stress - Concrete</b>					
Standard Strength 28 days cylinder	$\sigma_{ck}$	MPa	<b>30</b>	<b>30</b>	
Allowable bending compressive stress	$\sigma_{ca}$	MPa	11.0	11.0	
Allowable shear stress without links	$\tau_a$	MPa	1.00	1.00	
Young's modulus ratio	n		7.14	7.14	
<b>Rebar Arrangement</b>					
<Tensile bar>	Bar-1		<b>6.67 -D16</b>	<b>5 -D22</b>	
	Bar-2		<b>0 -D16</b>	<b>5 -D22</b>	
	As	mm <sup>2</sup>	1,341	3,801	
	p		0.003887	0.010443	
Cover	C	mm	<b>75</b>	<b>75</b>	
Spacing	Cs	mm	<b>150</b>	<b>100</b>	
<Link>			<b>0 -D13</b>	<b>0 -D13</b>	
	Area	mm <sup>2</sup>	0	0	
	Spacing	mm	<b>200</b>	<b>200</b>	
<b>Working Stress</b>					
Increment Factor for Allowable Stress			<b>1.00</b>	<b>1.00</b>	
Concrete Compressive Stress	Working stress	$\sigma_c$	MPa	2.78	7.12
	Allowable stress	$\sigma_{ca}$	MPa	11.00	11.00
	Check !!			<b>OK</b>	<b>OK</b>
Steel Reinforcement	Working stress	$\sigma_s$	MPa	75.03	108.75
	Allowable stress	$\sigma_{sa}$	MPa	180.00	180.00
	Check !!			<b>OK</b>	<b>OK</b>
Shear	Working stress	$\tau$	MPa	0.00	0.00
	Allowable stress	$\tau_a$	MPa	1.00	1.00
	Req'd area of rebar	Aw req	mm <sup>2</sup>	0.00	0.00
	Rebar provided	Aw	mm <sup>2</sup>	0.00	0.00
	Check !!			<b>OK</b>	<b>OK</b>
<b>Crack Width</b>					
Flexural Crack	Calculated crack width	w	mm	0.227	0.271
	Allowable crack width	wa	mm	0.300	0.300
	Check !!			<b>OK</b>	<b>OK</b>



Phụ lục 16-3

Kết quả tính toán biến dạng nền tôn tạo bằng chương trình phân tích GEOASIA

Lach Huyen Port Infrastructure Construction Project

Examination Result on  
Reclaimed Ground Deformations  
Using the analysis program ***GEOASIA***

All Soils All States All Round Geo-analysis Integration

***GEOASIA*** Research Society

## Outline of the analysis

The Lach Huyen Port Infrastructure Construction Project consists of the construction of land reclamation and ground improvement by using PVD and surcharge for the terminal area, and CDM behind the retaining wall. In this analysis, the deformation behavior of the reclaimed ground was examined in consideration of the construction and reclamation histories. The typical cross sections 1-1' and 2-2' shown in Figure.1 were targeted for the analysis under 2-dimentional plane strain conditions. The soil-water coupled finite deformation analysis was employed using the analysis program **GEOASIA**, in which the constitutive equation for the soil skeleton is mounted with the SYS Cam-clay model.

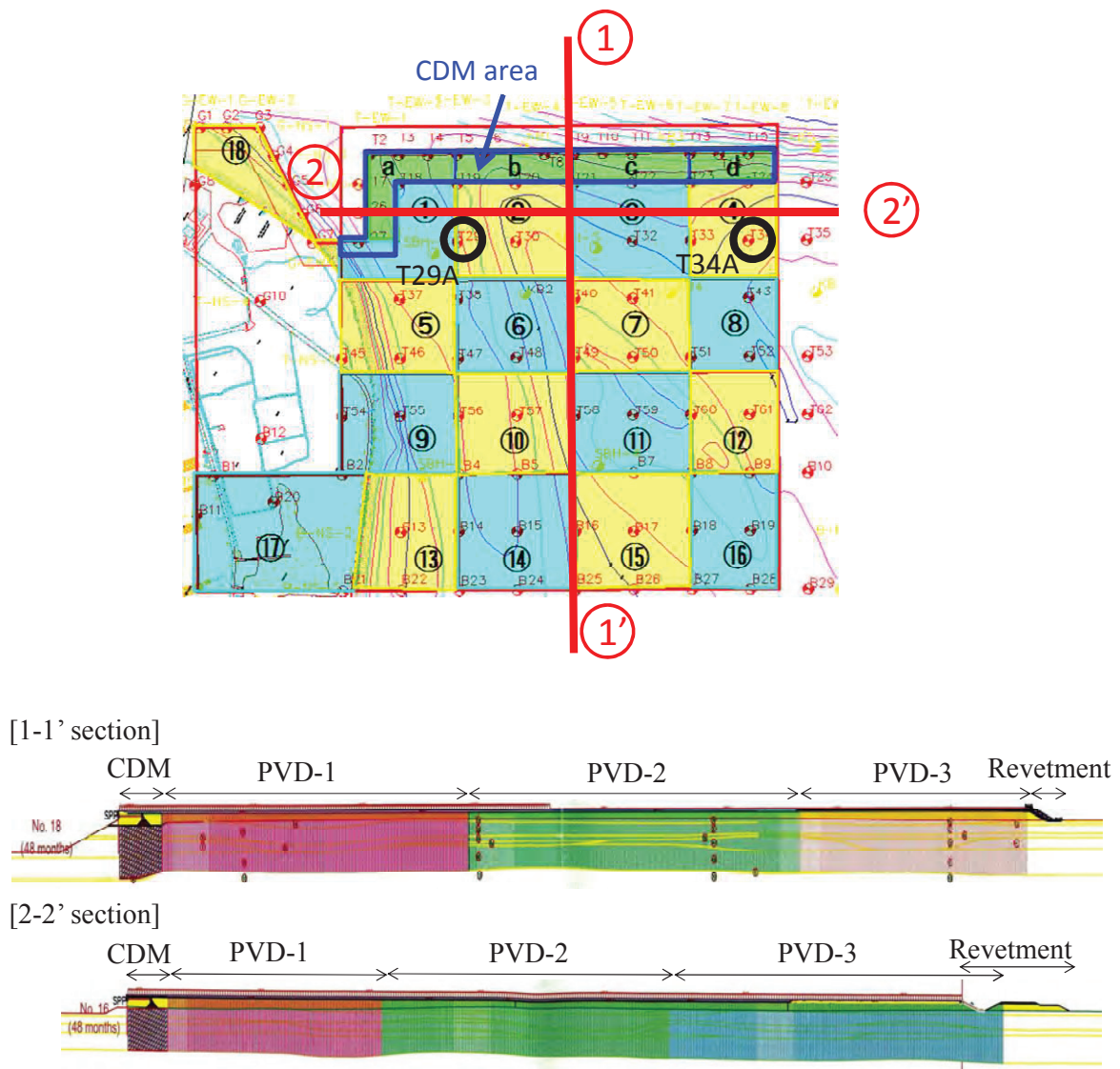


Figure 1. Sections targeted for the analysis

**Determination of the layer structure of initial ground**

Before determining the analysis parameters, the layer structure of the initial ground to be analyzed was decided first by considering the columnar sections. In the site targeted for the analysis, the boring investigations were carried out additionally in 2011 at the points of T29A and T34A in Figure 1, for the purpose of getting the higher accuracy in soil data. In this study, the columnar section of the point T34A, is considered in the determination of the layer structure of initial ground as shown in Figure 2, because the layer-2 which is capable of causing failure and settlement is thicker at the point of T34A than T29A. And, for simplicity, the initial ground was modeled assuming the horizontally stratified conditions (i.e., it was assumed in this analysis that the difference of deformation characteristics between 1-1' section and 2-2' section depends on not the initial ground conditions, but the construction history. The details were described later.)

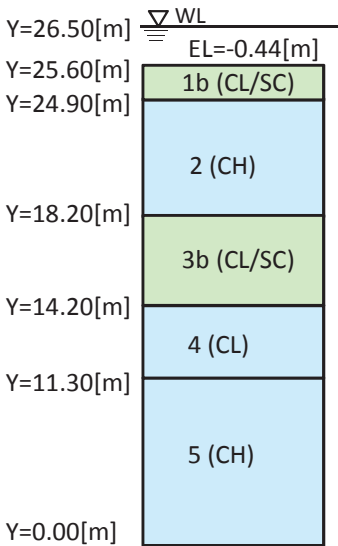


Figure 2. Determination of the layer structure of the initial ground

**Results of laboratory tests and determination of the analytical parameters of initial ground**

The material parameters were determined by simulating the result of the oedometer tests on undisturbed and disturbed specimens as well as those of the undrained triaxial tests of undisturbed specimens through the response of the SYS Cam-clay model. In other words, by using the same material parameters for each layer and by varying only the initial values at the time of commencement of the laboratory tests, those parameters that would describe their behavior best were determined through trial and error.

The results of the laboratory tests and simulations with respect to Layer-1b to 5 are illustrated in Figures 3 to 7. The open circle and the solid lines indicate the laboratory test results, and the simulation results, respectively. In the undrained triaxial test results, the stresses have been converted to show the  $p'-q$  relationship.

From the compression index and sensitivity ratios (the Figures of test results are omitted), it can be said that the soils targeted for analysis are sensitive to disturbance and that the compressibilities of the in-situ soils are higher than those of the specimen of the laboratory tests, especially in Layer-2, and 5. So, the

initial conditions of the in-situ soils were deduced using the oedometer results of the undisturbed samples, giving due the consideration to the effects of disturbance, employing the method proposed by Inagaki et al. (2010), which draws on the Schmertmann's graphic method(1953). In Figures 3 to 7, the purple bold line indicate the deduced in-situ compression curves of initial grounds

Table 1 shows the material parameters and in-situ initial conditions of the each layer of initial ground as determined by the method of computation described above. The permeability of each layer was estimated from the results of the oedometer tests on undisturbed specimens, and for simplicity, was assumed to be unaffected by variations in the pore ratios and always remain constant.

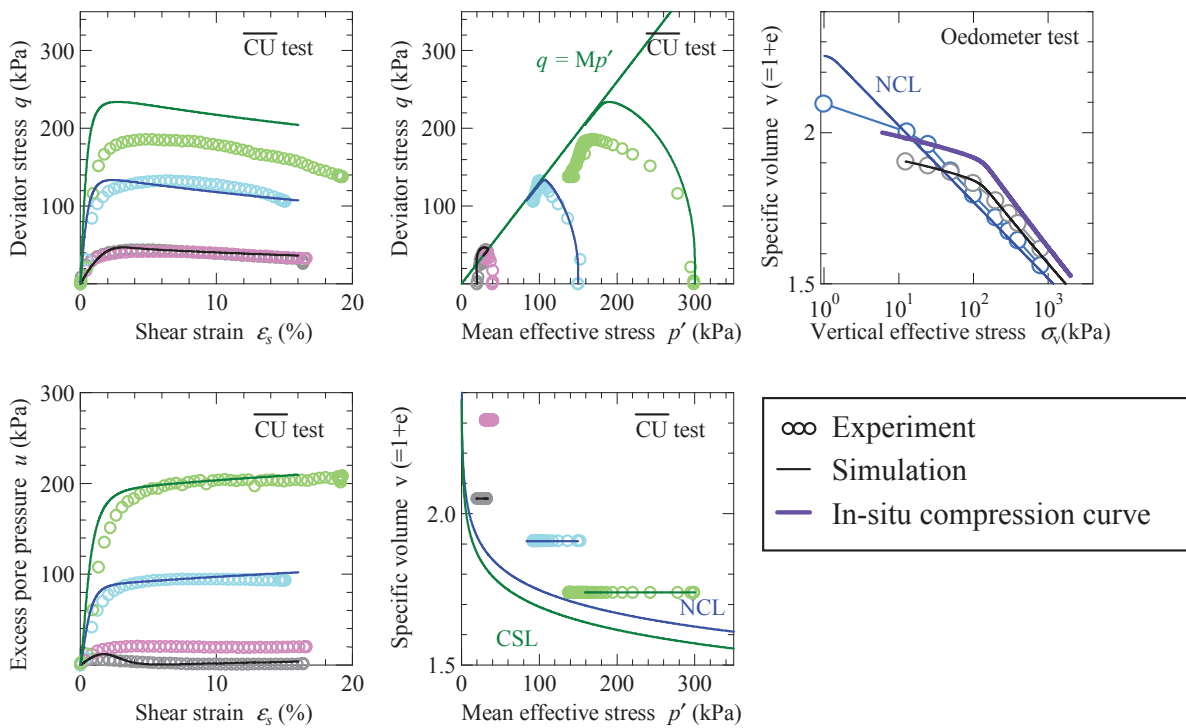


Figure 3. Results of simulation and deduced in-situ compression curve of Layer-1b

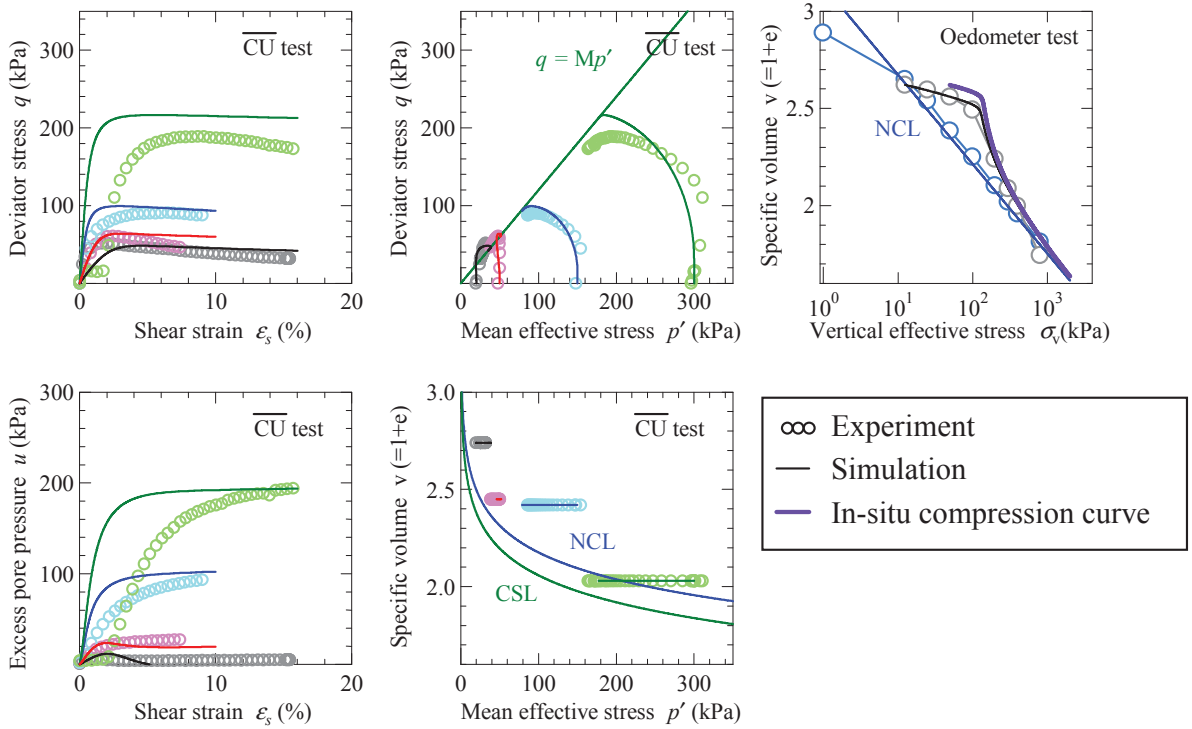


Figure 4. Results of simulation and deduced in-situ compression curve of Layer-2

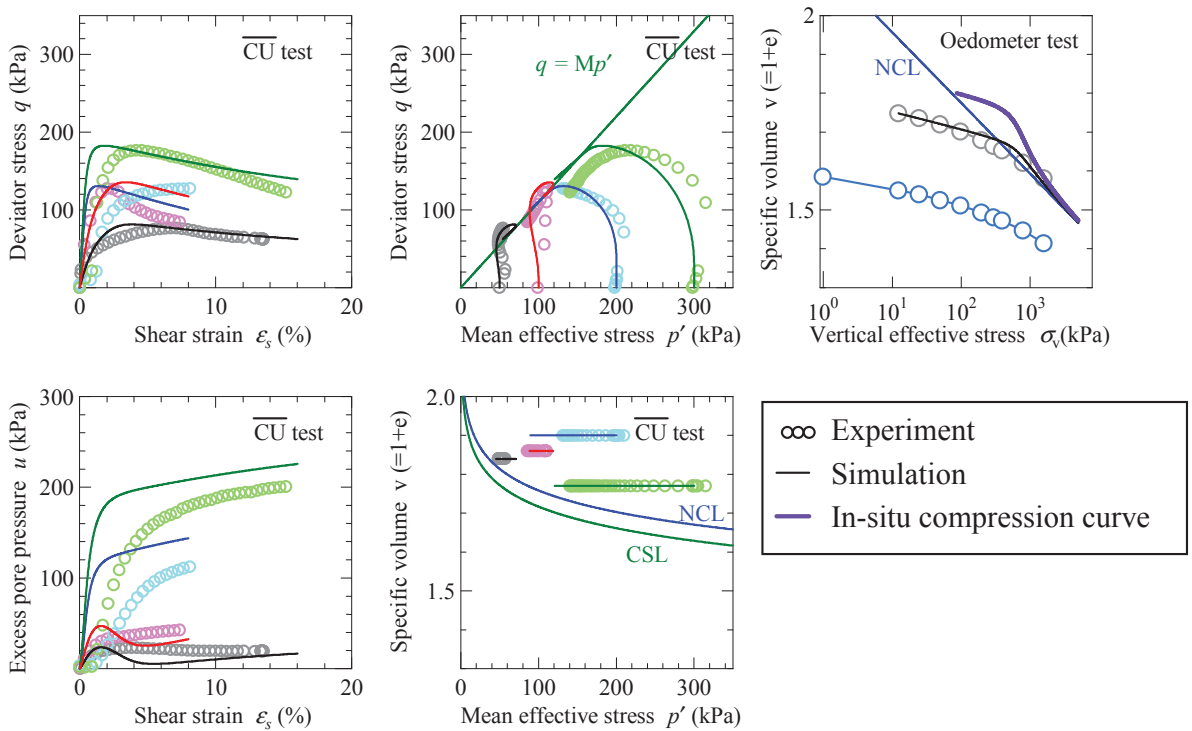


Figure 5. Results of simulation and deduced in-situ compression curve of Layer-3b

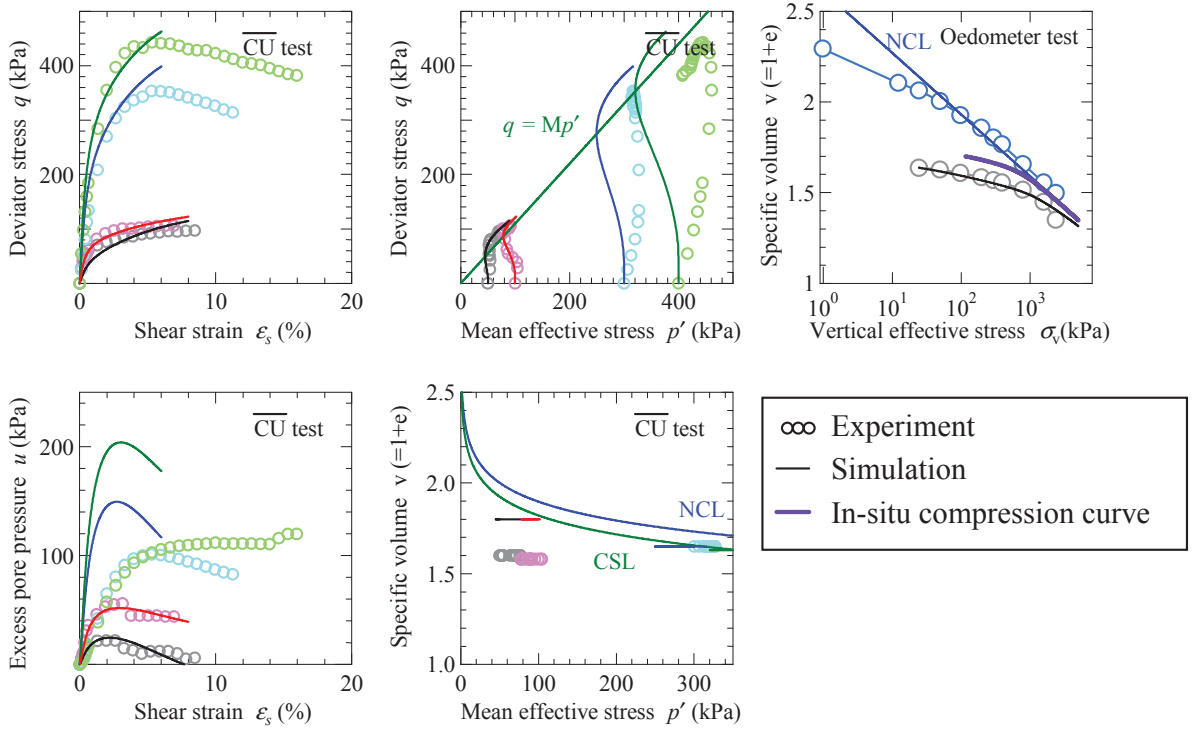


Figure 6. Results of simulation and deduced in-situ compression curve of Layer-4

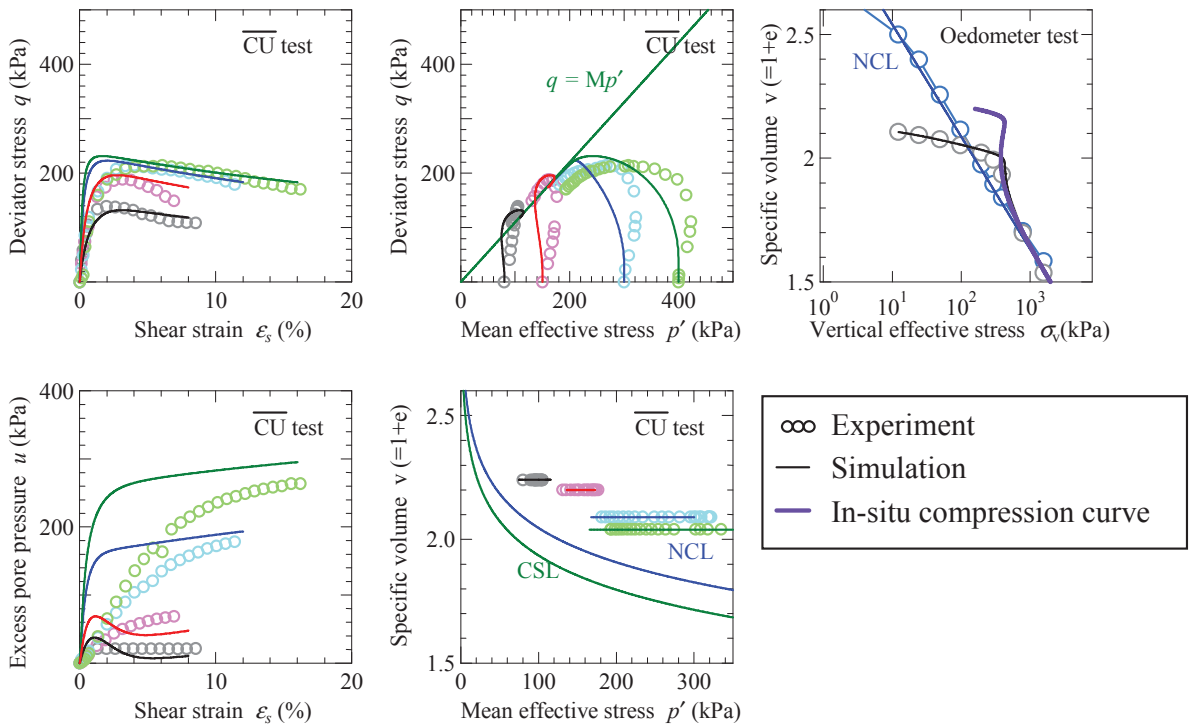


Figure 7. Results of simulation and deduced in-situ compression curve of Layer-5

Table1. Material parameters and deduced in-situ initial conditions of initial ground

	Layer-1b	Layer-2	Layer-3b	Layer-4	Laer-5
Elasto-plastic parameters					
Compressive index $\tilde{\lambda}$	0.11	0.20	0.08	0.15	0.20
Swelling index $\tilde{\kappa}$	0.03	0.05	0.02	0.04	0.03
Critical state constant M	1.30	1.20	1.10	1.10	1.10
NCL intercept N	1.75	2.18	1.76	1.90	2.05
Poisson's ratio $\nu$	0.20	0.20	0.30	0.30	0.10
Evolution parameters					
Normal consolidation index $m$	5.0	10.0	1.0	0.5	10.0
Structural degradation index $a$	0.3	0.8	1.0	0.8	1.5
“ $b$	1.0	1.0	1.0	1.0	1.0
“ $c_s$	0.3	0.1	0.2	0.5	0.2
Rotational hardening index $b_r$	0.01	0.01	0.10	0.10	0.10
Rotational hardening limit $m_b$	1.0	1.0	1.0	1.0	1.0
Permeability $k$ (cm/sec)	$5.0 \times 10^{-7}$	$2.0 \times 10^{-7}$	$1.0 \times 10^{-7}$	$1.0 \times 10^{-7}$	$2.0 \times 10^{-7}$
Density of soil $\rho_s$ (t/m <sup>3</sup> )	2.70	2.71	2.70	2.71	2.71
In- situ conditions					
Void ratio $v_0$	2.00	2.62	1.80	1.70	2.20
Degree of $1/R_0^*$	8.0	15.0	11.0	2.0	10.0
Stress ratio $\eta_0$	0.75	0.75	0.75	0.75	0.75
Degree of anisotropy $\zeta_0$	0.10	0.10	0.10	0.10	0.10

### Determination of the analytical parameters of the materials added on the ground under the constructions

Table 2 and 3 show the analytical parameters of the elasto-plastic materials and the elastic ones respectively, that are added on the initial ground in the process of the reclamation and the ground improvement.

Since laboratory mechanical tests using sampled specimens were not conducted with respect to the materials other than the initial ground, the land reclamation material was assumed to consist of the medium sand with a unit weight of 18.0kN/m<sup>3</sup>. For the rubble mound and the covering block, the material parameters of rock were decided from the results of drain triaxial tests using the crashed granite.

In this analysis, it was assumed that the soil improvement using PVD is equivalent to improve permeability 100 times in comparison with the original soil (but any other parameter is same as in the original).

The soil improvement by CDM and the construction of Anchor wall and Sheet Pile were simulated by the method of replacing the relevant elements as the elastic materials as shown in Tab. 3.



Table 2. Material parameters and initial conditions of the elasto-plastic materials added on the ground under the constructions

	Rock	Sand	After improving using PVD					
			Sand	Layer-1b	Layer-2	Layer-3	Layer-4	Laer-5
Elasto-plastic parameters								
$\tilde{\lambda}$	0.105	0.05	Same as Table 1					
$\tilde{\kappa}$	0.0005	0.012						
M	1.70	1.00						
N	1.90	1.98						
$\nu$	0.3	0.3						
Evolution parameters								
$m$	1.2	0.1	Same as Table 1					
$a$	2.0	2.2						
$b$	1.0	1.0						
$c_s$	1.0	1.0						
$b_r$	1.0	3.5						
$m_b$	0.001	0.7						
$k$ (cm/sec)	$1.0 \times 10^{-3}$	$1.0 \times 10^{-3}$	$5.0 \times 10^{-3}$	$5.0 \times 10^{-5}$	$2.0 \times 10^{-5}$	$1.0 \times 10^{-5}$	$1.0 \times 10^{-5}$	$2.0 \times 10^{-5}$
$\rho_s$ (t/m <sup>3</sup> )	2.593	2.578		Same as Table 1				
In- situ conditions								
$v_0$	1.593	1.88	Same as the conditions before PVD improvement					
$1/R_0^*$	1.0	2.04						
$\eta_0$	0.0	0.75						
$\zeta_0$	0.776	0.13						

Table 3 Elastic material parameters

	CDM (2-phase)	Anchor wall (concrete)	Sheet Pile
Young's modulus E (kN/m <sup>2</sup> )	$3.06 \times 10^4$	$1.50 \times 10^3$	$2.26 \times 10^5$
Poisson's ratio $\nu$	0.45	0.35	0.30
Unit weight (g/cm <sup>3</sup> )	2.0	2.35	7.845
Permeability $k$ (cm/sec)	$1.0 \times 10^{-7}$	—	—

### Construction Schedule on the analysis

The construction and reclamation were simulated by the method of adding/removing elements to/from the ground as needed. Table 4 and 5 show the construction schedules of 1-1' and 2-2' section respectively, which are simplified for analysis.

Table 4 Construction schedule of 1-1' section

Stage	At the end of each stage	Process (area)
No.2	1 month	• Excavation: DL-2.0m (ahead of CDM-area)
No.4	6 months	• Reclamation: CD+4.0m (1/3 of the area from PVD-1 to PVD-2)
No.5	9 months	• Reclamation: CD+4.0m (2/3 of the area from PVD-1 to PVD-2) • Improvement by CDM (CDM-area) • Construction of Sheet Pile (CDM-area)
No.6	12 months	• Construction of rubble mound (CDM-area) • Construction of Anchor wall (CDM-area) • Connection between Sheet Pile and Anchor wall using Tie-rod (CDM-area) • Reclamation: CD+4.0m (PVD-1 area and PVD-2 area)
No.7	15 months	• Improvement by PVD (PVD-1 area) • Sand mat: CD+4.5m (CDM area and PVD-1 area)
No.8	18 months	• Reclamation: CD+4.0m (1/2 of PVD-3 area) • Construction of 1 <sup>st</sup> Preload: CD+5.0m (CDM area and PVD-1 area)
No.9	21 months	• Reclamation: CD+4.0m (PVD-3 area) • Improvement by PVD (PVD-2 area) • Sand mat: CD+4.5m (PVD-2 area) • Construction of 2 <sup>nd</sup> Preload: CD+8.4m ( PVD-1 area)
No.10	24 months	• Construction of 1 <sup>st</sup> Preload: CD+5.0m ( PVD-2 area)
No.11	27 months	• Improvement by PVD (PVD-3 area) • Preload removal: CD+4.5m (PVD-1 area) • Construction of 2 <sup>nd</sup> Preload: CD+7.3 (PVD-2 area) • Construction of 2 <sup>nd</sup> Preload: CD+8.4 (1/2 of PVD-2 area)
No.12	30 months	• Preloading (PVD-3 area) • Preloading (Revetment area) • Excavation: DL-16.0m (ahead of CDM-area)
No.13	33 months	• Preload removal: CD+4.5m (PVD-2 area)
No.15	39 months	• Preload removal: CD+5.5m (PVD-3 area)
No.16	42 months	• Preload removal: CD+4.0m (Revetment area) • Excavation (Revetment area)
No.17	45months	• Construction of rubble mound (Revetment area) • Covering the excavated slope with rubble (Revetment area) • Pavement (CDM, PVD-1, PVD-2 and PVD-3 area)
No.18	48months	• Surcharge load in Service (CDM, PVD-1, and 1/2 of PVD-3 area)

Table 5 Construction schedule of 2-2' section

Stage	At the end of each stage	Process (area)
No.2	1 month	• Excavation: DL-2.0m (ahead of CDM-area)
No.3	3 months	• Improvement by CDM (CDM-area)
No.4	6 months	• Construction of Sheet Pile (CDM-area)
No.5	9 months	• Construction of rubble mound (CDM-area) • Construction of Anchor wall (CDM-area) • Connection between Sheet Pile and Anchor wall using Tie-rod (CDM-area) • Reclamation: CD+4.0m (PVD-1 area and 1/2 of PVD-2 area)
No.6	12 months	• Reclamation: CD+4.0m (PVD-2 area) • Sand mat: CD+4.5m (CDM area and PVD-1 area) • Improvement by PVD (PVD-1 area)
No.7	14 months	• Reclamation: CD+4.0m (PVD-3 area) • Sand mat: CD+4.5m (PVD-2 area) • Improvement by PVD (PVD-2 area) • Construction of 1 <sup>st</sup> Preload: CD+5.0m (CDM area and PVD-1 area)
No.8	16 months	• Sand mat: CD+4.5m (PVD-3 area) • Improvement by PVD (PVD-3 area) • Construction of 1 <sup>st</sup> Preload: CD+5.0m (PVD-2 area) • Construction of 2 <sup>nd</sup> Preload: CD+8.9m ( PVD-1 area)
No.9	19months	• Construction of 1 <sup>st</sup> Preload: CD+5.0m (PVD-3 area) • Construction of 2 <sup>nd</sup> Preload: CD+8.6 (PVD-2 area)
No.10	22months	• Construction of 2 <sup>nd</sup> Preload: CD+8.6 (PVD-3 area) • Preload removal: CD+4.5m (PVD-1 area)
No.11	25months	• Preload removal: CD+4.5m (PVD-2 area) • Excavation (ahead of CDM area)
No.12	28months	• Preload removal: CD+4.5m (PVD-3 area without Revetment area)
No.13	31months	• Preload removal: CD+4.5m (Revetment area) • Excavation (Revetment area)
No.14	33months	• Construction of rubble mound (Revetment area) • Covering the excavated slope with rubble (Revetment area)
No.15	45months	• Pavement (CDM, PVD-1, PVD-2 and PVD-3 area)
No.16	48months	• Surcharge load in Service (CDM, PVD-1, PVD-2 and PVD-3 area)

**Finite element mesh and boundary conditions**

Figure 8 shows the finite element meshes and the boundary conditions of the 1-1' and 2-2' sections which were used for the analysis at the end of constructions (Stage No.16). Although the actual ground inevitably possesses some sort of initial irregularity, the initial ground was modeled assuming the horizontally stratified conditions under two-dimensional plane strain conditions for the sake of simplicity. The ground surface above sea level was taken as the hydraulic boundary and was made to coincide with the water level by making the water pressure always zero. By assuming a hard layer with low-permeability to exist below it, the bottom surface of the ground was taken to be the impermeable boundary. The lateral faces were made to be impermeable boundaries.

Figure 9 shows the outline of constructions. In this analysis, tie-rod, which connects between sheet pile and anchor wall, was expressed by allotting distance invariance constraint conditions to the finite element node intervals under consideration, for simplicity.

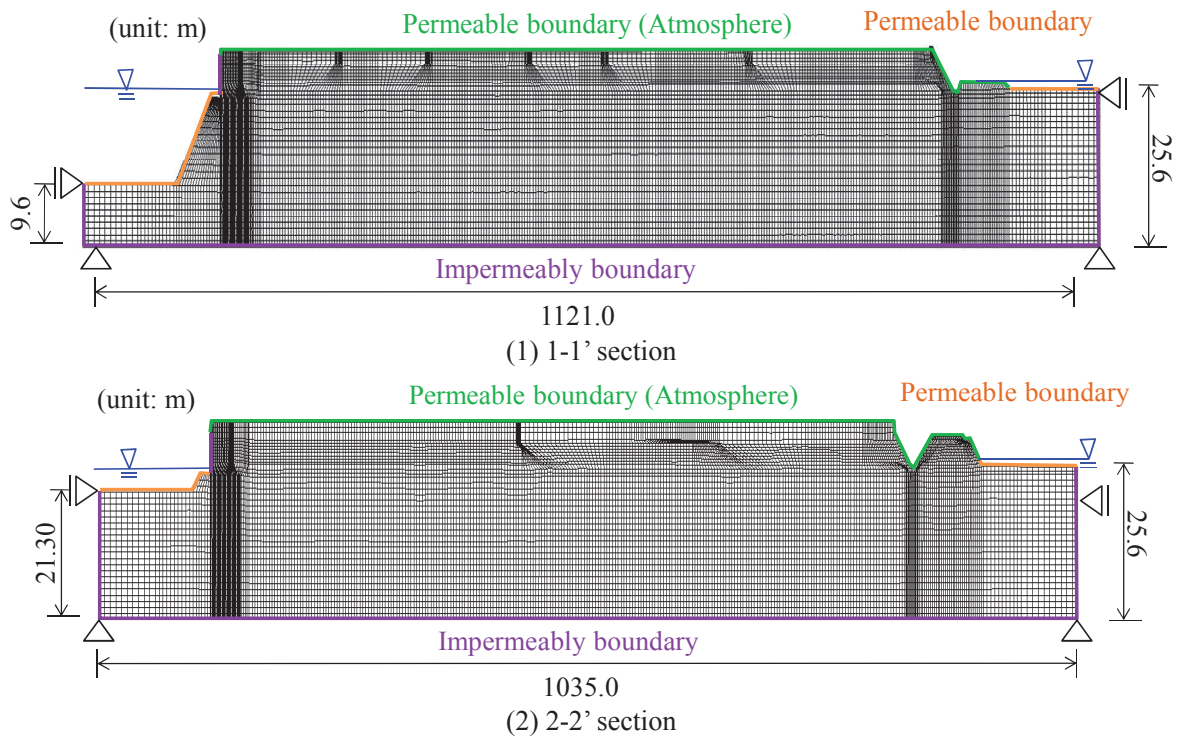


Figure 8. Finite element meshes and boundary conditions

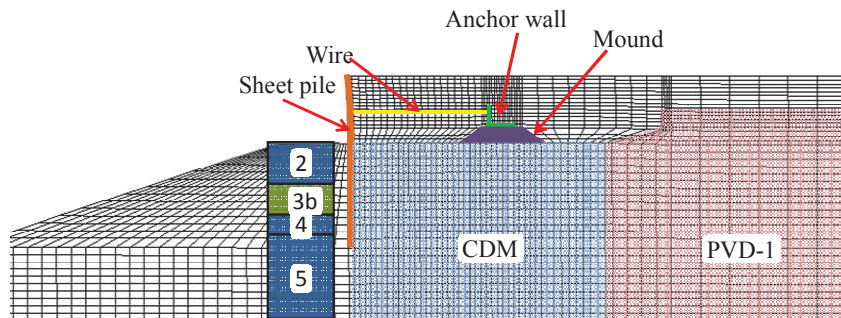
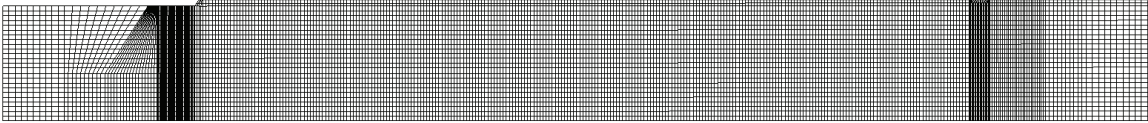


Figure 9. Outline of the constructions around CDM-area.

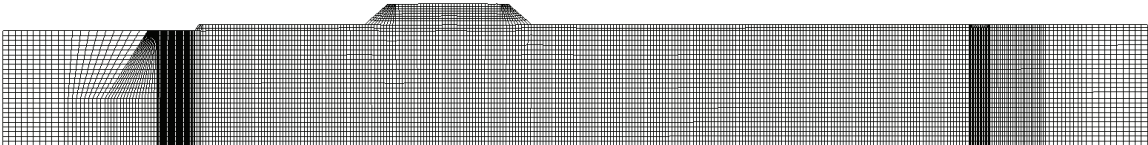
**Results of the Computations**

(1) Finite element mesh at the typical stage

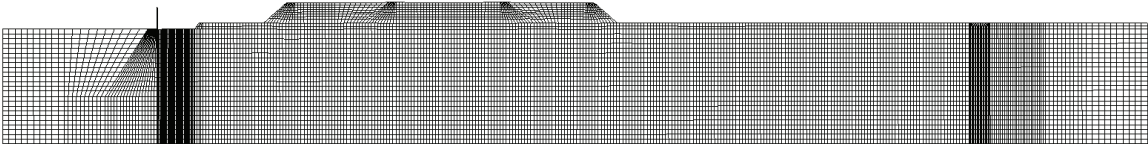
[1-1' section]



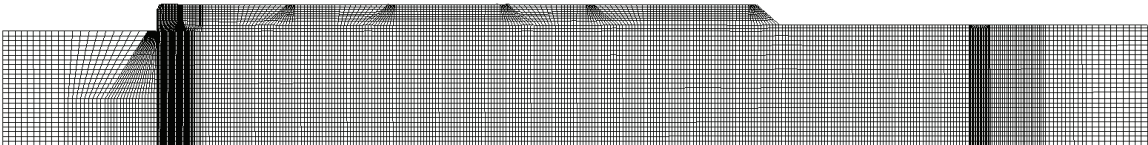
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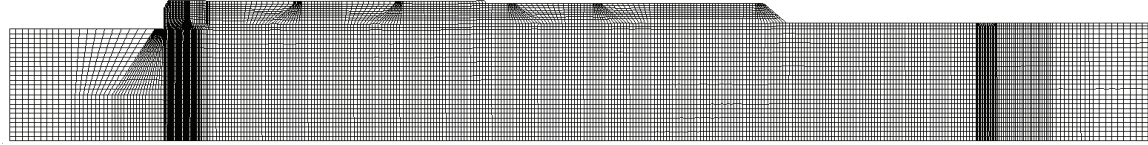
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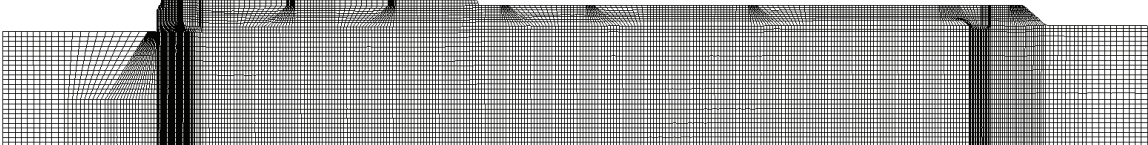
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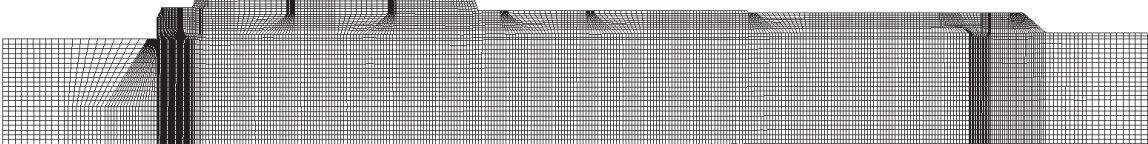
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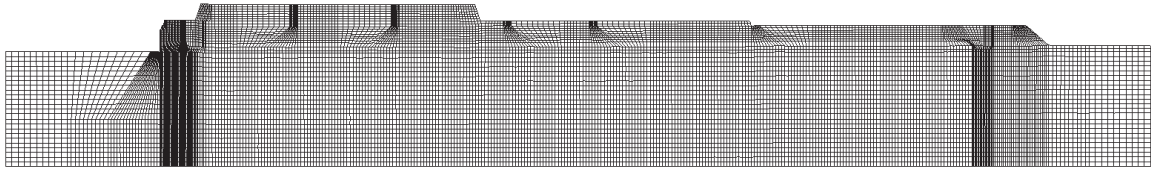
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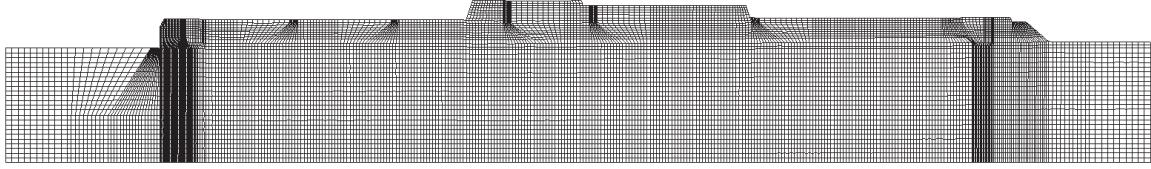
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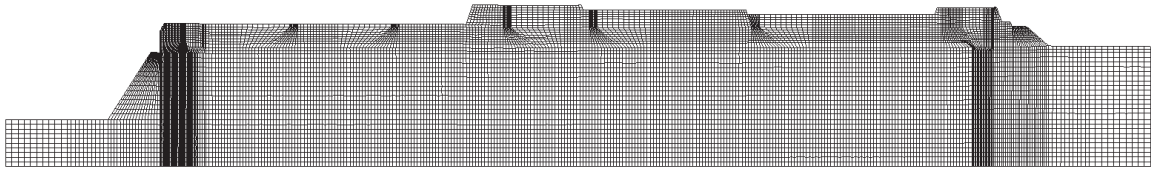
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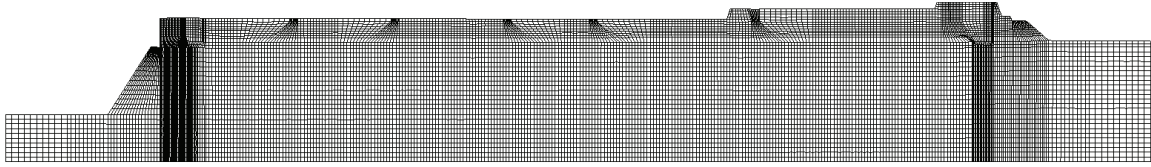
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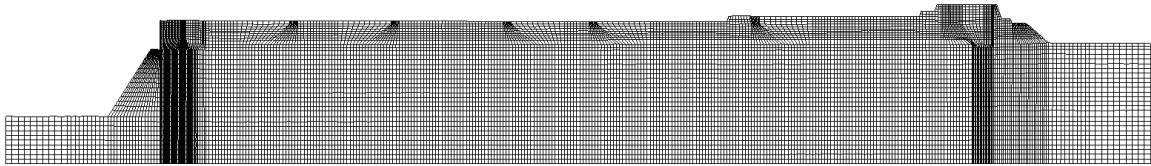
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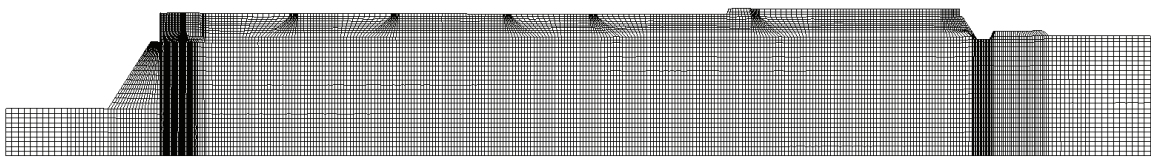
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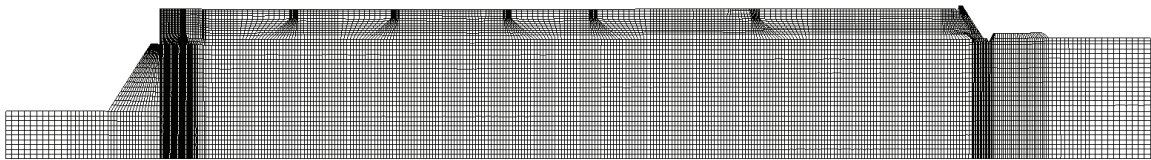
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No.15

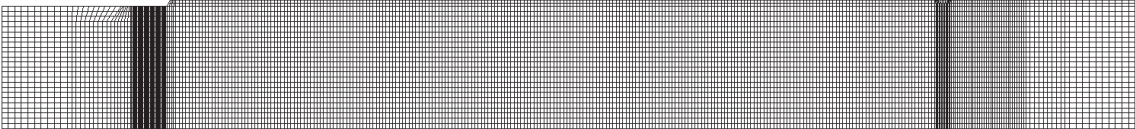


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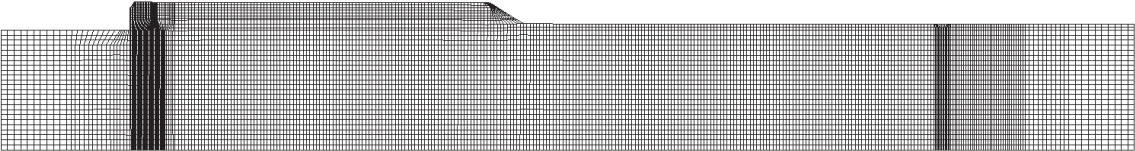


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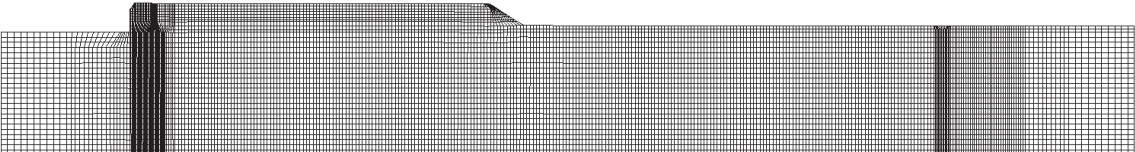
[2-2' section]



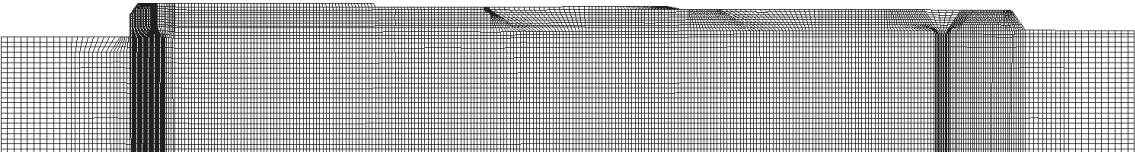
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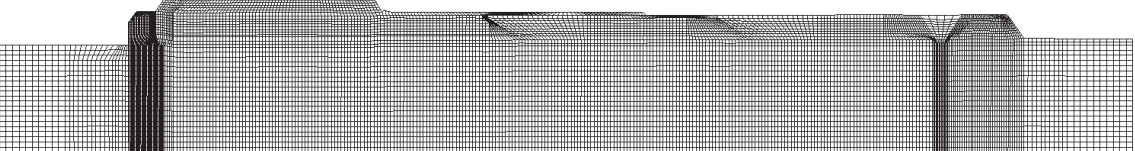
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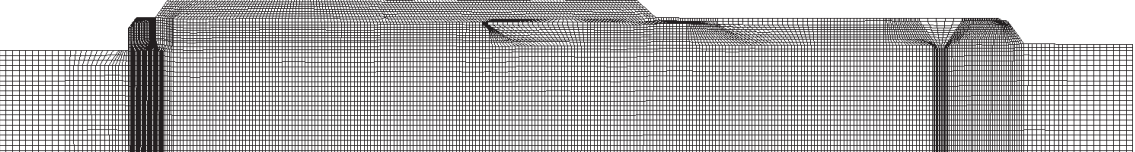
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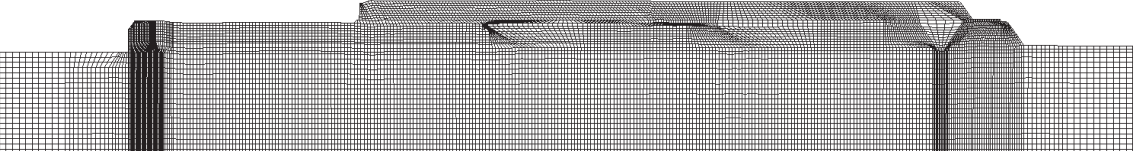
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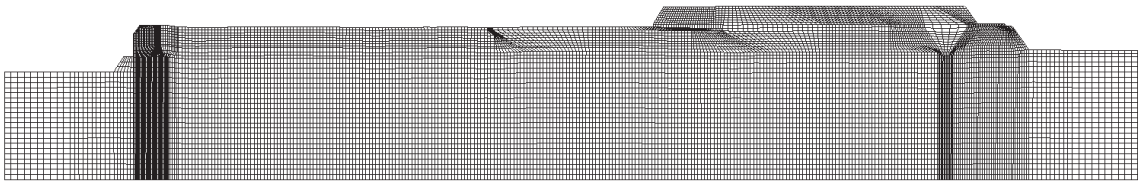
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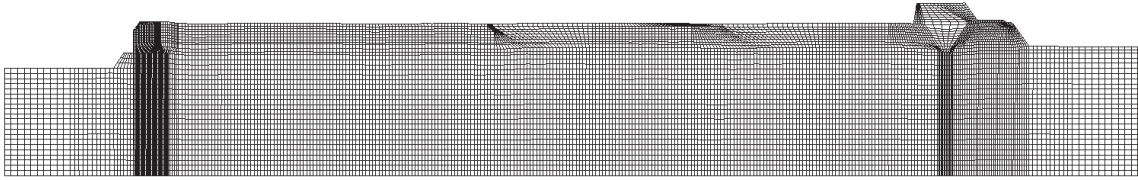
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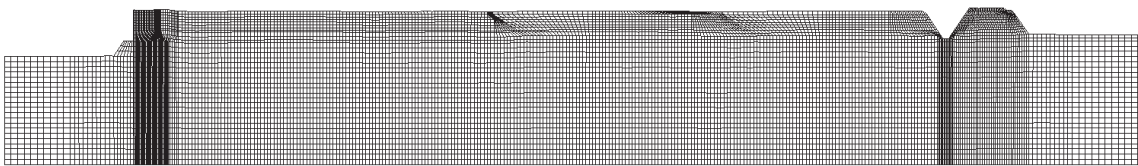
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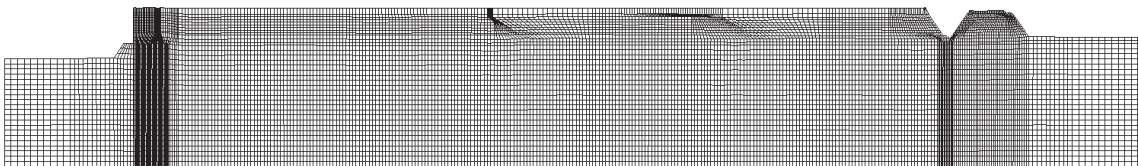
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No.12



No.13

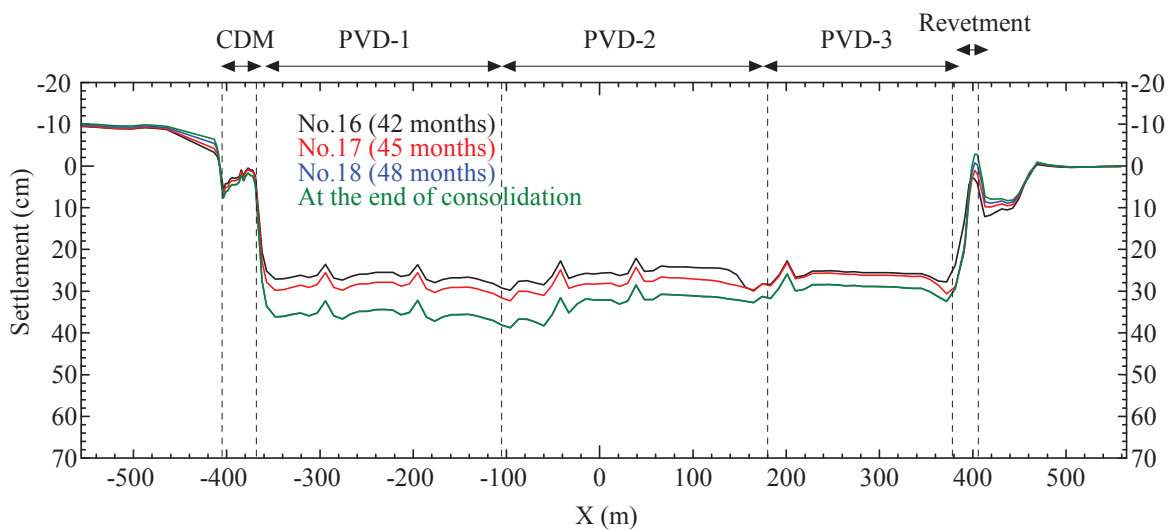
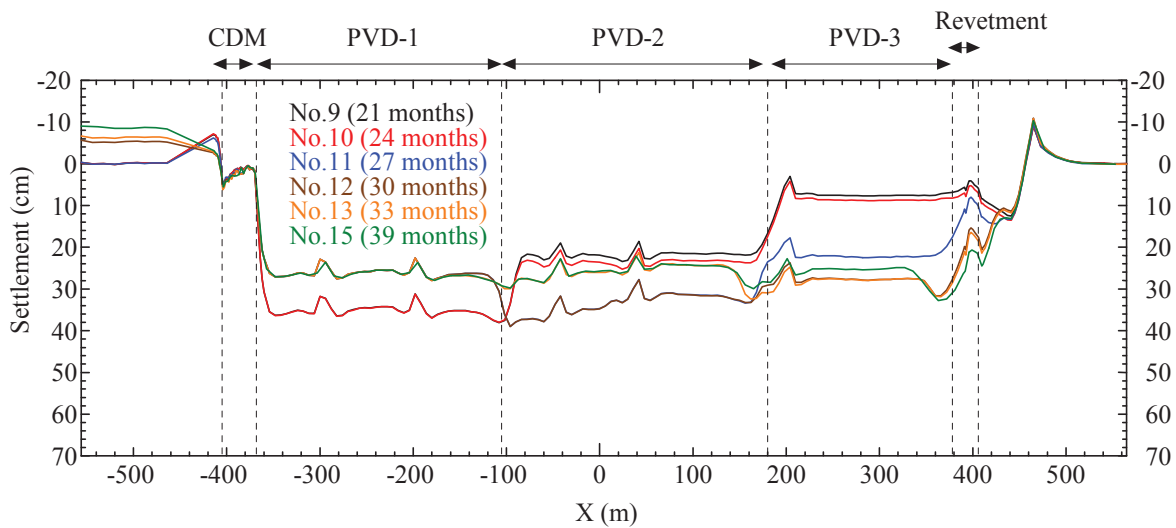
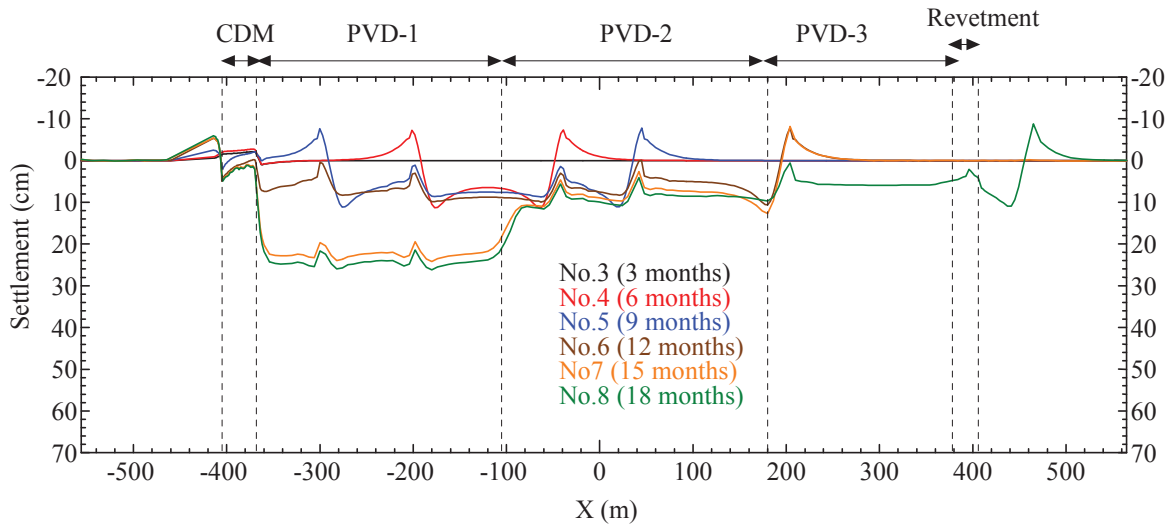


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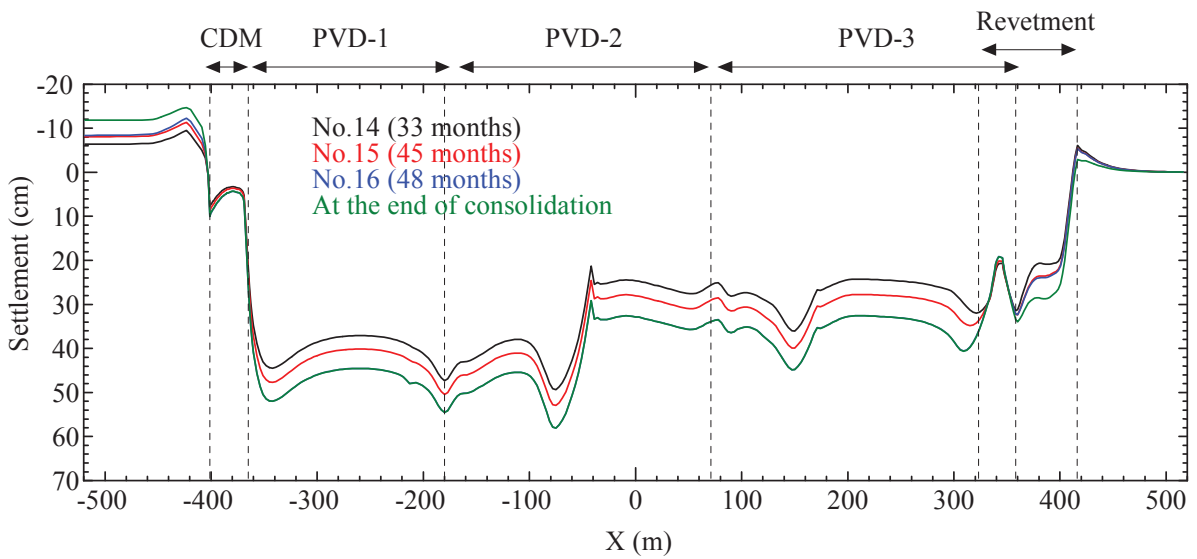
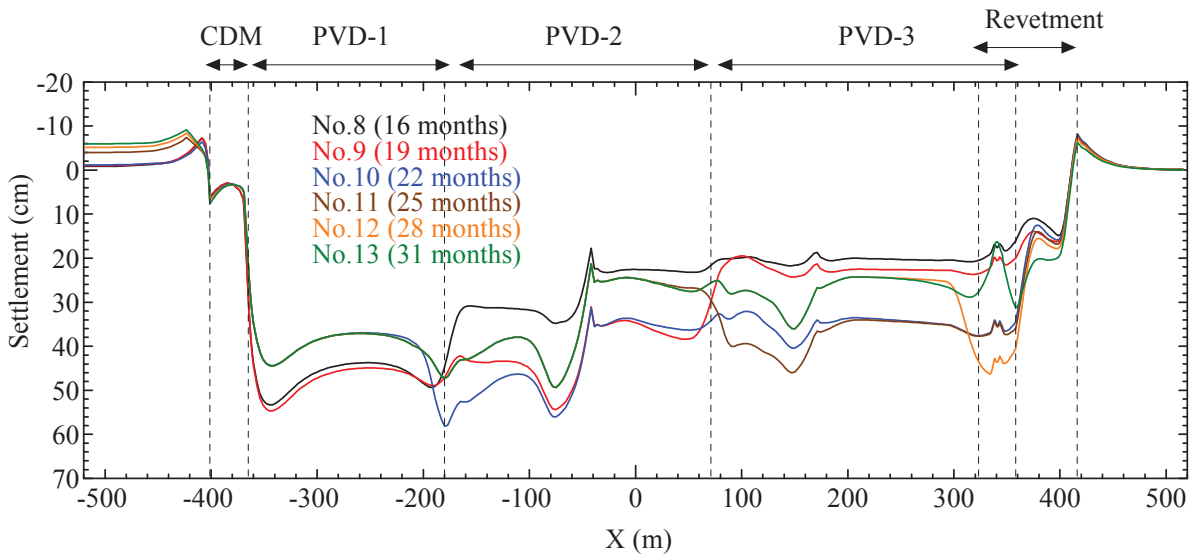
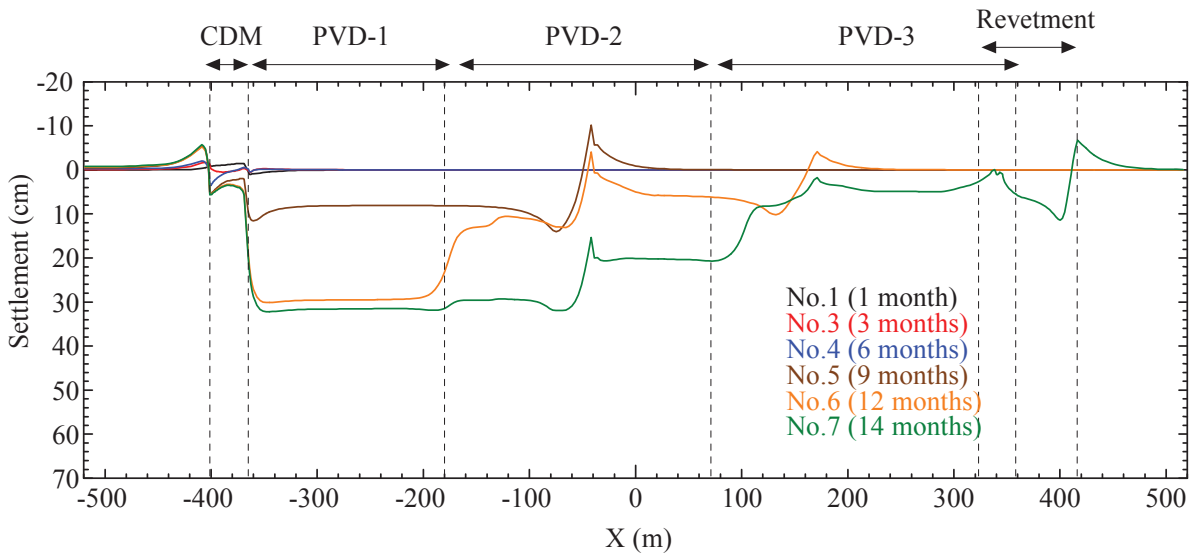


(2) Settlement profiles of ground surface at the end of each stage

[1-1' section]



[2-2' section]



- The ground upheaval by the excavation ahead of CDM-area in maximum

1-1' section = about 10cm

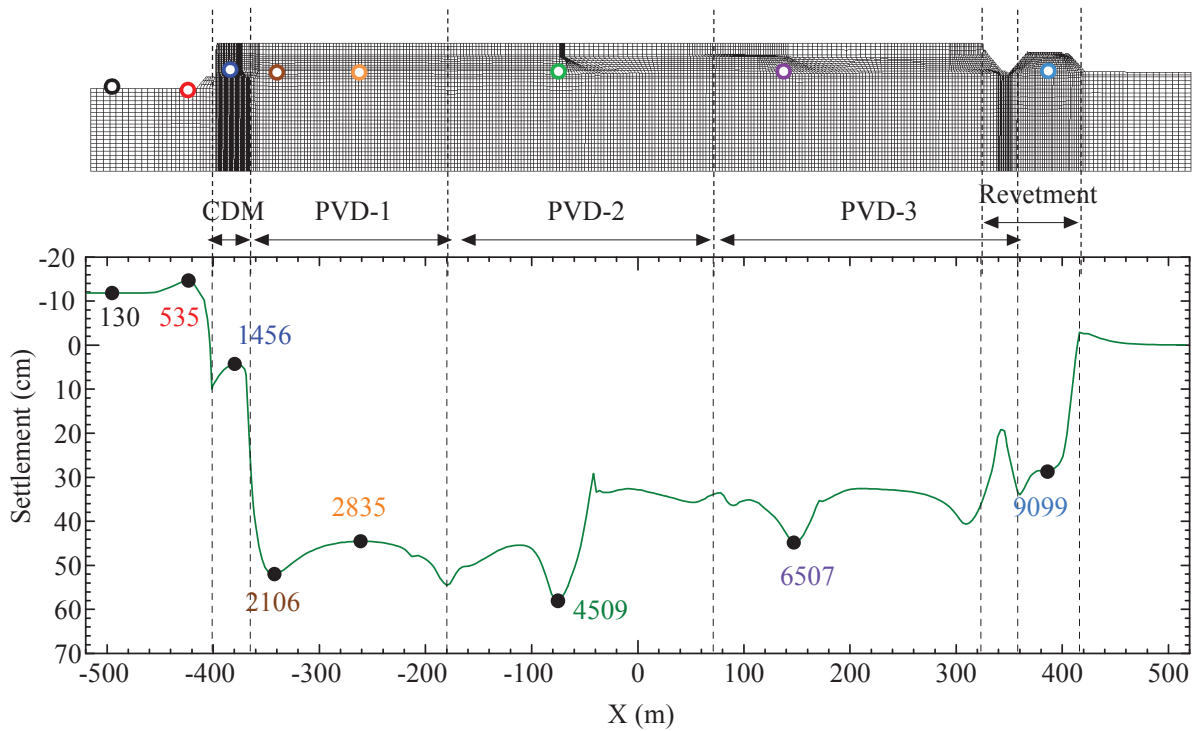
2-2' section = about 15cm

This difference is because the reclamation was started from the area that is near the sea in 2-2' section.

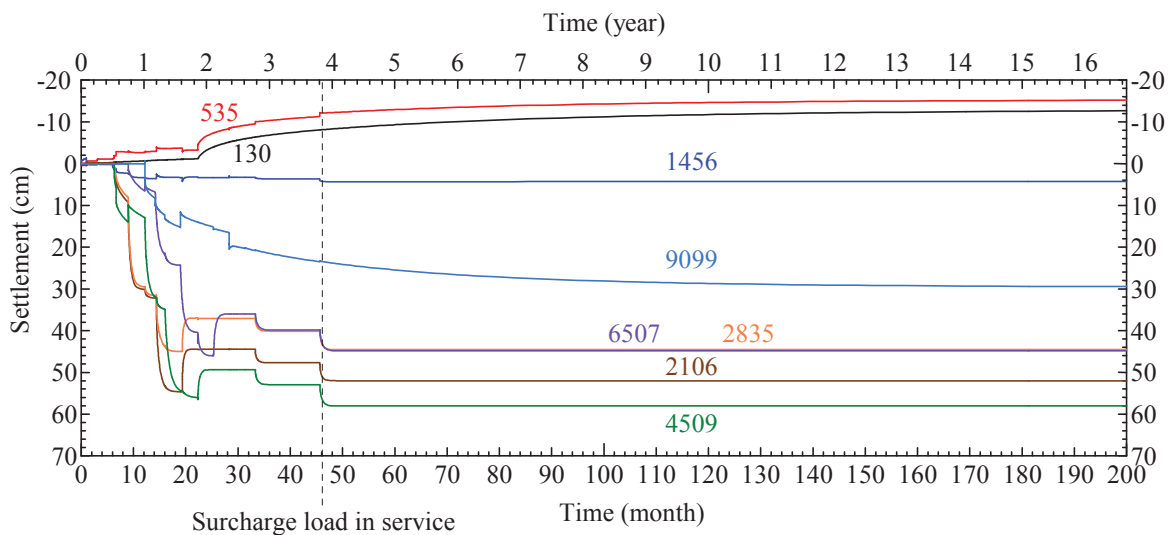
- The settlement of ground surface in CDM-area is the minimum in the reclaimed ground, which ranges from about 5cm to about 10cm.
- In 2-2' section, the largest settlement is obtained in areas from PVD-1 to a half of PVD-2, where the ground reclamation is carried out at early stage. The maximum settlement of about 58cm is obtained at PVD-2 area.
- In 2-2' section, settlement of the ground under the rubble-mound revetment does not subside sooner, because the ground is not improved by PVD.
- Since the previous information provided for this analysis was not sufficient, the reclamation and the excavation are assumed to be carried out under the instant loading and unloading condition, for the safety side evaluation. Because of this, the ground upheaval may be estimated larger than the actual phenomenon.

(3) Time-settlement behaviors at the typical points of ground surface

(as an example, shown the case of 2-2' section)



Location of the typical points and node numbers (2-2' section)

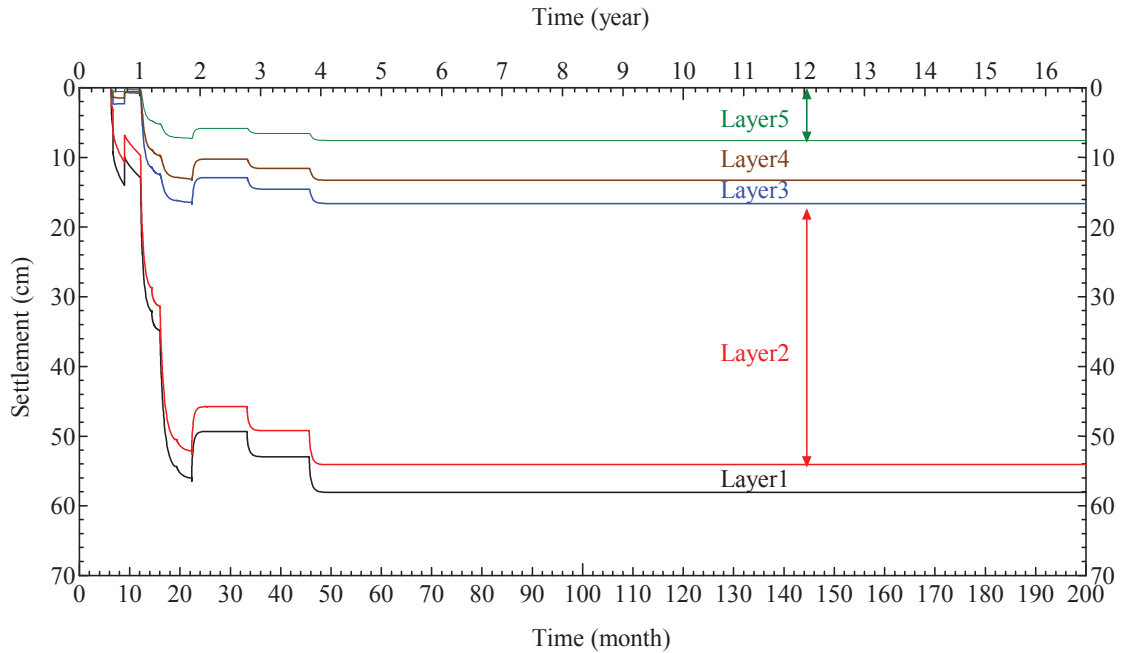


Time-settlement behaviors of the nodes illustrated in the upper figure (2-2' section)

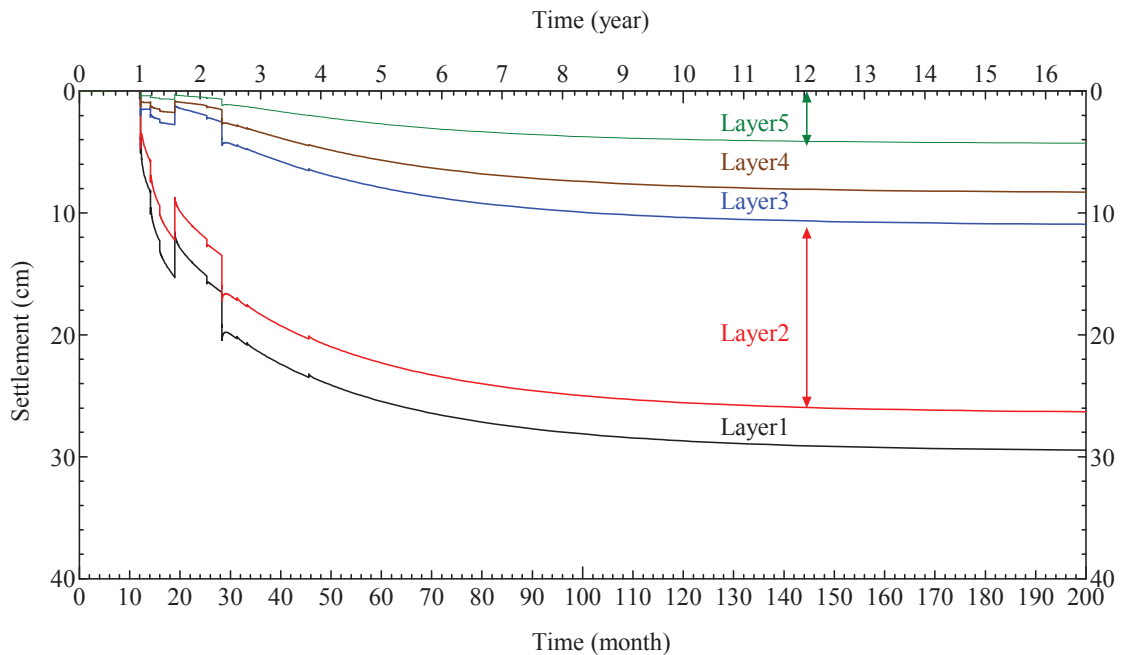
- In the point where the ground is improved by PVD (node-2106, 2835, 4509, 6507), the settlement subsides sooner and the residual settlement after the entry of service is small.
- In the point without PVD (node-535, 130, 9099), the settlement continues over about 10-year period.
- It is better that the ground under the counter fill is also improved by PVD for safety.

(4) Settlement of each layer

(as an example, shown the case of 2-2' section)



Settlement of each layer on the vertical section concluding node-4509 (improved by PVD)



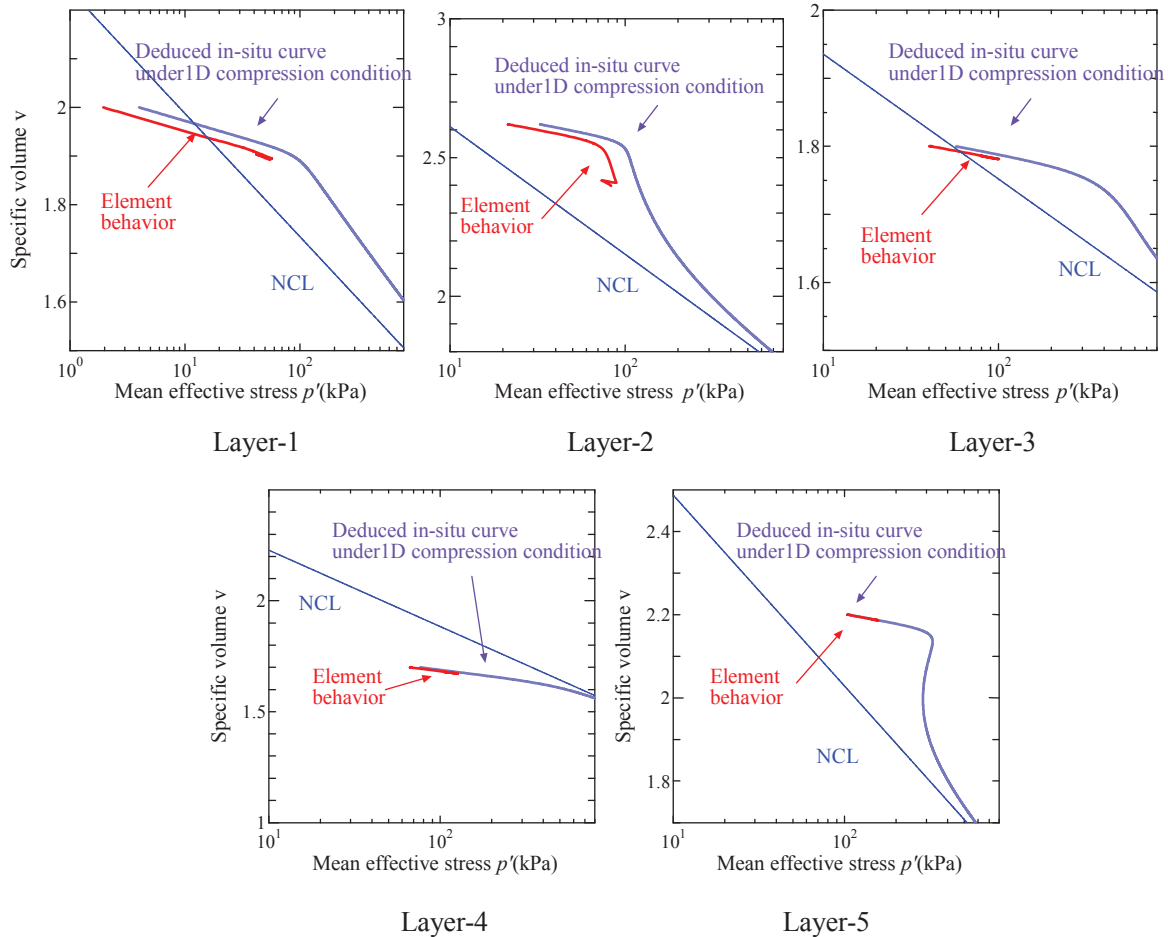
Settlement of each layer on the vertical section concluding node-9099 (without PVD improvement)

- Layer2 is the most dominant layer of ground settlement.
- Although Layer5 is the thickest, its settlement is small. It is expected that the stress states of layer 5 does not exceed the consolidation yield stress by reclamation loading (see next section).
- In the ground improvement by PVD (node-4509), all layer's settlement subside sooner.

- In the ground without PVD improvement, the settlement of deep layers such as Layer4 and Layer5 continues for a long time.

(5) Behaviors of the typical elements at the middle depth of each layer

(as an example, shown the case of 2-2' section)

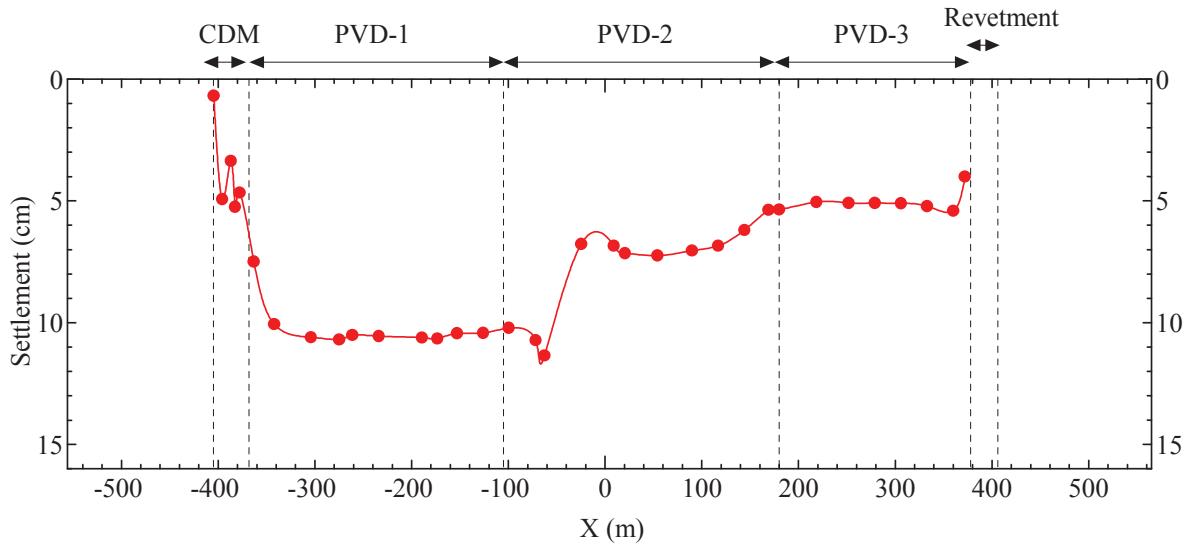


Behaviors of the elements at the middle depth of each layer on the vertical section concluding node-4509

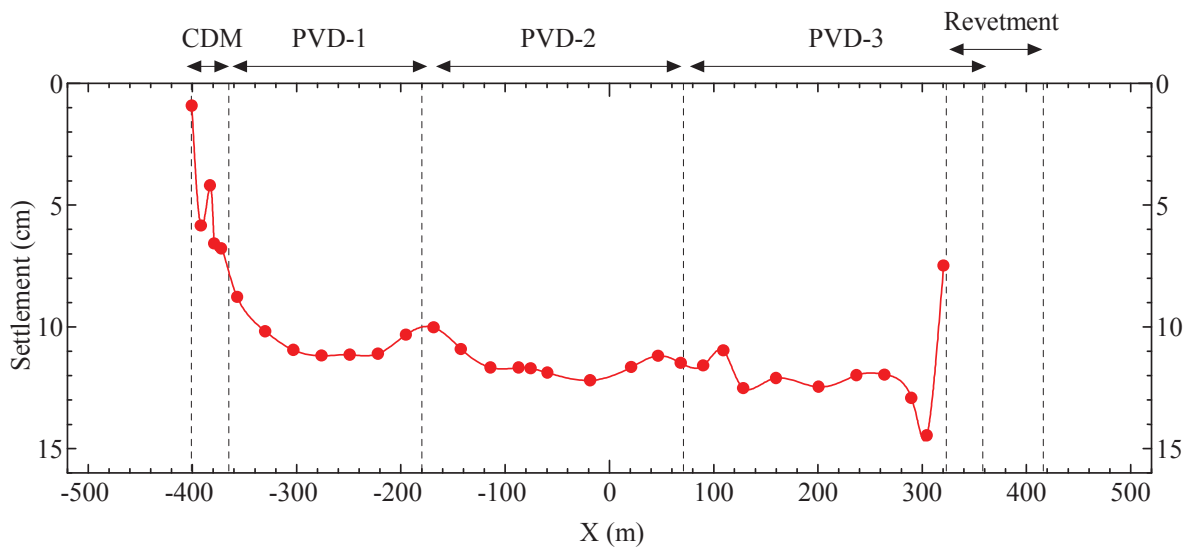
- The deduced in-situ curves illustrated in the each figures by purple lines, are same as those in Figures 3 to 7, which are obtained by the responses of SYS Cam-clay model under 1D compression condition.
- The element behaviors shown by red lines are calculated by **GEOASIA** under the loading histories, such as reclamation, excavation, preloading, and so on.
- In Layer-2 only, the stress state exceeds the consolidation yield stress, and it caused the large compression.

(6) Settlement of the paved surface from the end of construction to the end of consolidation

[1-1' section]



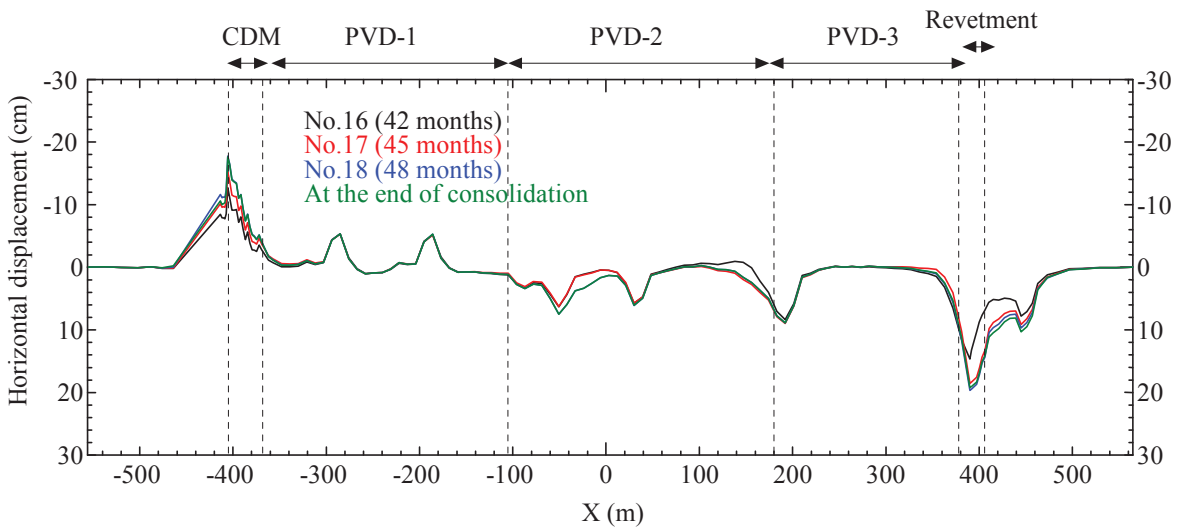
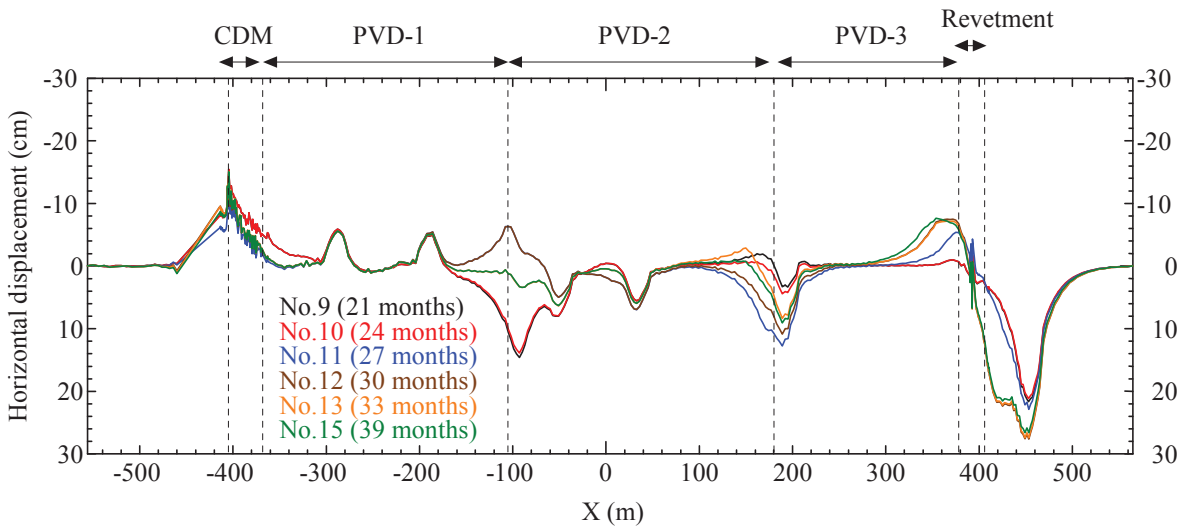
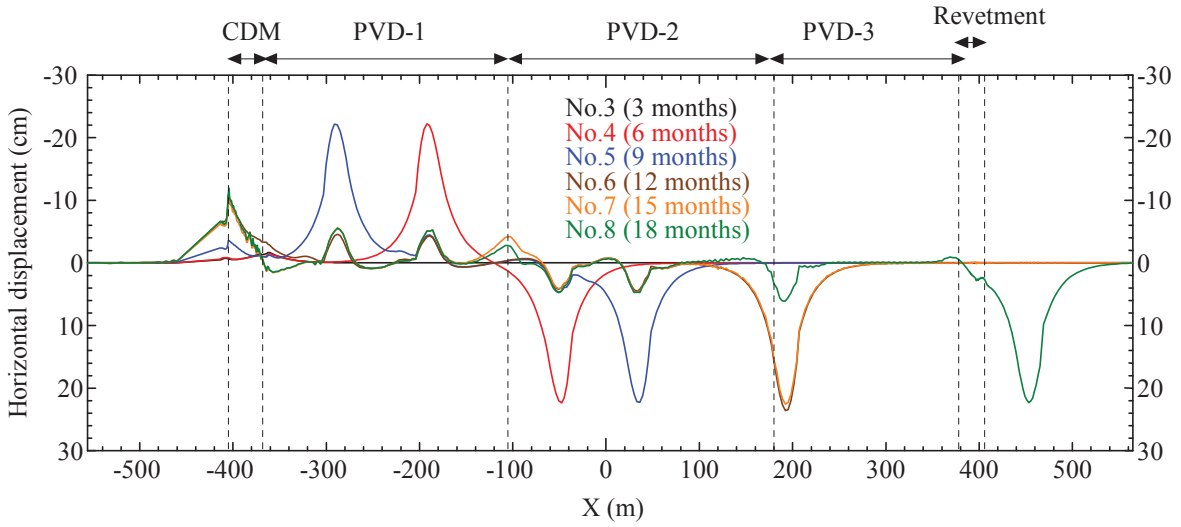
[2-2' section]



- In 1-1' section, the settlement of paved surface is larger in the left area (PVD-1 area) because surcharge load in service was larger.
- In the area improved by CDM, the settlement of paved surface is smaller.
- If the surcharge load is added uniformly, the point where larger settlement has already occurred by preloading tends to cause smaller settlement of the paved surface.
- Near the rubble-mound revetment, the settlement of paved surface is affected by the continuous settlement of the non-PVD ground under the counter fill

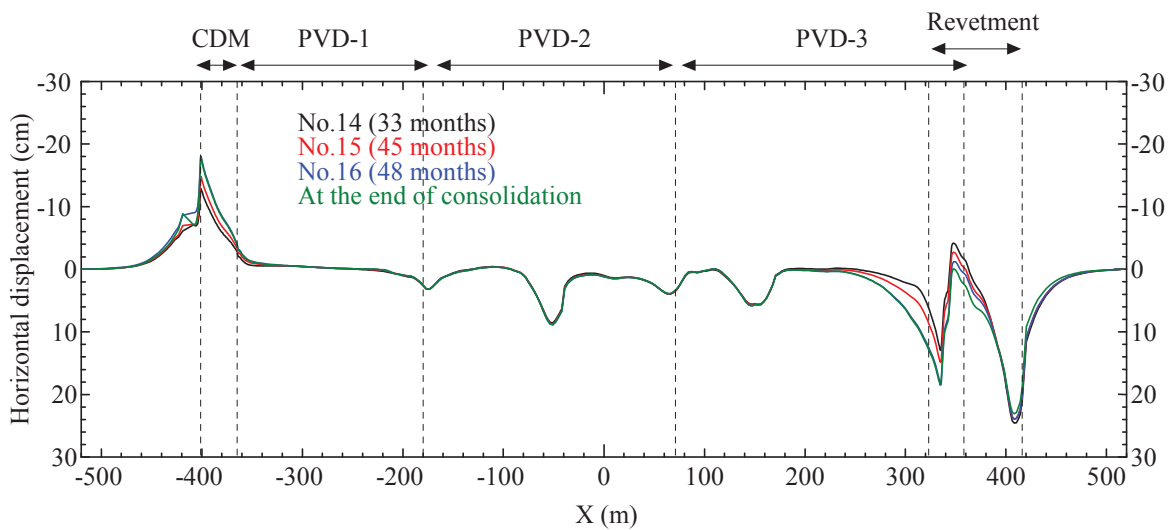
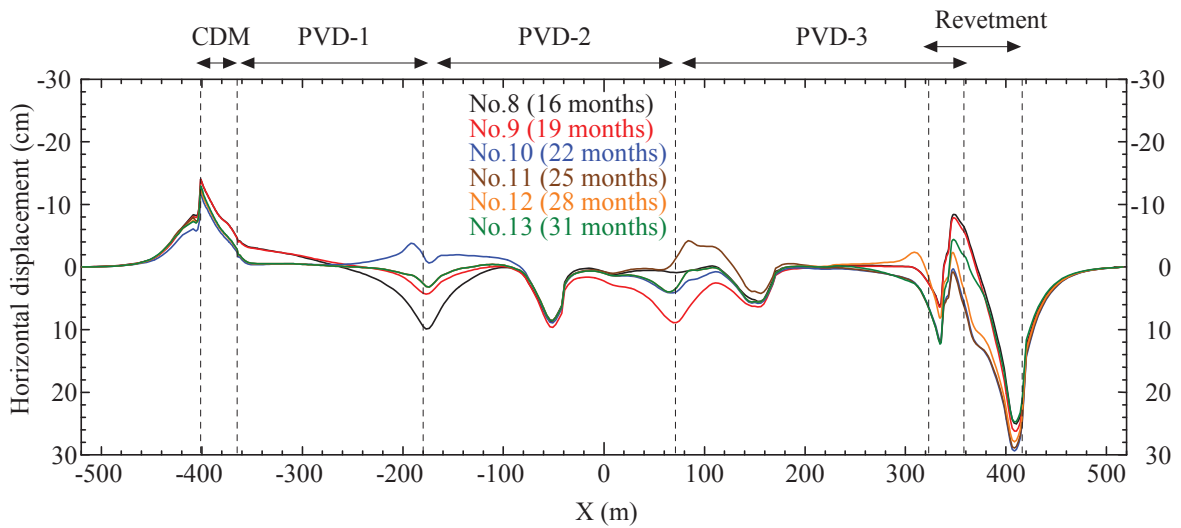
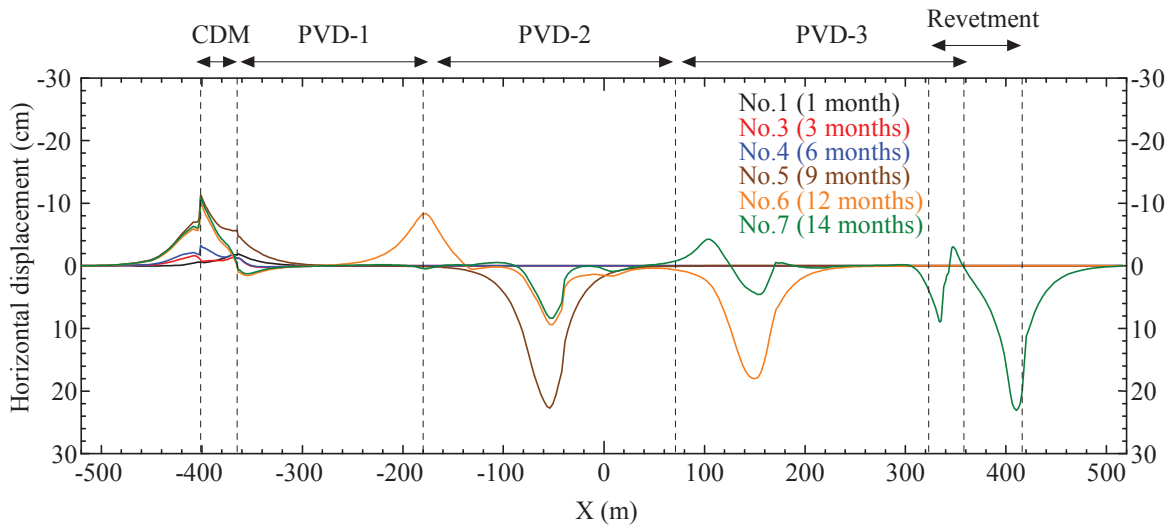
(7) Horizontal displacements of ground surface at the end of each stage

[1-1' section]





[2-2' section]

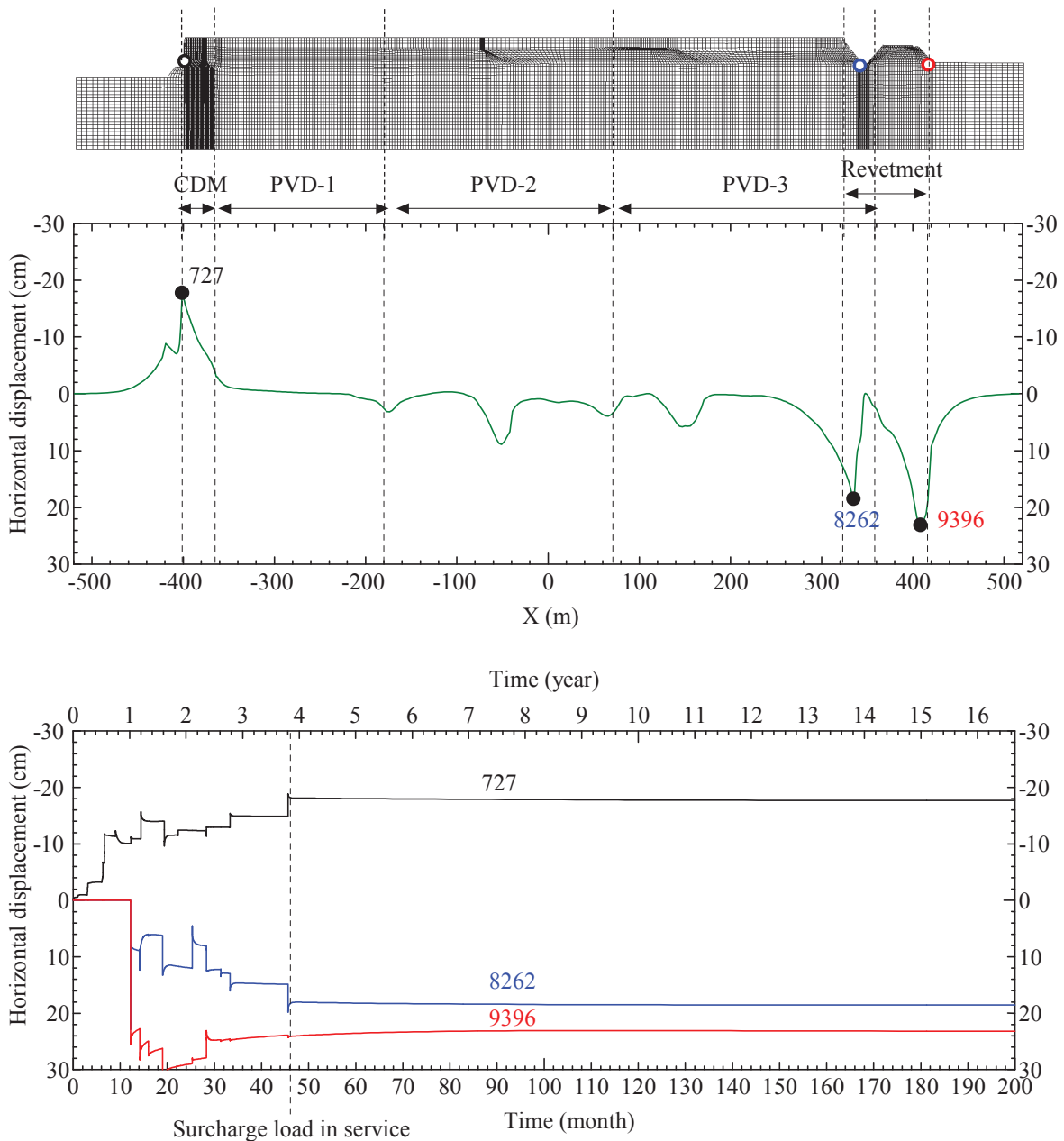


- Both of the sheet-pile revetment and the counter fill force out to sea-side.

- In 2-2' section, the deformation of the rubble-mound revetment is reduced by the counter fill.

(8) Time-Horizontal displacement behaviors at the typical points of ground surface

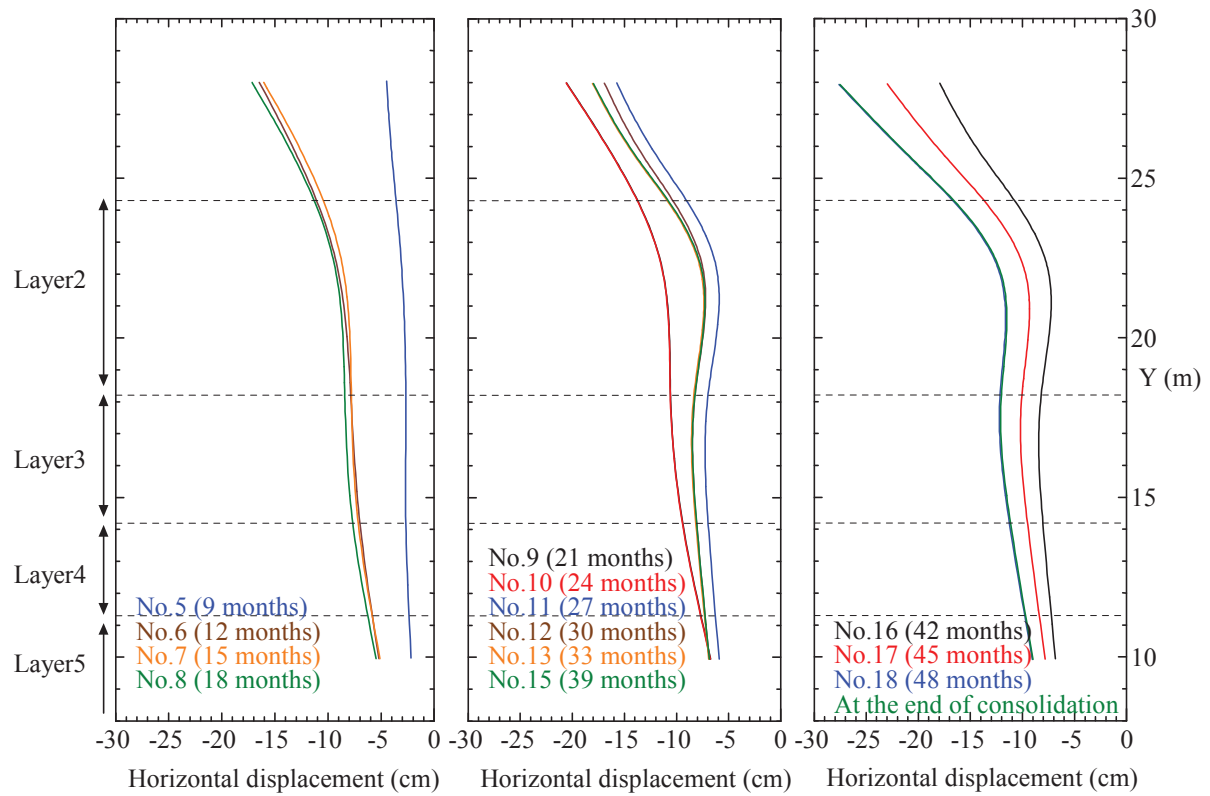
(as an example, shown the case of 2-2' section)



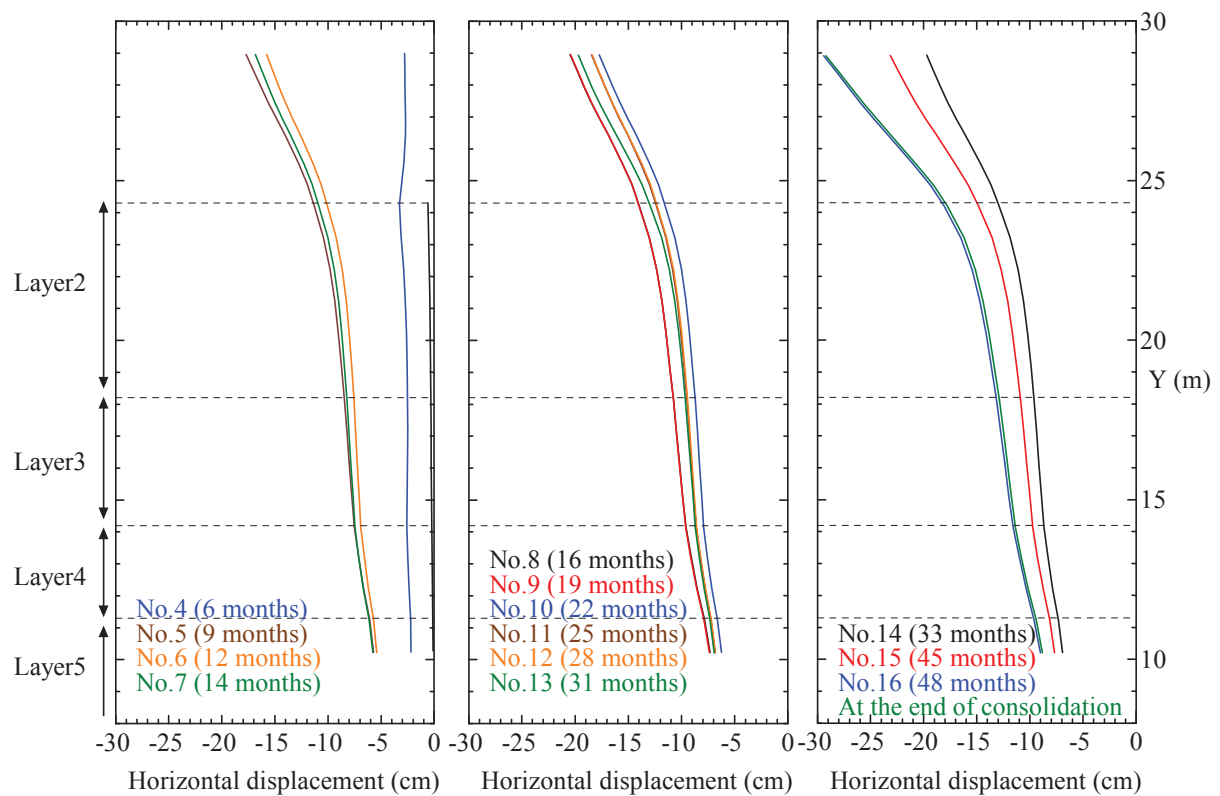
- The outward deformation by reclamation and the inward deformation by unloading and consolidation are repeated. The horizontal displacement converges sooner than the settlement.
- The sheet-pile revetment (node-727) and the rubble-mound revetment (node-8262) are forced out to sea-side by the pavement loading and the service loading. However, these deformations change inside immediately, and stabilized sooner.
- The small deformation to out-side of the counter fill continues after the entry of service, because the ground is not improved by PVD.

(9) Horizontal displacements of sheet pile at the end of each stage

[1-1' section]



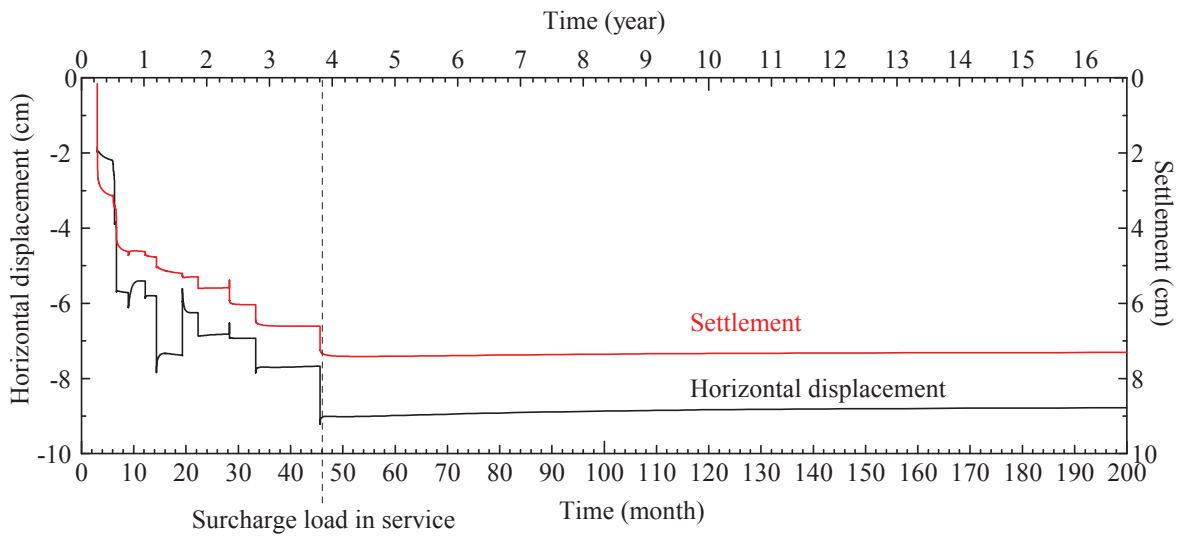
[2-2' section]



- The maximum horizontal displacements to out-side (sea-side) at the bottom and top of sheet-pile are about 10cm and about 30cm respectively.
- The displacement is larger in Layer2.

(10) Time-deformation behaviors at the bottom of sheet pile

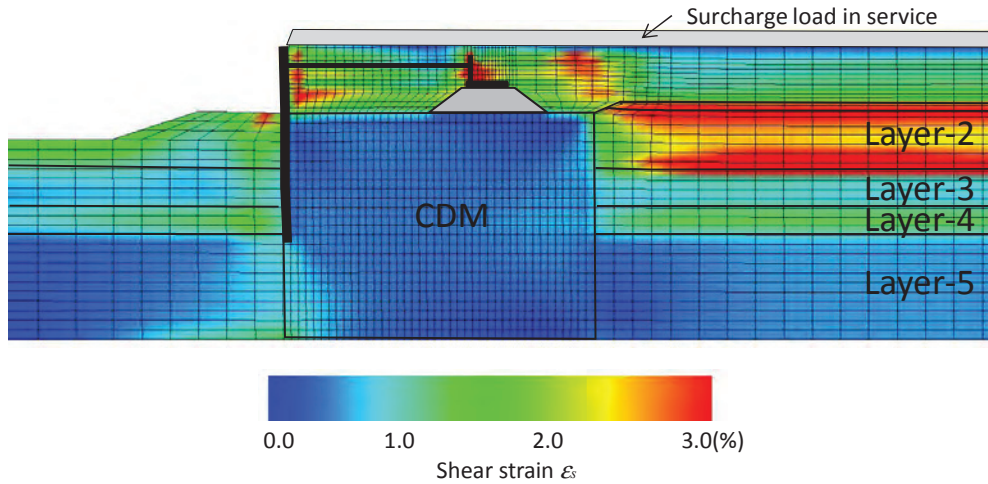
(as an example, shown the case of 2-2' section)



- In this analysis, the bottom of sheet pile is assumed to be located in the shallow part of Layer-5. As a result, the settlement is caused about 7cm, and the horizontal displacement to sea-side is about 9cm.
- After the entry of service, the small inward-deformation continues because the outside ground of sheet pile is not improved by PVD.

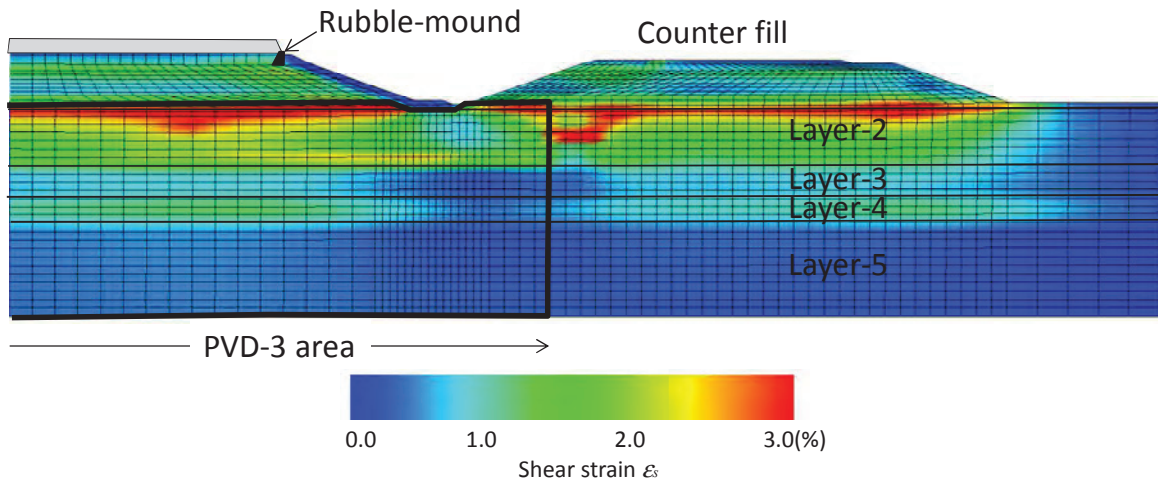
(11) Distribution of shear strain at the end of consolidation

[Sheet pile revetment and CDM-area (as an example, shown the case of 2-2' section)]



- Improvement by CDM is effective to reduce the ground deformation.
- Layer2 possesses the highest compressibility.
- The ground around the bottom of sheet pile is disturbed by the settlement of the pile.
- The reclaimed soils behind the sheet pile and the anchor wall are deformed in association with the outward deformation of sheet pile.

[Rubble-mound revetment (as an example, shown the case of 2-2' section)]



- The larger shear strain is observed in Layer2 directly below the left top of the counter fill and near the boundary line between improved and non-improved area by PVD, although it is not the levels that cause slip failure. However it is better that ground under the counter fill is also improved by PVD for safety constructions.

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- 5) Inagaki, M., Nakano, M., Noda, T., Tashiro, M., and Asaoka, A. (2010): Proposal of simple method of judging naturally deposited clay grounds exhibiting large long-term settlement due to embankment loading, *Soils and Foundations*, **50** (1), pp. 109-122.
- 6) Schmertmann, J.H. (1953): Estimating the true consolidation behavior of clay from laboratory test results, *Proceedings ASCE*, **79** (312), pp. 1-26.

## Appendix

### *The quantified expression of structure, overconsolidation, anisotropy, and their respective evolution rules*

Naturally deposited soils, whether clayey or sandy, generally exist in a ‘structured’ and overconsolidated state. To describe the deformation behavior of a soil in this state, we have to start from the base of an elasto-plastic model of a de-structured soil in a state of normal consolidation. Given that a soil in this unstructured and normally consolidated state still possesses anisotropy, we take for our ‘base’ in this paper the corrected Cam-clay model of Roscoe and Burland (1968) with the introduced addition of the rotational hardening concept of Sekiguchi and Ohta (1977), which treats stress parameter  $\eta^*$  and its evolution rule as an expression of anisotropy. The degrees of structure and overconsolidation are then introduced and quantified by means of the two concepts of the superloading surface for structure (Asaoka et al., 1998a, 2000, 2002), and the subloading surface for overconsolidation (Hashiguchi, 1978, 1989; Asaoka et al., 1997). That is to say, the degree of structure is expressed by means of a superloading surface situated on the outside of the Cam-clay normal-yield surface and similar to it (the center of similarity being the origin  $p' = q = 0$  and the similarity rate being given by  $R^*$  ( $0 < R^* \leq 1$ )), while the overconsolidation state is expressed by means of a subloading surface situated on the inside of the superloading surface and again similar to it (center of similarity  $p' = q = 0$ , similarity rate  $R$  ( $0 < R \leq 1$ ); reciprocal  $1/R$  is the overconsolidation ratio).  $p'$  here is the mean effective stress and  $q$  is the shear stress. Using effective stress tensor  $r'$  (tension: positive), we can say:  $p' = -\text{tr}T'/3$ ,  $q = \sqrt{3/2 S \cdot S}$ .

The closer  $R^*$  is to 0 the higher the degree of structure, but with the loss of structure that accompanies progressive plastic deformation  $R^*$  will approach 1 (evolution rule for  $R^*$ ). Similarly, the closer  $R$  is to 0 the more overconsolidated the state of the soil, but as  $R$  increases toward 1 with plastic deformation, the state of the soil will also approach normal consolidation (evolution rule for  $R$ ). It can thus be assumed that the loss of structure with progressive plastic deformation brings a simultaneous release from overconsolidation (a transition to the normally consolidated state), resulting finally in conditions that match those in the Cam-clay model. The relative positions of the three loading surfaces, assuming conditions of axial symmetry, are as shown in Fig. A1-1.

If we start from Cam-clay equation (A1-1) below as our base, given that the current effective stress exists on the subloading surface, we first need to adapt relations to the subloading surface through the application of various elasto-plastic principles such as the associated flow rule and Prager’s consistency condition, so as to give equation (A1-2).

The Cam-clay potential:

$$MD \ln \frac{\tilde{p}'}{\tilde{p}'_0} + MD \ln \frac{M^2 + \eta^{*2}}{M^2} + \int_0^t J \text{tr} \mathbf{D}^p d\tau = f(\tilde{p}', \eta^*) + \int_0^t J \text{tr} \mathbf{D}^p d\tau = 0 \quad (\text{A1-1})$$

The subloading surface:

$$f(p', \eta^*) + MD \ln R^* - MD \ln R + \int_0^t J \text{tr} \mathbf{D}^p d\tau = 0 \quad (\text{A1-2})$$

Here,  $D = (\tilde{\lambda} - \tilde{\kappa})/M/(1 + e_0)$  is the dilatancy coefficient, and  $M$ ,  $\tilde{\lambda}$ ,  $\tilde{\kappa}$  and  $e_0$  are the critical state constant,

compression index, swelling index, and initial void ratio.  $J = (1 + e)/(1 + e_0)$  ( $e$  is the void ratio at time  $t = t$ ).

$-\int_0^t J \text{tr} \mathbf{D}^p d\tau$  (compression: positive) corresponds to the plastic volumetric strain  $\eta^*$ , the expression of anisotropy, is obtained using the rotational hardening variable  $\boldsymbol{\beta}$ , from the calculation  $\eta^* = \sqrt{3/2} \hat{\boldsymbol{\eta}} \cdot \hat{\boldsymbol{\eta}}$ ,  $\hat{\boldsymbol{\eta}} = \boldsymbol{\eta} - \boldsymbol{\beta}$ ,  $\boldsymbol{\eta} = \mathbf{S} / p'$ ,  $\mathbf{S} = \mathbf{T}' + p' \mathbf{I}$ .  $\boldsymbol{\beta} = \mathbf{0}$  expresses a state of no anisotropy. In the present paper, the evolution rules for  $R^*$ ,  $R$  and  $\boldsymbol{\beta}$  are given by the following equations.

$$\text{Evolution rule for } R^* : \dot{R}^* = JU^* \left\{ c_s \sqrt{2/3} \|\mathbf{D}_s^p\| + (1 - c_s) (-D_v^p) \right\}, \quad U^* = \frac{a}{D} R^{*b} (1 - R^*)^c \quad (\text{A1-3})$$

$$\text{Evolution rule for } R : \dot{R} = JU \|\mathbf{D}^p\|, \quad U = -\frac{m}{D} \ln R \quad (\text{A1-4})$$

$$\text{Evolution rule for } \boldsymbol{\beta} : \dot{\boldsymbol{\beta}} = J \frac{br}{D} \sqrt{\frac{2}{3}} \|\mathbf{D}_s^p\| \|\hat{\boldsymbol{\eta}}\| \left( m_b \frac{\hat{\boldsymbol{\eta}}}{\|\hat{\boldsymbol{\eta}}\|} - \boldsymbol{\beta} \right) \quad (\text{A1-5})$$

$\mathbf{D}^p$  here is the plastic stretching tensor,  $\mathbf{D}_s^p$  is the deviator component of  $\mathbf{D}^p$ ,  $-D_v^p$  is the volumetric component of  $\mathbf{D}^p$ , and  $\|\cdot\|$  represents its norms.  $\dot{\boldsymbol{\beta}}$  in equation (A1-5) is the Green and Nahdhi's (1965) rate of  $\boldsymbol{\beta}$ . The parameter groups for the evolution rules in equations (A1-3) – (A1-5) all consists of constants, and from their respective functions we may call  $a$ ,  $b$ ,  $c$ ,  $c_s$  the degradation indices of structure,  $m$  the degradation index of overconsolidation,  $br$  the rotational hardening index, and  $m_b$  the rotational hardening limit constant.

A new evolution rule for  $R^*$  is proposed to replace Eq. (A1-3) for expressing structural upgradation, dividing the contribution of plastic deformation to the change of soil structure into the deviator of the plastic stretching  $\sqrt{2/3} \mathbf{D}_s^p$  and volumetric component  $-D_v^p$ . The ratio between these terms is given by  $c_s$  ( $0 \leq c_s \leq 1$ ).

*The associated flow rule and the constitutive equation*

$$\text{Associated flow rule : } \mathbf{D}^p = \lambda \frac{\partial f}{\partial \mathbf{T}'}, \quad \lambda = \frac{\frac{\partial f}{\partial \mathbf{T}'} \cdot \dot{\mathbf{T}}'}{J \frac{MD}{p'(M^2 + \eta^{*2})} (M_s^2 - \eta^2)} > 0 \quad (\text{A1-6})$$

$$\text{Constitutive equation : } \dot{\mathbf{T}}' = \mathbf{E} \mathbf{D} - \Lambda \mathbf{E} \frac{\partial f}{\partial \mathbf{T}'} \quad (\text{A1-7})$$

$\mathbf{E}$  is the elastic modulus tensor,  $\dot{\mathbf{T}}'$  is the Green and Naghdi's (1965) rate of  $\mathbf{T}'$ , and  $\Lambda$  is the expression of plastic multiplier  $\lambda$  in terms to stretching  $\mathbf{D}$ . Further, we can establish the relations

$$M_s^2 = M_a^2 + br \frac{4M\eta^{*2}}{M^2 + \eta^{*2}} (m_b \eta^* - \sqrt{\frac{3}{2}} \hat{\boldsymbol{\eta}} \cdot \boldsymbol{\beta}) - MD \left( \frac{U^*}{R^*} 2\eta^* + \frac{U}{R} \sqrt{6\eta^{*2} + \frac{1}{3} (M_a^2 - \eta^2)^2} \right) \quad (\text{A1-8})$$

and

$$M_a^2 = M^2 + \zeta^2, \quad \zeta = \sqrt{3/2} \|\boldsymbol{\beta}\| \quad (\text{A1-9})$$

The slope  $M_s$  of the threshold between hardening and softening  $q = M_s p'$ , obtained under loading



conditions  $\lambda > 0$ , varies according to structural degradation, loss of overconsolidation and development or loss of anisotropy, as well as with the current stress ratio. Similarly, the slope  $M_a$  of the threshold between plastic compression and expansion  $q = M_a p'$  varies in response to the development or loss of anisotropy. For details, the reader is referred to Asaoka et al., 2002.

Phụ lục 17

Phân tích tính toán Đê chắn sóng A & B

## **APPENDIX 17: CALCULATION ANALYSIS OF OUTER REVETMENT A & B**

### ***Table of Contents***

- 17.1 Armor Weight Calculation
- 17.2 Thickness of Underlayer
- 17.3 Width of Stone berm
- 17.4 Stability of Toe berm of Revetment (ORA & ORB)
- 17.5 Armour Stone Layer of channel side of Outer revetment B
- 17.6 Stability Calculation of Concrete cap of Outer Revetment A
- 17.7 Stability Calculation of Concrete cap of Outer Revetment B (ORB-A & B)
- 17.8 Stability Calculation of Concrete cap of Outer Revetment B (ORB-C & D)
- 17.9 Slope Stability Analysis of ORB-A (After 5 years)
- 17.10 Slope Stability Analysis of ORB-A (Until 5 years)
- 17.11 Slope Stability Analysis of ORB-B (After 5 years)
- 17.12 Slope Stability Analysis of ORB-B (Until 5 years)
- 17.13 Slope Stability Analysis of ORB-C (After 5 years)
- 17.14 Slope Stability Analysis of ORB-C (Until 5 years)
- 17.15 Slope Stability Analysis of ORB-D (After 5 years)
- 17.16 Slope Stability Analysis of ORB-D (Until 5 years)

### 17.1. ARMOR WEIGHT CALCULATION

(Shore Protection Manual - US Army Corps of Engineers-1984)

**Hudson Formula:**

$$W = \frac{\rho_s H_s^3}{K_D \left( \frac{\rho_s}{\rho_w} - 1 \right)^3 \cot \alpha}$$

Where in:

$\rho_s$  - mass density of armor units = 2 t/m<sup>3</sup>

$\rho_w$  - mass density of water (t/m<sup>3</sup>) = 1.025t/m<sup>3</sup>

$K_D$  - stability coefficient Tetrapod = **6.93** breaking zone

$\alpha$  - slope angle

$H_s$  - characteristic wave height

Concrete layer placing Qty. 2

Ord.	Wave height H1/3 (m)	Tetrapod (tons)								Remark
		Slope	W (tons)	Type (tons)	Thicknes s (m)	Bs (m)	Row of block placing on top	B (m)	B' (m)	
1	3.0	1 : 3	1.55	<b>2.00</b>	1.90	1.30	2	1.60	2.20	ORA
2	2.7	1 : 4/3	2.55	<b>3.20</b>	2.20	1.50	2	1.80	2.50	ORB_A&B
3	3.2	1 : 4/3	4.24	<b>5.00</b>	2.60	1.80	2	2.10	2.90	ORB_C&D

### 17.2. THICKNESS OF UNDERLAYER

$$r = nk_{\Delta} \left( \frac{W}{w_a} \right)^{1/3}$$

Where: r - average thickness of armour layer or underlayer (m)

n - number of quarystone or concrete armor units in tl = 2.00

W - weight of individual armor units

$w_a$  - specific weight of the armor unit material = 2.65 t/m<sup>3</sup>

$k_{\Delta}$  - layer coefficient, table VI-5-51 EM 1110-2-1100 = 1.00

**Equivalent diameter of Stone**

$$D_s = 1.24 \left( \frac{W_s}{\gamma_s} \right)^{1/3}$$

Where:  $W_s$  - stone weight (t)

$\gamma_s$  - stone specific weight in compatible units (t/m<sup>3</sup>)

W (kg)	200	300	400	500	600	700	800	1000
r (m)	0.85	0.97	1.06	1.15	1.22	1.28	1.34	1.45
$D_s$ (m)	0.52	0.60	0.66	0.71	0.76	0.80	0.83	0.90

### 17.1.(1) ARMOR WEIGHT CALCULATION (for reference)

(Shore Protection Manual - US Army Corps of Engineers-1984)

#### Hudson Formula:

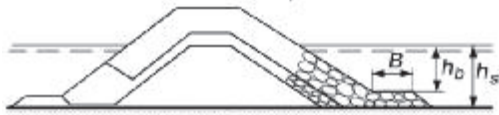
$$W = \frac{\rho_s H_s^3}{K_D \left( \frac{\rho_s}{\rho_w} - 1 \right)^3 \cot \alpha}$$

Where in:

$\rho_s$ - mass density of armor units	=	2.3 t/m <sup>3</sup>
$\rho_w$ - mass density of water	=	1.025 t/m <sup>3</sup>
$K_D$ - stability coefficient	RAKUNA-IV=	10.8 (for sloping breakwater)
$\alpha$ - slope angle		
$H_s$ - characteristic wave height		
Concrete layer	=	2 layers

Ord.	Wave height $H_{1/3}$ (m)	RAKUNA-IV (tons)								Remark
		Slope	W (tons)	Type (tons)	Thickness (m)	Bs (m)	Row of block placing on top	B (m)	B' (m)	
1	3	1 : 3	1.01	2	1.8	1.2	2	1.66	1.95	ORA
2	2.7	1 : 4/3	1.65	2	1.8	1.2	2	1.66	2.26	ORB_A&B
3	3.2	1 : 4/3	2.75	4	2.2	1.5	2	2.03	2.76	ORB_C&D

17.3. WIDTH OF STONE BERM (Table VI-5-45, Part VI, pVI-5-105)



Two layer armor stone toe berm for exposed sides of rubble-mound breakwaters and jetties (CERC 1986)

3  $B = 3t$  for ( $W_{50}$ ) berm  
 where  $t = (W_{50} / \gamma)^{1/3}$

$W_{50}$ (t)	2	3	4	5
$\gamma$ (t/m <sup>3</sup> )	2.65	2.65	2.65	2.65
t (m)	0.91	1.04	1.15	1.24
B (m)	2.73	<b>3.13</b>	3.44	3.71

### 17.4. STABILITY OF TOE BERM OF OUTER REVETMENT

(According to "The Rock Manual, CIRIA 683")

Van der Meer (1995) formular:

$$\frac{H_s}{\Delta D_{n50}} = \left( 2 + 6.2 \left( \frac{h_t}{h} \right)^{2.7} \right) N_{od}^{0.15} \quad (5.188) \quad M_{n50} = \rho_r D_{n50}^3$$

Hudson formular:

$$M_{50} = \frac{\rho_s}{K_D \Delta^3 \cot \alpha} H_s^3 \quad (5.133)$$

Where:

H<sub>s</sub> - significant wave height, H<sub>1/3</sub> of the incident waves at the toe of the structure (m)

Δ - relative buoyant density, ρ<sub>s</sub>/ρ<sub>w</sub>-1

D<sub>n50</sub> - equivalent cube length of median stone (m)

M<sub>n50</sub> - medium mass of rocks

ρ <sub>s</sub> (t/m <sup>3</sup> )	ρ <sub>w</sub> (t/m <sup>3</sup> )	Δ (t/m <sup>3</sup> )	N <sub>od</sub>	cotα	K <sub>D</sub>
2.65	1.025	1.59	0.5	2	3.5

Pos	Seabed Level (m)	Berm Level (m)	Water level (m)	H <sub>s</sub> (m)	h (m)	h <sub>t</sub> (m)	h <sub>t</sub> /h	H <sub>s</sub> /ΔD <sub>n50</sub> (m)	D <sub>n50</sub> (m)	M <sub>n50</sub> (kg)		
										Van der Meer	Hudson	Selected
ORA-1	-0.5	0.4	HHWL +4.43	3.00	4.93	4.03	0.82	<b>5.04</b>	0.38	140	-	<b>200</b>
ORA-2	-1.0	-0.1	HHWL +4.43	3.00	5.43	4.53	0.83	<b>5.23</b>	0.36	126	-	<b>200</b>
ORA-3	0.0	0.9	HHWL +4.43	3.00	4.43	3.53	0.80	<b>4.83</b>	0.39	159	-	<b>200</b>
ORA-4	1.0	1.9	HHWL +4.43	3.00	3.43	2.53	0.74	<b>4.26</b>	0.44	232	-	<b>200</b>
ORB-A	1.0	2.0	HHWL +4.43	2.70	3.43	2.43	0.71	<b>4.01</b>	0.43	204	-	<b>300</b>
ORB-B	1.5	2.5	HHWL +4.43	2.70	2.93	1.93	0.66	<b>3.61</b>	0.47	278	-	<b>300</b>
ORB-C	0.0	1.0	HHWL +4.43	3.20	4.43	3.43	0.77	<b>4.60</b>	0.44	223	-	<b>300</b>
ORB-D	0.0	1.0	HHWL +4.43	3.20	4.43	3.43	0.77	<b>4.60</b>	0.44	223	-	<b>300</b>
ORA-1	-0.50	0.40	LWL +0.43	0.87	0.93	0.03	0.03	<b>1.80</b>	0.30	75	63	<b>200</b>
ORA-2	-1.00	-0.10	LWL +0.43	0.87	1.43	0.53	0.37	<b>2.19</b>	0.25	42	63	<b>200</b>
ORA-3	0.00	0.90	LWL +0.43	0.87	0.43	-0.47	-1.09	<b>Out of range =&gt; Using Hudson's formular to check stability of armour stone</b>			63	<b>200</b>
ORA-4	1.00	1.90	LWL +0.43	0.87	-0.57	-1.47	2.58				63	<b>200</b>
ORB-A	1.00	2.00	LWL +0.43	0.80	-0.57	-1.57	2.75				49	<b>300</b>
ORB-B	1.50	2.50	LWL +0.43	0.80	-1.07	-2.07	1.93				49	<b>300</b>
ORB-C	0.00	1.00	LWL +0.43	0.80	0.43	-0.57	-1.33				49	<b>300</b>
ORB-D	0.00	1.00	LWL +0.43	0.90	0.43	-0.57	-1.33				69	<b>300</b>

## 17.5. ARMOR STONE CALCULATION

(According to "The Rock Manual, 2nd edition" applied for Shallow water condition)

### Armour Stone Layer of channel side of Outer revetment B

Stability parameter:

$$N_s = \frac{H_s}{\Delta D_n} = \frac{(K_D \cot \alpha)^{\frac{1}{3}}}{1.27} \quad (\text{Eq. 5.134})$$

$$W_n = D_n^3 \rho_s$$

Where:  $N_s$  - stability parameter

$H_s$  - significant wave height,  $H_{1/3}$  of the incident waves at the toe of the structure (m)

$\Delta$  - relative buoyant density,  $\rho_r/\rho_w - 1$

$\alpha$  - slope angle ( $^\circ$ )

Surf similarity parameter, using the energy wave period  $T_{m-1,0}$ :

$$\xi_{s-1,0} = \frac{\tan \alpha}{\sqrt{\frac{2\pi H_s}{g T_{m-1,0}^2}}}$$

Critical surf similarity parameter:

$$\xi_{cr} = \left[ \frac{c_{pl}}{c_s} P^{0.31} \sqrt{\tan \alpha} \right]^{\frac{1}{P+0.5}} \quad (\text{Eq. 5.138})$$

For  $\xi_{s-1,0} < \xi_{cr}$  wave are plunging and Equation (5.139) applies:

$$\frac{H_s}{\Delta D_n} = c_{pl} P^{0.18} \left( \frac{S_d}{\sqrt{N}} \right)^{0.2} \left( \frac{H_s}{H_{2\%}} \right) (\xi_{s-1,0})^{-0.5} \quad (\text{Eq. 139})$$

For  $\xi_{s-1,0} > \xi_{cr}$  wave are surging and Equation (5.140) applies:

$$\frac{H_s}{\Delta D_n} = c_s P^{-0.13} \left( \frac{S_d}{\sqrt{N}} \right)^{0.2} \left( \frac{H_s}{H_{2\%}} \right) \sqrt{\cot \alpha} (\xi_{s-1,0})^P \quad (\text{Eq. 140})$$

Where:  $c_{pl} = 8.4$ , with a standard deviation of  $\sigma = 0.7$

$c_s = 1.3$ , with a standard deviation of  $\sigma = 0.15$

$P$  - notional permeability of structure,  $0.1 < P < 0.6$  (see Fig. 5.39)

$N$  - number of incident waves at the toe, which depends on the duration of the wave conditions

$H_{2\%}$  - wave height exceeded by 2 per cent of the incident waves at the toe (m),  $H_{2\%} = (1.2 \div 1.4) H_s$

$T_{m-1,0}$  - the spectral mean energy period (s)

$$T_{m-1,0} = \frac{T_p}{1.1} \quad (\text{Eq. 4.62}) \quad T_p = \frac{T_{1/3}}{(0.9 \div 0.96)}$$

$\rho_r$ (t/m <sup>3</sup> )	$\rho_w$ (t/m <sup>3</sup> )	$\Delta$ (t/m <sup>3</sup> )	$\alpha$ ( $^\circ$ )	$H_s$ (m)	$T_{1/3}$ (s)	$H_{2\%}$ (m)	$T_p$ (s)	$T_{m-1,0}$ (s)	$g$ (m/s <sup>2</sup> )
2.65	1.025	1.59	33.69	2.4	5.98	3.12	6.43	5.85	9.81

$P$	$S_d$	Damage level (%)	$N$ in 3 hours	$\xi_{s-1,0}$	$\xi_{cr}$	$N_s$	$D_n$	$W_s$ (t)	$W_s$ (t) Selected
0.5	10	10-15	1848	3.14	4.26	2.40	0.63	0.66	0.70



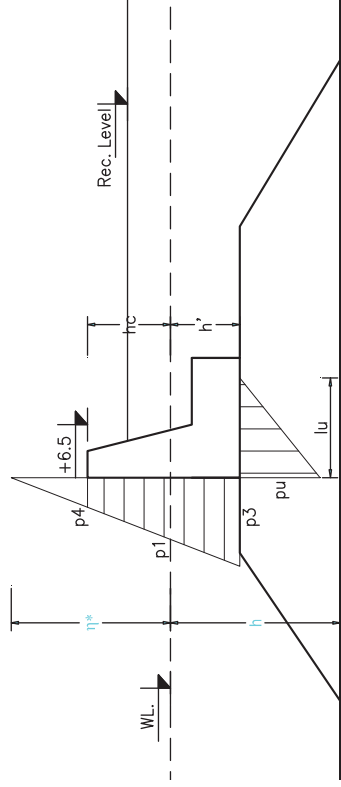
# 17.6. STABILITY CALCULATION OF CONCRETE CAP OF OUTER REVETMENT A

## A. Wave Force under Wave Crest

### 1. Input data:

	50 years	5 years
- Tide Level - HHWL (CDL):	1% (100 yr)	20% (20 yr)
	<b>4.43</b>	<b>3.92</b>
- Crest Level (CDL):		<b>+ 6.5</b>
- Bottom Upright Wall Level (CDL):		+ 3.7
- Bottom Width of Upright Wall (m):		2.5
- Reclamation Level (CDL):		+ 5.5
- Seabed slope:		
- Seabed Surface Level (CDL):		0.5
- Significant Wave Height (H <sub>1/3</sub> ), m:		3
- H <sub>max</sub> (Godal's Index), m:		3.9
- Wave Period (T), sec:		13.3
- Wave Length (L), m:		81.05
		70

### 2. Wave Pressure on the Front Face of an Upright Wall



Wave Pressure Distribution Used in Design Calculation

$$p_1 = 0.5(1 + \cos\beta)\lambda\alpha_1\rho_0gH_D$$

$$p_3 = pu = \alpha_3p_1$$

$$\lambda = \exp\left[-10\left(\frac{h}{L}\right)^{1.5}\left(1 - \frac{h'}{h}\right)^5\right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta} \quad (\text{if } h' \leq 0)$$

$$\eta = 0.75(1 + \cos\beta)\lambda_1 H_D$$

### 3. Uplift beneath Upright Wall

$$p_3 = pu = \alpha_3p_1$$

$$l_u = \min\left\{B, 0.2 \frac{(\eta' + h')}{|h'|}\right\}$$

### 4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>b</sub> = H<sub>max</sub>  
 5.1. Input data

	50 years	5 years
β (deg) =		<b>32</b>
cos β:		0.848
H <sub>b</sub> (m):		3.9
L (m):		81.0474397
HHWL (CDL):		4.43
Sea bed Level (CDL):		0.5
Crest Level (CDL):		6.5
h (m):		3.93
h' (m):		0.73
h <sub>b</sub> (t/m3):		1.025
g (m/s2):		9.81
h <sub>c</sub> (m):		2.07
4 πh/L:		0.609
2 πh/L:		0.305
λ:		0.963
		0.925

### 5.2. Wave pressure on the front face of an Upright wall

η* =	5.20	4.58
α <sub>1</sub> =	1.042	1.042
α <sub>3</sub> =	0.992	0.997
h <sub>c</sub> * = min{η*, h <sub>c</sub> }	2.07	2.58
α <sub>4</sub> =	0.602	0.437
(1 + cosβ)	1.848	
α <sub>1</sub> λ p <sub>0</sub> H <sub>b</sub>	4.011	3.528
p <sub>1</sub> (t/m2) =	3.71	3.26
p <sub>3</sub> (t/m2) =	3.68	3.25
p <sub>4</sub> (t/m2) =	2.23	1.42
p <sub>c</sub> (at crest level), (t/m2) =	1.94	1.35

### 5.3. Uplift beneath Upright Wall

p <sub>u</sub> (t/m2) =	3.68	3.25
l <sub>u</sub> (m) =	1.63	2.50

### 5.4. Total wave pressure on Upright Wall

P (t/m) =	8.27	6.54
U (t/m) =	2.99	4.06

## B. Effect of Reclamation Soil

	50 years	5 years
H (m)		1.8
φ (deg)		30.0
γ <sub>s</sub> (t/m3)		1.6
γ' <sub>s</sub> (t/m3)		1.03
h <sub>top</sub> (m)		1.1
h <sub>bot</sub> (m)		0.7
K <sub>p</sub> = tan <sup>2</sup> (45° + φ/2)		3.0
p <sub>p</sub> (dry), t/m2		5.14
p <sub>p</sub> (wet), t/m2		7.38
p <sub>p</sub> (dry), t/m		2.75
p <sub>p</sub> (wet), t/m		4.57
<b>Pp (final), t/m</b>		<b>7.32</b>

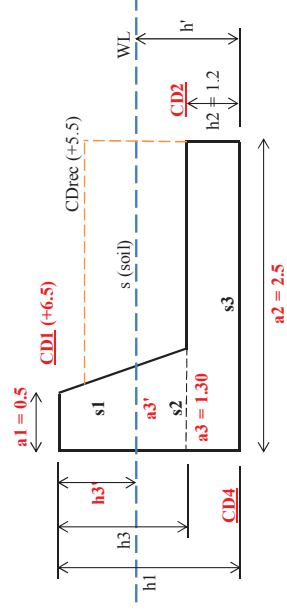
### C. Safety Factor

μ (friction factor between conc. & stone):	<b>0.60</b>
W (total), t/m	8.79
μ (friction factor between stone & stone):	<b>0.80</b>

Sliding	<b>S.F. (Normal)</b>	<b>0.42</b>	<b>0.55</b>
	<b>S.F. (Soil effect)</b>	<b>3.65</b>	<b>OK</b>

M <sub>pp</sub>	10.63	7.97
M <sub>fp</sub>	4.55	4.66
M <sub>w</sub> (=W*1)	13.64	15.28
<b>S.F. (Soil Effect)</b>	<b>1.28</b>	<b>2.58</b>

	50 years	5 years
<b>CD1</b>	<b>+ 6.5</b>	<b>+ 6.5</b>
<b>CD2</b>	<b>+ 4.9</b>	<b>+ 4.9</b>
<b>CD4</b>	<b>+ 3.7</b>	<b>+ 3.7</b>
<b>CD Rec.</b>	<b>+ 5.5</b>	<b>+ 5.5</b>
WL	<b>+ 4.43</b>	<b>+ 3.92</b>
<b>a1</b>	<b>0.5</b>	<b>0.5</b>
<b>a2</b>	<b>2.5</b>	<b>2.5</b>
<b>a3</b>	<b>1.30</b>	<b>1.3</b>
<b>a3'</b>	<b>1.300</b>	<b>1.300</b>
h1	2.8	2.8
h2	<b>1.2</b>	<b>1.2</b>
h3	1.60	1.6
<b>h3'</b>	<b>1.6</b>	<b>1.6</b>
h'	0.73	0.22
s1	1.44	1.44
s2	0.00	0.00
s3 upperWL	1.175	2.45
<b>s3 underWL</b>	<b>1.825</b>	<b>0.55</b>
<b>ΣS</b>	<b>4.44</b>	<b>4.44</b>
T	2.4	2.4
T'	1.375	1.375
W (s1)	3.46	3.46
W (s2)	0.00	0.00
W (s3)	5.33	6.64
<b>W (total)</b>	<b>8.79</b>	<b>10.09</b>



	50 years	5 years
Conc.		
X <sub>s1</sub>	2.02	2.02
M (s1)	6.98	6.98
X <sub>s2</sub>	0.00	0.00
M (s2)	0.00	0.00
X <sub>s3</sub>	1.25	1.25
M (s3)	6.66	8.30
M <sub>w</sub>	<b>13.64</b>	<b>15.28</b>
Soil pressure		
Pp upperWL	2.75	5.99
Pp underWL	4.57	1.74
Y <sub>upper</sub>	<b>1.087</b>	<b>0.747</b>
Y <sub>under</sub>	<b>0.343</b>	<b>0.108</b>
Mp upperWL	2.99	4.47
Mp underWL	1.57	0.19
Mp (total)	<b>4.55</b>	<b>4.66</b>
Wave		
P	8.27	6.54
U	2.99	4.06
Y <sub>p</sub>	<b>1.286</b>	<b>1.218</b>
X <sub>w</sub>	<b>1.958</b>	<b>1.667</b>
M <sub>p</sub>	<b>10.63</b>	<b>7.97</b>
M <sub>w</sub>	<b>5.85</b>	<b>6.77</b>
M total	16.48	14.74

17.6.(1) STABILITY CALCULATION OF CONCRETE CAP OF OUTER REVETMENT A (for reference)

A. Wave Force under Wave Crest

1. Input data:

	50 years	5 years
Tidal Level HHWL(CDL):	1%(100 yr) 4.43	20%(20 yr) 3.92
Crest Level(CDL):	6.5	
Bottom Upright Wall Level (CDL):	3.8	
Bottom Width of Upright wall (m):	2.5	
Reclamation Level (CDL):	5.5	
Crest Level of Armor stone (CDL):		
Seabed slope:		
Seabed sSurface Level (CDL):	0.5	
Significant Wave Height (H <sub>1/3</sub> ),m:	3	2.8
H <sub>max</sub> (Goda's Index),m:	3.9	3.57
Wave Period (T),sec:	13.3	11
Wave Length (L),m:	81.05	70

2. Results of Calculation

2.1. Input data

	50 years	5 years
β(deg)=	32	
cosβ:	0.848	
H <sub>D</sub> (m):	3.9	3.57
L(m):	81.05	70
HHWL(CDL):	4.43	3.92
Sea bed Level(CDL):	0.5	
Crest Level(m):	6.5	
h (m):	3.93	3.42
h' (m):	0.63	0.12
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.025	
g (m/s <sup>2</sup> ):	9.81	
h <sub>c</sub> (m):	2.07	2.58
4 πh/L:	0.609	0.614
2 πh/L:	0.305	0.307
λ:	0.956	0.914

2.2. Wave pressure on the front face of an Upright wall

η <sup>+</sup> =	5.17	4.52
α <sub>1</sub> =	1.042	1.042
α <sub>3</sub> =	0.993	0.998
h <sub>c</sub> <sup>+</sup> =min(η <sup>+</sup> ,h <sub>c</sub> )	2.07	2.58
α <sub>2</sub> =	0.600	0.429
(1+cosβ)	1.848	
α <sub>1</sub> λρ <sub>0</sub> H <sub>D</sub>	3.986	3.482
p <sub>1</sub> (t/m <sup>2</sup> )	3.68	3.22
p <sub>3</sub> (t/m <sup>2</sup> )	3.66	3.21
p <sub>4</sub> (t/m <sup>2</sup> )	2.21	1.38
p <sub>c</sub> (at crest level)(t/m <sup>2</sup> )=	1.92	1.31

2.3. Uplift beneath Upright Wall

P <sub>u</sub>	3.66	3.21
l <sub>u</sub>	1.84	2.50

2.4. Total wave pressure on Upright Wall

P	8.41	6.32
U	3.37	4.02

B. Effect of Reclamation Soil

	50 years	5 years
H(m)	1.7	
φ(deg)	30	
γ <sub>s</sub> (t/m <sup>3</sup> )	1.8	
γ' <sub>s</sub> (t/m <sup>3</sup> )	1	
h <sub>dry</sub> (m)	1.07	1.58
h <sub>wet</sub> (m)	0.63	0.12
K <sub>p</sub> =tan <sup>2</sup> (45°+φ/2)	3	
P <sub>s</sub> (dry),t/m <sup>2</sup>	5.78	8.53
P <sub>s</sub> (wet),t/m <sup>2</sup>	7.67	8.89
P <sub>p</sub> (dry),t/m	3.09	6.74
P <sub>p</sub> (wet),t/m	4.24	1.05
P <sub>p</sub> (final),t/m	7.33	7.79

C. Safety Factor

μ (coc.&stone)	0.6	
W	8.44	9.75
μ(stone&stone)	0.8	

Sliding	S.F.(normal)	0.36	0.54
	S.F.(soil effect)	1.23	1.78

Overturning	M <sub>p</sub>	10.42	7.40
	M <sub>Pp</sub>	4.38	4.42
	M <sub>w</sub>	13.21	14.85
	S.F.(Normal)	0.87	1.10
	S.F.(Soil effect)	1.29	1.70

Concrete Cap

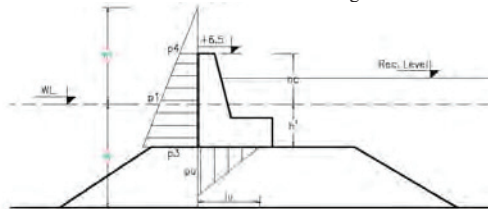
	50 years	5 years
CD1	6.5	6.5
CD2	4.9	4.9
CD4	3.8	3.8
CD Rec.	5.5	5.5
WL	4.43	3.92
a1	0.5	0.5
a2	2.5	2.5
a3	1.3	1.3
a3'	1.3	1.3
h1	2.7	2.7
h2	1.1	1.1
h3	1.6	1.6
h3'	1.6	1.6
h'	0.63	0.12
s1	1.44	1.44
s2	0E+00	0E+00
s3upperWL	1.175	2.45
s3underWL	1.575	0.3
ΣS	4.19	4.19
γ	2.4	2.4
γ'	1.375	1.375
W (s1)	3.46	3.46
W (s2)	0.00	0.00
W (s3)	4.99	6.29
W (total)	8.44	9.75

Concrete	x <sub>s1</sub>	2.02	2.02
	M (s1)	6.98	6.98
	x <sub>s2</sub>	0	0
	M (s2)	0	0
	x <sub>s3</sub>	1.25	1.25
M (s3)	6.23	7.87	
M <sub>w</sub>	13.21	14.85	

Soil pressure	P <sub>p</sub> upperWL	3.09	6.74
	P <sub>p</sub> underWL	4.24	1.05
	y <sub>p</sub> .upper	0.99	0.65
	y <sub>p</sub> .under	0.315	0.06
	M <sub>p</sub> upperWL	3.05	4.36
	M <sub>p</sub> underWL	1.33	0.06
	M <sub>p</sub> total	4.38	4.42

Wave	P	8.41	6.32
	U	3.37	4.02
	y <sub>p</sub>	1.24	1.17
	x <sub>u</sub>	1.23	1.67
	M <sub>p</sub>	10.42	7.40
	M <sub>u</sub>	4.13	6.69
	M total	14.55	14.09

Wave Pressure Distribution Used in Design Calculation



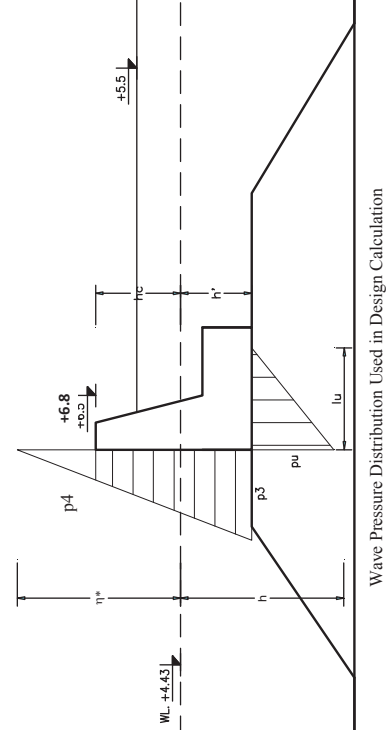
## 17.7. STABILITY CALCULATION OF CONCRETE CAP OF OUTER REVETMENT B (ORB-A & ORB-B)

### A. Wave Force under Wave Crest

#### 1. Input data:

	50 years	5 years
- Tide Level - HHWL (CDL):	1 % (100.YT) 4.43	20 % (20.YT) 3.92
- Crest Level (CDL):	6.8	
- Bottom Upright Wall Level (CDL):	4.6	
- Bottom Width of Upright Wall (m):	2.5	
- Reclamation Level (CDL):	5.5	
- Crest Level of Armor stone (CDL):		
- Seabed slope:		
- Seabed Surface Level (CDL):	1.0	
- Significant Wave Height (H <sub>1/3</sub> ), m:	2.7	2.3
- H <sub>max</sub> (Goda's Index), m:	3.5	3.1
- Wave Period (T), sec:	13.3	11
- Wave Length (L), m:	76.90	59.62

#### 2. Wave Pressure on the Front Face of an Upright Wall



Wave Pressure Distribution Used in Design Calculation

$$P_1 = 0.5(1 + \cos\beta) \lambda \alpha_1 \rho_0 g H_D$$

$$P_3 = pu = \alpha_3 P_1$$

$$\lambda = \exp\left[-10 \left(\frac{h}{L}\right)^{1.5} \left(1 - \frac{h'}{h}\right)^5\right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta} \quad (if \ h' \leq 0)$$

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

#### 3. Uplift beneath Upright Wall

$$P_3 = pu = \alpha_3 P_1$$

$$l_u = \min\left(B, 0.2 \frac{(\eta^* + h')}{|h'|}\right)$$

$$H_0 = H_{max}$$

#### 4. Wave Height and Wavelength Used in the Wave Pressure Calculation

$$H_0 = H_{max}$$

#### 5. Results of Calculation

##### 5.1. Input data

	50 years	5 years
$\beta$ (deg) =	41	
$\cos \beta$ :	0.755	
H <sub>D</sub> (m):	3.5	3.1
L (m):	76.90	59.62
HHWL (CDL):	4.43	3.92
Sea bed Level (CDL):	1.0	
Crest Level (CDL):	6.8	
h (m):	3.43	2.92
h' (m):	-0.17	-0.68
p <sub>0</sub> (t/m <sup>3</sup> ):	1.025	
g (m/s <sup>2</sup> ):	9.81	
h <sub>c</sub> (m):	2.37	2.88
4 πh/L:	0.561	0.615
2 πh/L:	0.280	0.308
λ:	0.887	0.734

##### 5.2. Wave pressure on the front face of an Upright wall

η* =	4.09	3.00
α <sub>1</sub> =	1.051	1.041
α <sub>3</sub> =	0.958	0.773
h <sub>c</sub> * = min{η*, h <sub>c</sub> }	2.37	2.88
α <sub>4</sub> =	0.420	0.039
(1 + cosβ)	1.755	
α <sub>1</sub> λ p <sub>0</sub> H <sub>D</sub>	3.343	2.430
P <sub>1</sub> (t/m <sup>2</sup> ) =	2.93	2.13
P <sub>3</sub> (t/m <sup>2</sup> ) =	2.81	1.65
P <sub>4</sub> (t/m <sup>2</sup> ) =	1.23	0.08
P <sub>u</sub> (at crest level), (t/m <sup>2</sup> ) =	1.13	0.05

##### 5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	2.81	1.65
l <sub>u</sub> (m) =	2.50	0.68

##### 5.4. Total wave pressure on Upright Wall

P (t/m) =	4.45	1.90
U (t/m) =	3.51	0.56

#### II. Effect of Reclamation Soil

	50 years	5 years
H (m)	0.9	
φ (deg)	30.0	
γ <sub>s</sub> (t/m <sup>3</sup> )	1.6	
γ <sub>s</sub> ' (t/m <sup>3</sup> )	1.0	
h <sub>dry</sub> (m)	0.9	0.9
h <sub>wet</sub> (m)	0.0	0.0
K <sub>p</sub> = tan <sup>2</sup> (45° - φ/2)	3.0	
P <sub>u</sub> (dry), t/m <sup>2</sup>	4.32	4.32
P <sub>u</sub> (wet), t/m <sup>2</sup>	0.00	0.00
P <sub>p</sub> (dry), t/m	1.94	1.94
P <sub>p</sub> (wet), t/m	0.00	0.00
<b>P<sub>p</sub>(final), t/m</b>	<b>1.94</b>	<b>1.94</b>

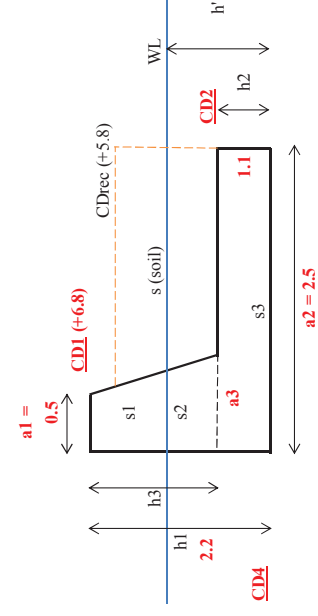
#### C. Safety Factor

μ (friction factor between conc. & stone):	0.60
W (total), t/m	8.65
μ (friction factor between stone & stone):	0.80

S.F. (Normal)	0.69	2.55
S.F. (Soil effect)	1.23	OK

Mu	5.86	0.94
Mp	4.25	1.46
S.F. (Normal)	1.57	7.93
S.F. (Soil Effect)	1.82	13.20

	5 years	50 years
CD1	6.8	
CD2	5.7	
CD4	4.6	
WL	3.92	4.43
a1	0.5	
a2	2.5	
a3*	1.05	1.05
h1	2.2	
h2	1.1	
h3	1.1	
h3*	1.1	1.1
h'	-0.68	-0.17
s1	0.85	0.85
s2	0.00	0.00
s3upperWL	2.75	2.75
s3underWL	0	0
γ	2.4	t/m <sup>3</sup>
γ'	1.37	
W (s1)	2.05	2.05
W (s2)	0.00	0.00
W (s3)	6.60	6.60
<b>W (total)</b>	<b>8.65</b>	<b>8.65</b>



	5 years	50 years
X <sub>s1</sub>	2.10	2.10
M (s1)	4.29	4.29
X <sub>s2</sub>	0.00	0.00
M (s2)	0.00	0.00
X <sub>s3</sub>	1.25	1.25
M (s3)	8.25	8.25
M <sub>w</sub>	12.54	12.54
Pp upperWL	1.94	1.94
Pp underWL	0.00	0.00
γ <sub>soil</sub> upper	0.3000	0.3000
γ <sub>soil</sub> under	0.0000	0.0000
Mjp up	0.58	0.58
Mjp un	0.00	0.00
Mjp (total)	0.58	0.58
P	1.90	4.45
U	0.56	3.51
γ <sub>so</sub>	0.768	0.957
X <sub>u</sub>	1.667	1.67
Mjp	1.46	4.25
Mu	0.94	5.86
M total	2.40	10.11

17.7.(1) STABILITY CALCULATION OF CONCRETE CAP OF OUTER REVETMENT B (ORB-A&ORB-B) (for reference)

A. Wave Force under Wave Crest

1. Input data:

	50 years	5 years
Tidal Level HHWL(CDL):	1%(100 yr) 4.43	20%(20 yr) 3.92
Crest Level(CDL):	6.8	
Bottom Upright Wall Level (CDL):	5.0	
Bottom Width of Upright wall (m):	2.5	
Reclamation Level (CDL):	5.5	
Crest Level of Armor stone (CDL):		
Seabed slope:		
Seabed sSurface Level (CDL):	0.5	
Significant Wave Height ( $H_{1/3}$ ),m:	2.7	2.3
$H_{max}$ (Goda's Index),m:	3.5	3.1
Wave Period (T),sec:	13.3	11
Wave Length (L),m:	76.9	59.62

2. Results of Calculation

2.1. Input data

	50 years	5 years
$\beta$ (deg)=	41	
$\cos\beta$ :	0.755	
$H_D$ (m):	3.5	3.1
L(m):	76.9	59.62
HHWL(CDL):	4.43	3.92
Sea bed Level(CDL):	0.5	
Crest Level(m):	6.8	
h (m):	3.93	3.42
h' (m):	-0.57	-1.08
$\rho_0$ (t/m <sup>3</sup> ):	1.025	
g (m/s <sup>2</sup> ):	9.81	
hc (m):	2.07	2.58
$4\pi h/L$ :	0.642	0.721
$2\pi h/L$ :	0.321	0.360
$\lambda$ :	0.797	0.582

2.2. Wave pressure on the front face of an Upright wall

$\eta^+$	3.67	2.37
$\alpha_1$	1.037	1.022
$\alpha_3$	1.007	1.019
$h_c^+ = \min(\eta^+, h_c)$	2.07	2.37
$\alpha_2$	0.436	0.000
$(1 + \cos\beta)$	1.755	
$\alpha_1 \lambda \rho_0 H_D$	2.962	1.888
$p_1$ (t/m <sup>2</sup> )	2.60	1.66
$p_3$ (t/m <sup>2</sup> )	2.62	1.69
$p_4$ (t/m <sup>2</sup> )	1.13	0.00
$p_c$ (at crest level) (t/m <sup>2</sup> )=	1.03	-0.03

2.3. Uplift beneath Upright Wall

Pu	2.62	1.69
lu	2.50	0.31

2.4. Total wave pressure on Upright Wall

P	3.86	1.97
U	3.27	0.26

B. Effect of Reclamation Soil

	50 years	5 years
H(m)	0.5	
$\phi$ (deg)	30	
$\gamma_s$ (t/m <sup>3</sup> )	1.8	
$\gamma'_s$ (t/m <sup>3</sup> )	1	
$h_{dry}$ (m)	0.5	0.5
$h_{wet}$ (m)	0	0
$K_p = \tan^2(45^\circ + \phi/2)$	3	
$P_s$ (dry), t/m <sup>2</sup>	2.70	2.70
$P_s$ (wet), t/m <sup>2</sup>	0.00	0.00
$P_p$ (dry), t/m	0.68	0.68
$P_p$ (wet), t/m	0.00	0.00
$P_p$ (final), t/m	0.68	0.68

C. Safety Factor

$\mu$ (coc.&stone)	0.6	
W	9.86	9.86
$\mu$ (stone&stone)	0.8	

Sliding	S.F.(normal)	1.02	2.93
	S.F.(soil effect)	1.20	3.27

Overturning	Mp	2.51	0.98
	MPp	0.11	0.11
	Mw	12.58	12.58
	S.F.(Normal)	2.83	12.16
	S.F.(Soil effect)	2.88	12.28

Concrete Cap

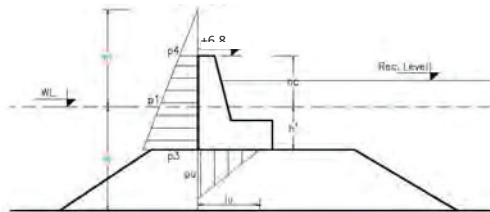
	50 years	5 years
CD1	6.8	6.8
CD2	5.9	5.9
CD4	5	5
CD Rec.	5.5	5.5
WL	4.43	3.92
a1	0.5	0.5
a2	2.5	2.5
a3	0.6	0.6
a3'	0.6	0.6
h1	1.8	1.8
h2	1.6	1.6
h3	0.2	0.2
h3'	0.2	0.2
h'	-0.57	-1.08
s1	0.11	0.11
s2	0	0
s3upperWL	4	4
s3underWL	0	0
ES	4.11	4.11
$\gamma$	2.4	2.4
$\gamma'$	1.375	1.375
W (s1)	0.26	0.26
W (s2)	0.00	0.00
W (s3)	9.60	9.60
W (total)	9.86	9.86

Concrete	$x_{s1}$	2.2	2.2
	M (s1)	0.58	0.58
	$x_{s2}$	0	0
	M (s2)	0	0
	$x_{s3}$	1.25	1.25
M (s3)	12.00	12.00	
Mw	12.58	12.58	

Soil pressure	Pp upperWL	0.68	0.68
	Pp underWL	0.00	0.00
	yp.upper	0.17	0.17
	yp.under	0	0
	Mp upperWL	0.11	0.11
	Mp underWL	0.00	0.00
	Mp total	0.11	0.11

Wave	P	3.86	1.97
	U	3.27	0.26
	yp	0.65	0.50
	xu	1.67	2.40
	Mp	2.51	0.98
	Mu	5.45	0.63
M total	7.97	1.61	

Wave Pressure Distribution Used in Design Calculation



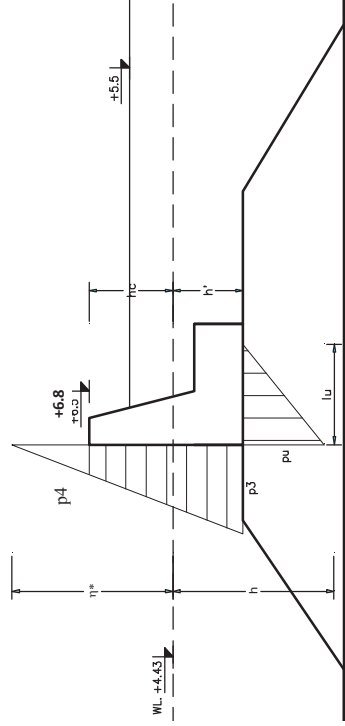
# 17.8. STABILITY CALCULATION OF CONCRETE CAP OF OUTER REVETMENT B (ORB-C & ORB-D)

## A. Wave Force under Wave Crest

### 1. Input data:

- Tide Level - HHWL (CDL):	50 years	5 years
- Crest Level (CDL):	1% (100 yr)	20% (20 yr)
- Bottom Upright Wall Level (CDL):	4.43	3.92
- Reclamation Level (CDL):	6.8	
- Crest Level of Armor stone (CDL):	4.3	
- Seabed slope:	2.5	
- Seabed Surface Level (CDL):	5.5	
- Significant Wave Height (H <sub>1/3</sub> ), m:	0	
- H <sub>max</sub> (Godá's Index), m:	3.2	2.6
- Wave Period (T), sec:	4.3	3.4
- Wave Length (L), m:	13.3	11
	84.97	62.33

### 2. Wave Pressure on the Front Face of an Upright Wall



Wave Pressure Distribution Used in Design Calculation

$$p_1 = 0.5(1+\cos\beta)\alpha_1\rho_0gH_0$$

$$p_3 = pu = \alpha_3 p_1$$

$$\lambda = \exp\left[-10\left(\frac{h}{L}\right)^{1.5}\left(1 - \frac{h'}{h}\right)^5\right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta}$$

$$\eta = 0.75(1 + \cos\beta)\lambda_1 H_0$$

### 3. Uplift beneath Upright Wall

$$p_3 = pu = \alpha_3 p_1$$

$$l_u = \min(B, 0.2\sqrt{h' + h'})$$

### 4. Wave Height and Wavelength Used in the Wave Pressure Calculation

$$H_0 = H_{max}$$

### 5. Results of Calculation

#### 5.1. Input data

β (deg):	50 years	5 years
cos β:	66	0.407
H <sub>0</sub> (m):	4.3	3.4
L (m):	84.97	62.33
HHWL (CDL):	4.43	3.92
Sea bed Level (CDL):	0	
Crest Level (CDL):	6.8	
h (m):	4.43	3.92
h' (m):	0.13	-0.38
p <sub>0</sub> (t/m <sup>3</sup> ):	1.025	
g (m/s <sup>2</sup> ):	9.81	
h <sub>c</sub> (m):	2.37	2.88
4 πh/L:	0.655	0.790
2 πh/L:	0.328	0.395
λ:	0.903	0.778

#### 5.2. Wave pressure on the front face of an Upright wall

η* =	4.09	2.79
α <sub>1</sub> =	1.034	1.008
α <sub>3</sub> =	0.998	0.864
h <sub>c</sub> * = min(η*, h <sub>c</sub> )	2.37	2.79
α <sub>4</sub> =	0.421	0.000
(1+cosβ)	1.407	
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>0</sub>	4.114	2.734
P <sub>1</sub> (t/m <sup>2</sup> ) =	2.89	1.92
P <sub>3</sub> (t/m <sup>2</sup> ) =	2.89	1.66
P <sub>4</sub> (t/m <sup>2</sup> ) =	1.22	0.00
P <sub>2</sub> (at crest level), (t/m <sup>2</sup> ) =	1.18	-0.05

#### 5.3. Uplift beneath Upright Wall

P <sub>3</sub> (t/m <sup>2</sup> ) =	2.89	1.66
l <sub>u</sub> (m) =	2.50	1.27

#### 5.4. Total wave pressure on Upright Wall

P (t/m) =	5.13	2.08
U (t/m) =	3.61	1.05

## B. Effect of Reclamation Soil

	50 years	5 years
H (m)	1.2	
φ (deg)	30.0	
γ <sub>s</sub> (t/m <sup>3</sup> )	1.6	
γ <sub>1</sub> * (t/m <sup>3</sup> )	1.0	
h <sub>dry</sub> (m)	1.1	1.2
h <sub>wet</sub> (m)	0.1	0.0
K <sub>p</sub> = tan <sup>2</sup> (45° - φ/2)	3.0	
P <sub>1</sub> (dry), t/m <sup>2</sup>	5.14	5.76
P <sub>1</sub> (wet), t/m <sup>2</sup>	5.53	0.00
P <sub>3</sub> (dry), t/m	2.75	3.46
P <sub>3</sub> (wet), t/m	0.69	0.00
<b>P<sub>3</sub>(final), t/m</b>	<b>3.44</b>	<b>3.46</b>

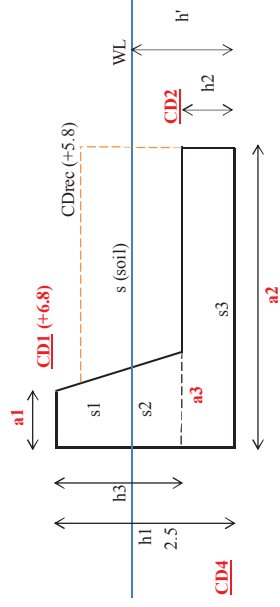
## C. Safety Factor

μ (friction factor between conc. & stone):	0.60
W (total), t/m	9.44
μ (friction factor between stone & stone):	0.80

S.F. (Normal)	0.68
S.F. (Soil effect)	2.06 OK

Mu	6.02	1.76
Mp	5.55	1.73
M <sub>Pp</sub>	1.38	1.38
M <sub>w (=W*)</sub>	13.90	14.32
S.F. (Normal)	1.42	7.26
S.F. (Soil Effect)	1.89	36.10

CD1	5 years	50 years
CD2	6.8	
CD4	5.5	
WL	4.3	
	3.92	4.43
a1	0.5	m
a2	2.5	m
a3	1.15	m
a3*	1.15	1.15 m
h1	2.5	m
h2	1.2	m
h3	1.3	m
h3*	1.3	1.3 m
h'	-0.38	0.13 m
s1	1.0725	1.07 m <sup>2</sup>
s2	0.00	0.00 m <sup>2</sup>
s3upperWL	3	2.675 m <sup>2</sup>
s3underWL	0	0.325 m <sup>2</sup>
γ	2.4	t/m <sup>3</sup>
γ'	1.37	
W (s1)	2.57	2.57 t
W (s2)	0.00	0.00 t
W (s3)	7.20	6.87 t
<b>W (total)</b>	<b>9.77</b>	<b>9.44 t</b>



Conc.	X <sub>1</sub>	2.07
	M (s1)	5.32
	X <sub>2</sub>	0.00
	M (s2)	0.00
	X <sub>3</sub>	1.25
	M (s3)	9.00
	M <sub>w</sub>	14.32
	Pp upperWL	3.46
	Pp underWL	0.00
Soil pressure	γ <sub>soil</sub> upper	0.4000
	γ <sub>soil</sub> under	0.0000
	Mp up	1.38
	Mp un	0.00
	Mp (total)	1.38
	P	2.08
	U	1.05
	γ <sub>s</sub>	0.833
	X <sub>u</sub>	1.667
	Mp	1.73
	Mu	1.76
	M total	3.49
		11.57

17.8.(1) STABILITY CALCULATION OF CONCRETE CAP OF OUTER REVETMENT C (ORB-C&ORB-D) (for reference)

A. Wave Force under Wave Crest

1. Input data:

	50 years	5 years
Tidal Level HHWL(CDL):	1%(100 yr) 4.43	20%(20 yr) 3.92
Crest Level(CDL):	6.8	
Bottom Upright Wall Level (CDL):	4.6	
Bottom Width of Upright wall (m):	2.5	
Reclamation Level (CDL):	5.5	
Crest Level of Armor stone (CDL):		
Seabed slope:		
Seabed sSurface Level (CDL):	0	
Significant Wave Height ( $H_{1/3}$ ),m:	3.2	2.3
$H_{max}$ (Goda's Index),m:	4.3	3.4
Wave Period (T),sec:	13.3	11
Wave Length (L),m:	84.97	62.33

2. Results of Calculation

2.1. Input data

	50 years	5 years
$\beta$ (deg)=	66	
$\cos\beta$ :	0.407	
$H_D$ (m):	4.3	3.4
L(m):	84.97	62.33
HHWL(CDL):	4.43	3.92
Sea bed Level(CDL):	0	
Crest Level(m):	6.8	
h (m):	4.43	3.92
h' (m):	-0.17	-0.68
$\rho_0$ (t/m <sup>3</sup> ):	1.025	
g (m/s <sup>2</sup> ):	9.81	
hc (m):	2.07	2.58
$4\pi h/L$ :	0.655	0.790
$2\pi h/L$ :	0.328	0.395
$\lambda$ :	0.866	0.704

2.2. Wave pressure on the front face of an Upright wall

$\eta^+$ :	3.93	2.53
$\alpha_1$ :	1.034	1.008
$\alpha_3$ :	1.002	1.013
$h_c^+ = \min(\eta^+, h_c)$ :	2.07	2.53
$\alpha_2$ :	0.473	0.000
$(1 + \cos\beta)$ :	1.407	
$\alpha_1 \lambda \rho_0 H_D$ :	3.948	2.472
$p_1$ (t/m <sup>2</sup> ):	2.78	1.74
$p_3$ (t/m <sup>2</sup> ):	2.78	1.76
$p_4$ (t/m <sup>2</sup> ):	1.31	0.00
$p_c$ (at crest level) (t/m <sup>2</sup> )=	1.28	-0.05

2.3. Uplift beneath Upright Wall

Pu	2.78	1.76
lu	2.50	1.00

2.4. Total wave pressure on Upright Wall

P	4.23	2.20
U	3.48	0.88

B. Effect of Reclamation Soil

	50 years	5 years
H(m)	0.9	
$\phi$ (deg)	30	
$\gamma_s$ (t/m <sup>3</sup> )	1.8	
$\gamma'_s$ (t/m <sup>3</sup> )	1	
$h_{dry}$ (m)	0.9	0.9
$h_{wet}$ (m)	0	0
$K_p = \tan^2(45^\circ + \phi/2)$ :	3	
$P_s$ (dry), t/m <sup>2</sup> :	4.86	4.86
$P_s$ (wet), t/m <sup>2</sup> :	0.00	0.00
$P_p$ (dry), t/m:	2.19	2.19
$P_p$ (wet), t/m:	0.00	0.00
$P_p$ (final), t/m:	2.19	2.19

C. Safety Factor

$\mu$ (coc.&stone)	0.6	
W	8.65	8.65
$\mu$ (stone&stone)	0.8	

Sliding	S.F.(normal)	0.73	2.12
	S.F.(soil effect)	1.25	3.12

Overturning	Mp	3.54	1.39
	MPp	0.66	0.66
	Mw	12.55	12.55
	S.F.(Normal)	1.91	7.65
	S.F.(Soil effect)	2.09	8.12

Concrete Cap

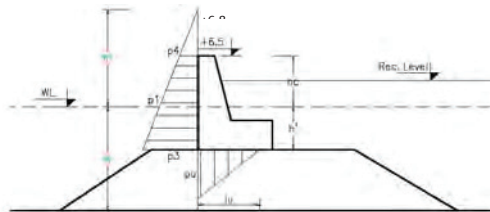
	50 years	5 years
CD1	6.8	6.8
CD2	5.7	5.7
CD4	4.6	4.6
CD Rec.	5.5	5.5
WL	4.43	3.92
a1	0.5	0.5
a2	2.5	2.5
a3	1.05	1.05
a3'	1.05	1.05
h1	2.2	2.2
h2	1.1	1.1
h3	1.1	1.1
h3'	1.1	1.1
h'	-0.17	-0.68
s1	0.8525	0.8525
s2	0	0
s3upperWL	2.75	2.75
s3underWL	0	0
ES	3.6025	3.6025
$\gamma$	2.4	2.4
$\gamma'$	1.37	1.37
W (s1)	2.05	2.05
W (s2)	0.00	0.00
W (s3)	6.60	6.60
W (total)	8.65	8.65

Concrete	$x_{s1}$	2.1	2.1
	M (s1)	4.30	4.30
	$x_{s2}$	0	0
	M (s2)	0	0
	$x_{s3}$	1.25	1.25
M (s3)	8.25	8.25	
Mw	12.55	12.55	

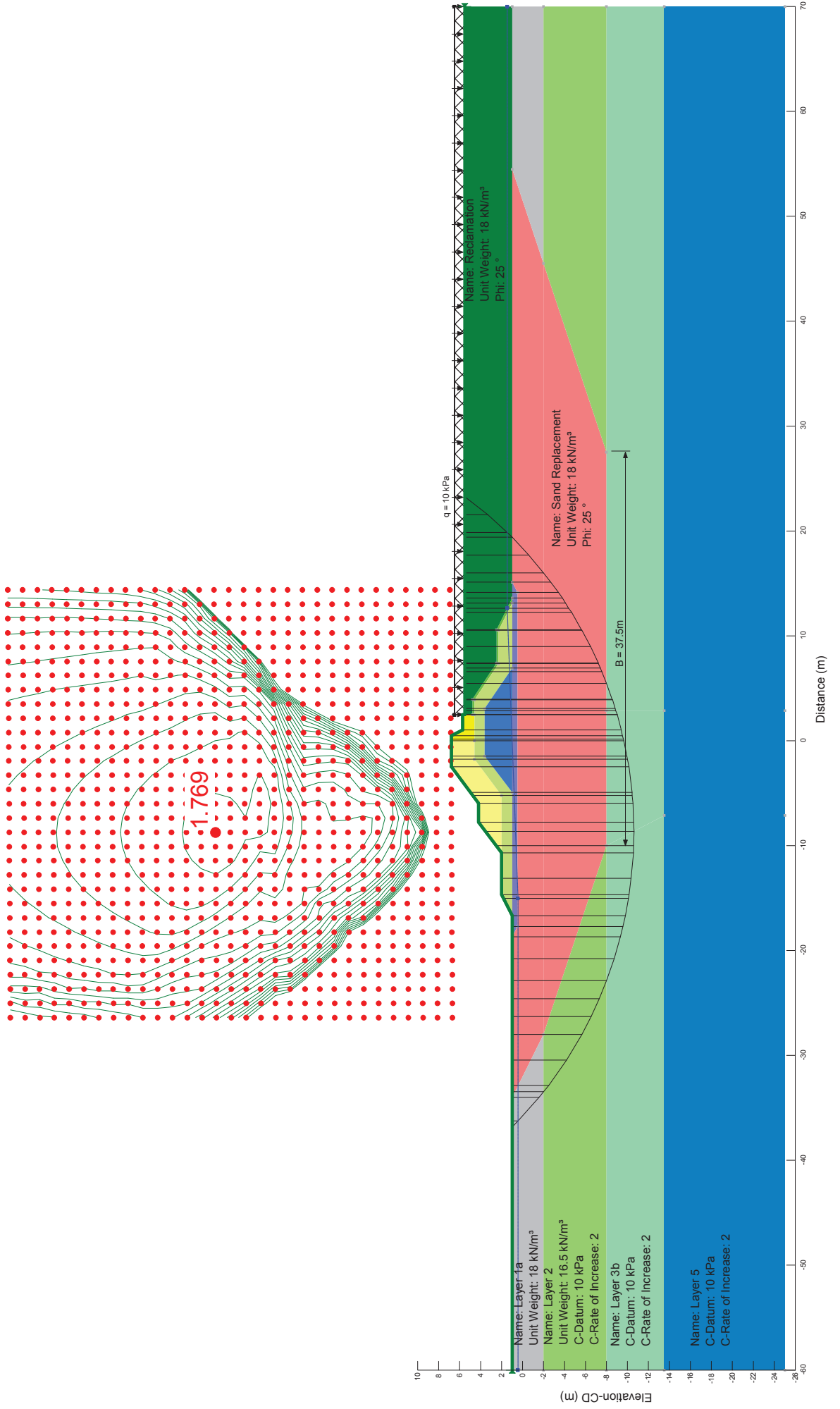
Soil pressure	Pp upperWL	2.19	2.19
	Pp underWL	0.00	0.00
	yp.upper	0.30	0.30
	yp.under	0	0
	Mp upperWL	0.66	0.66
	Mp underWL	0.00	0.00
	Mp total	0.66	0.66

Wave	P	4.23	2.20
	U	3.48	0.88
	yp	0.84	0.63
	xu	1.67	2.17
	Mp	3.54	1.39
	Mu	5.80	1.91
	M total	9.34	3.30

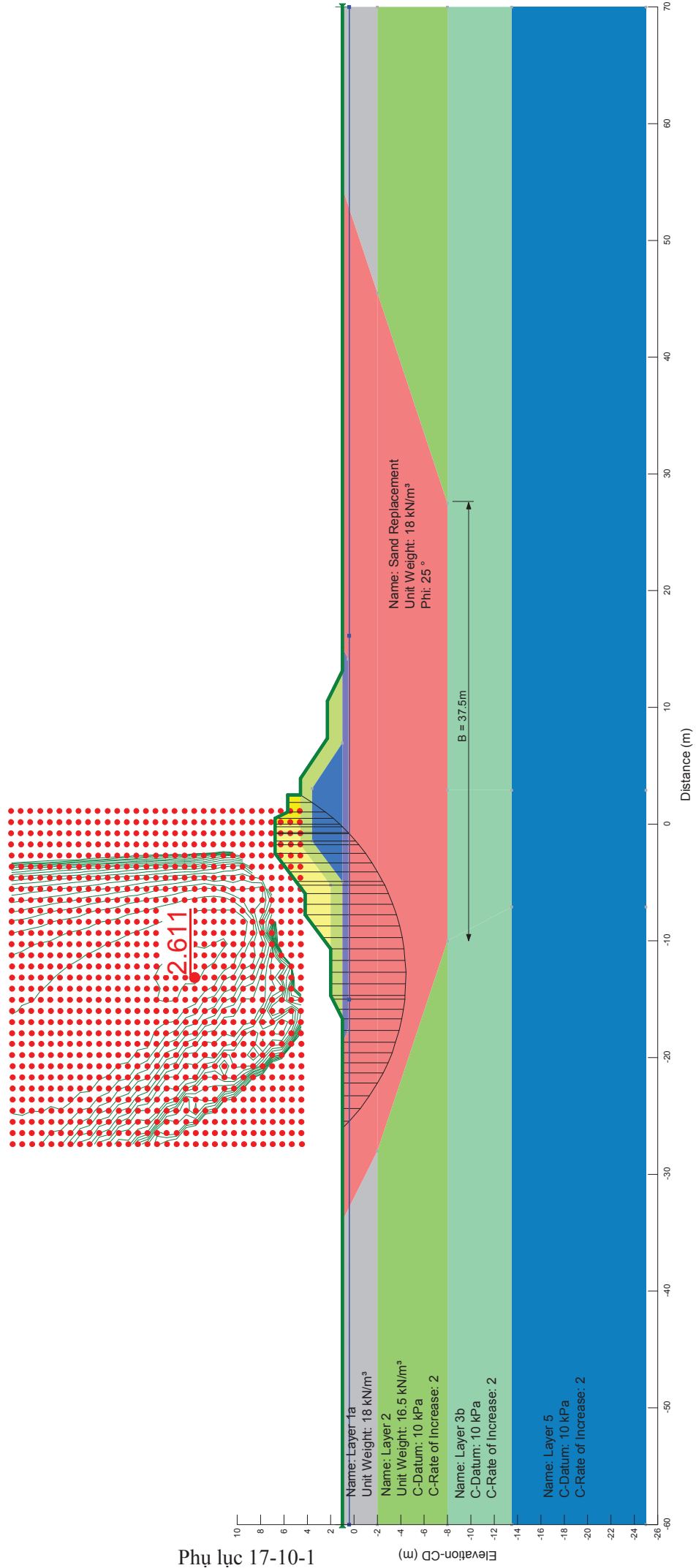
Wave Pressure Distribution Used in Design Calculation



# REVTMENT B\_AFTER 5 YRS - RECOMMENDED (FELLENIOUS METHOD - LONG TERM) Soil Condition at ORB-A



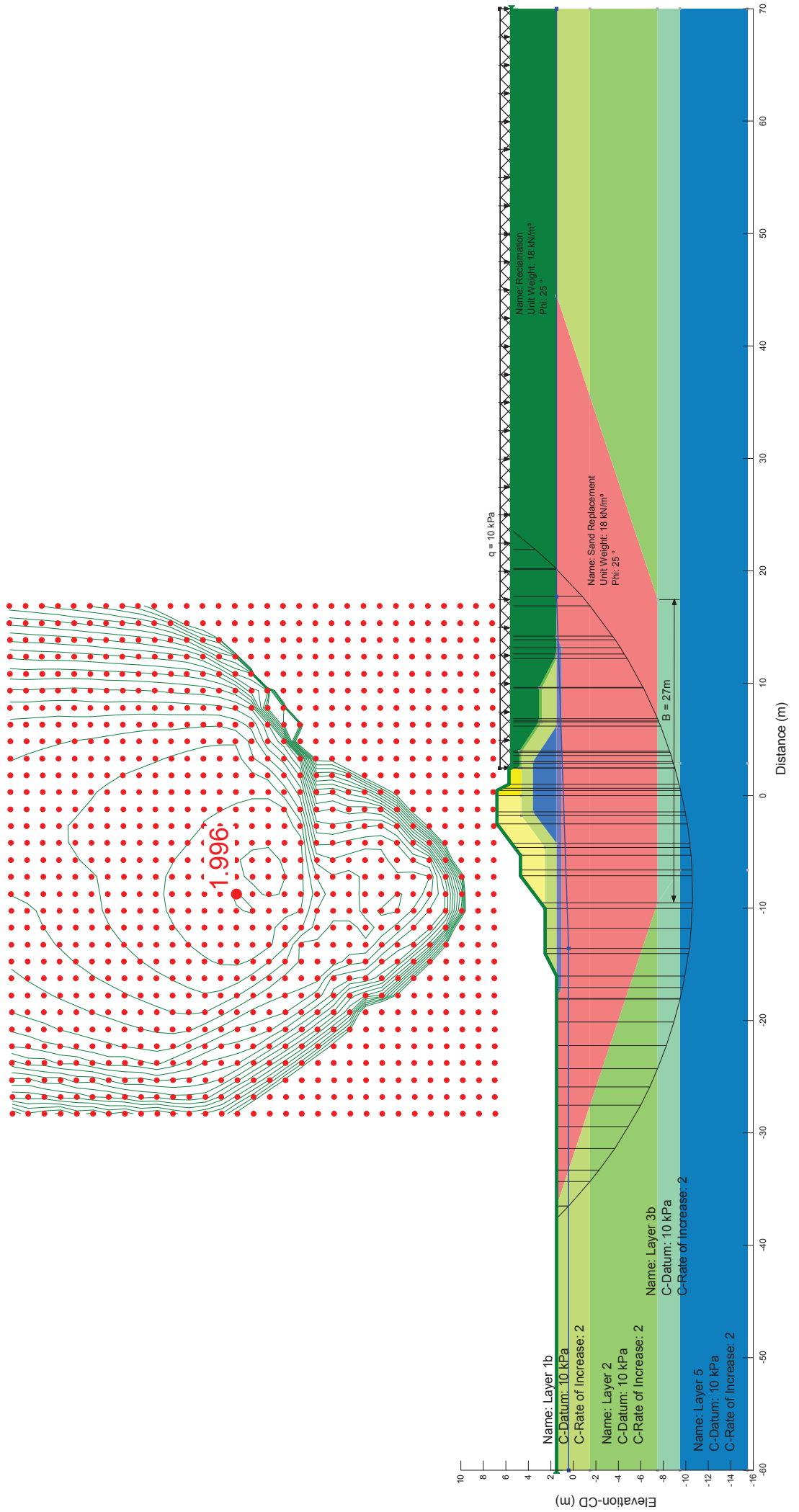
# REVETMENT B\_UNTIL 5 YRS - RECOMMENDED (FELLENIUS METHOD - SHORT TERM) Soil Condition at ORB-A



Phụ lục 17-10-1

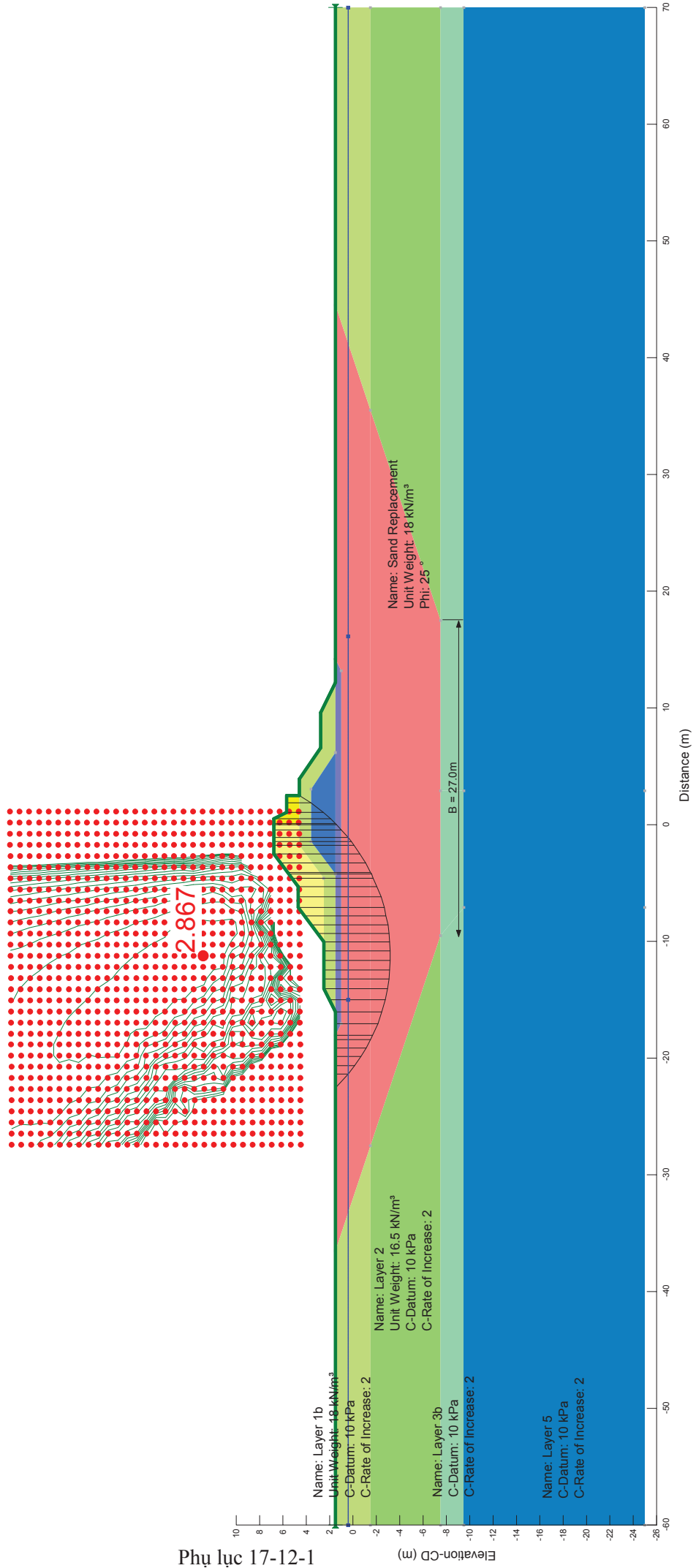


# REVETMENT B\_AFTER 5 YRS - RECOMMENDED (FELLENIOUS METHOD - LONG TERM) Soil Condition at ORB-B



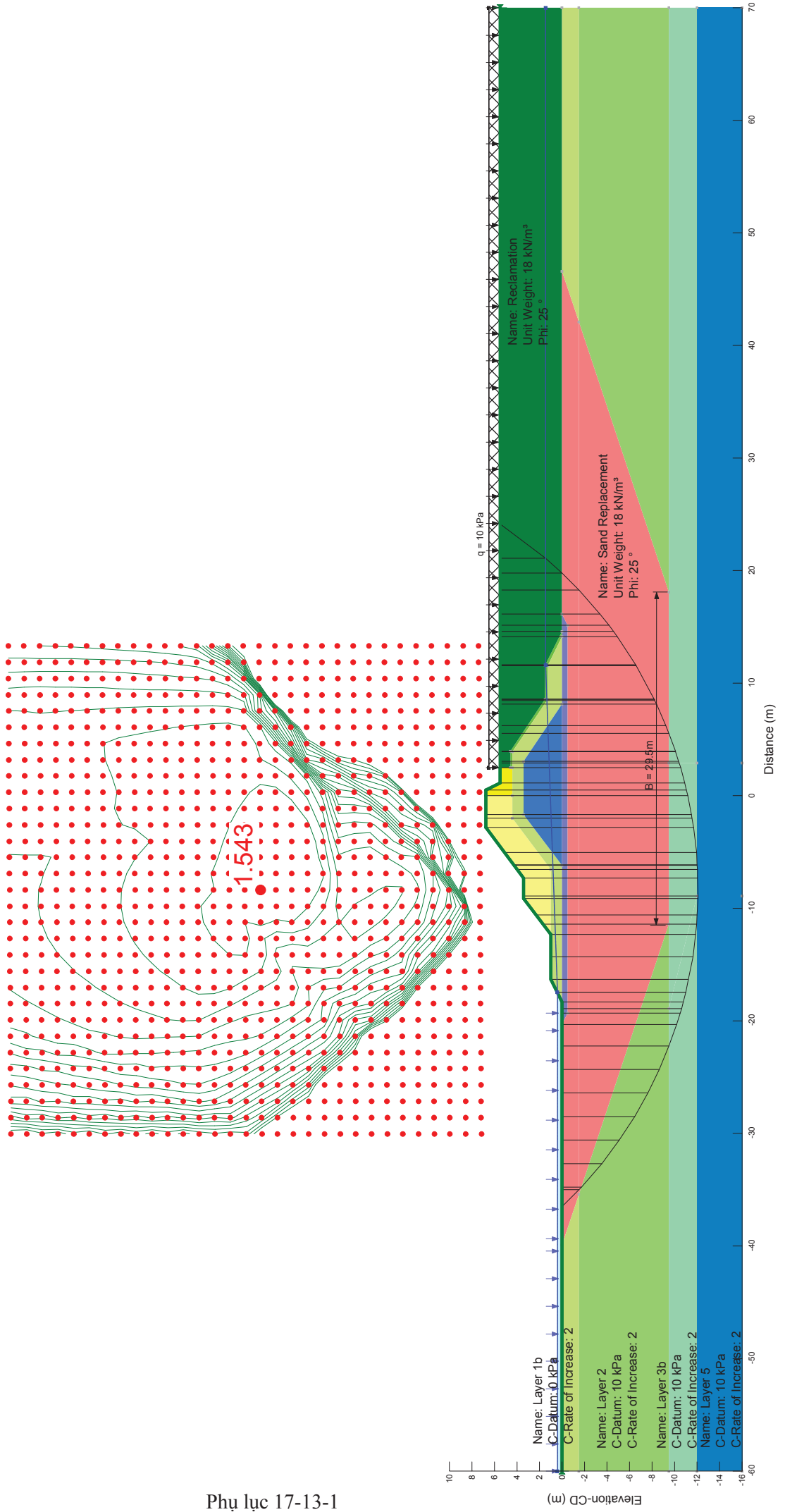
# RETIVEMENT B\_UNTIL 5 YRS - RECOMMENDED (FELLENIIUS METHOD - SHORT TERM)

## Soil Condition at ORB-B

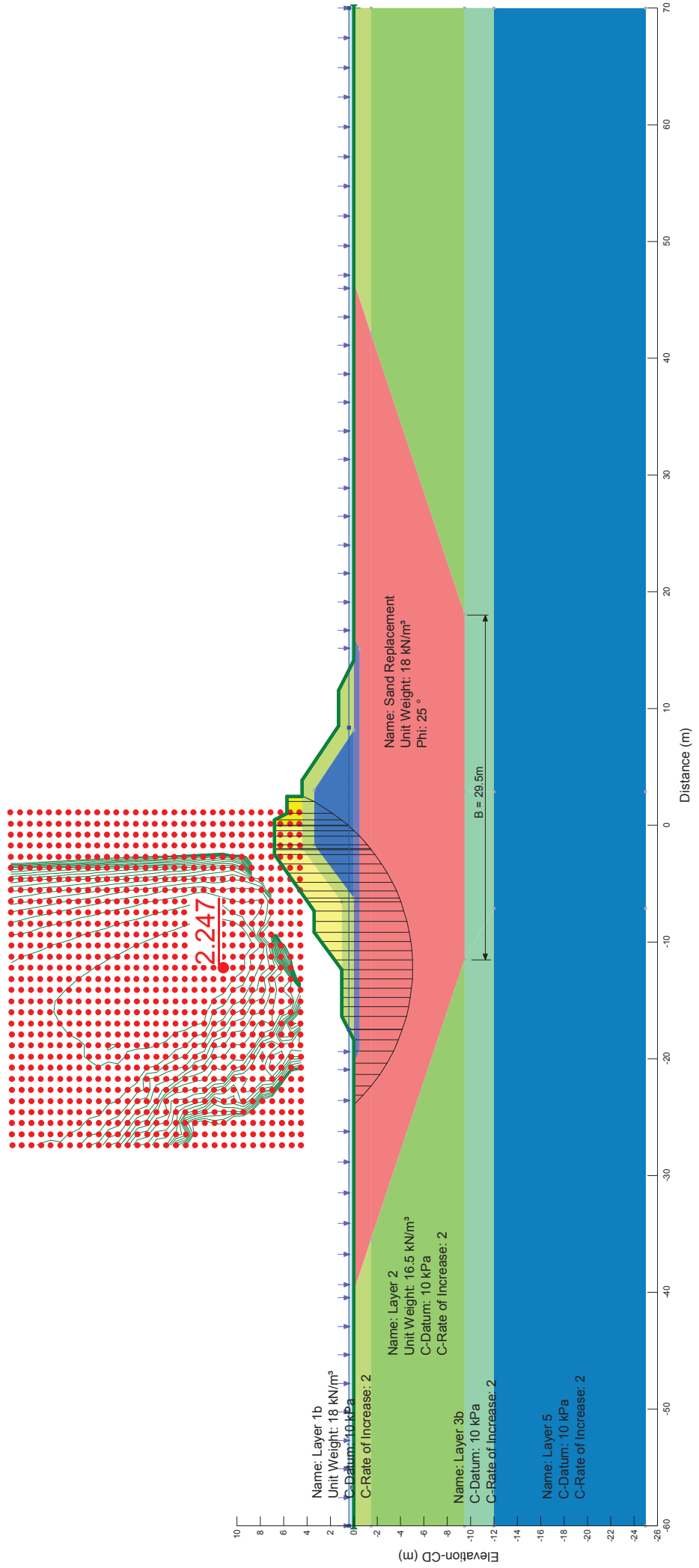


Phụ lục 17-12-1

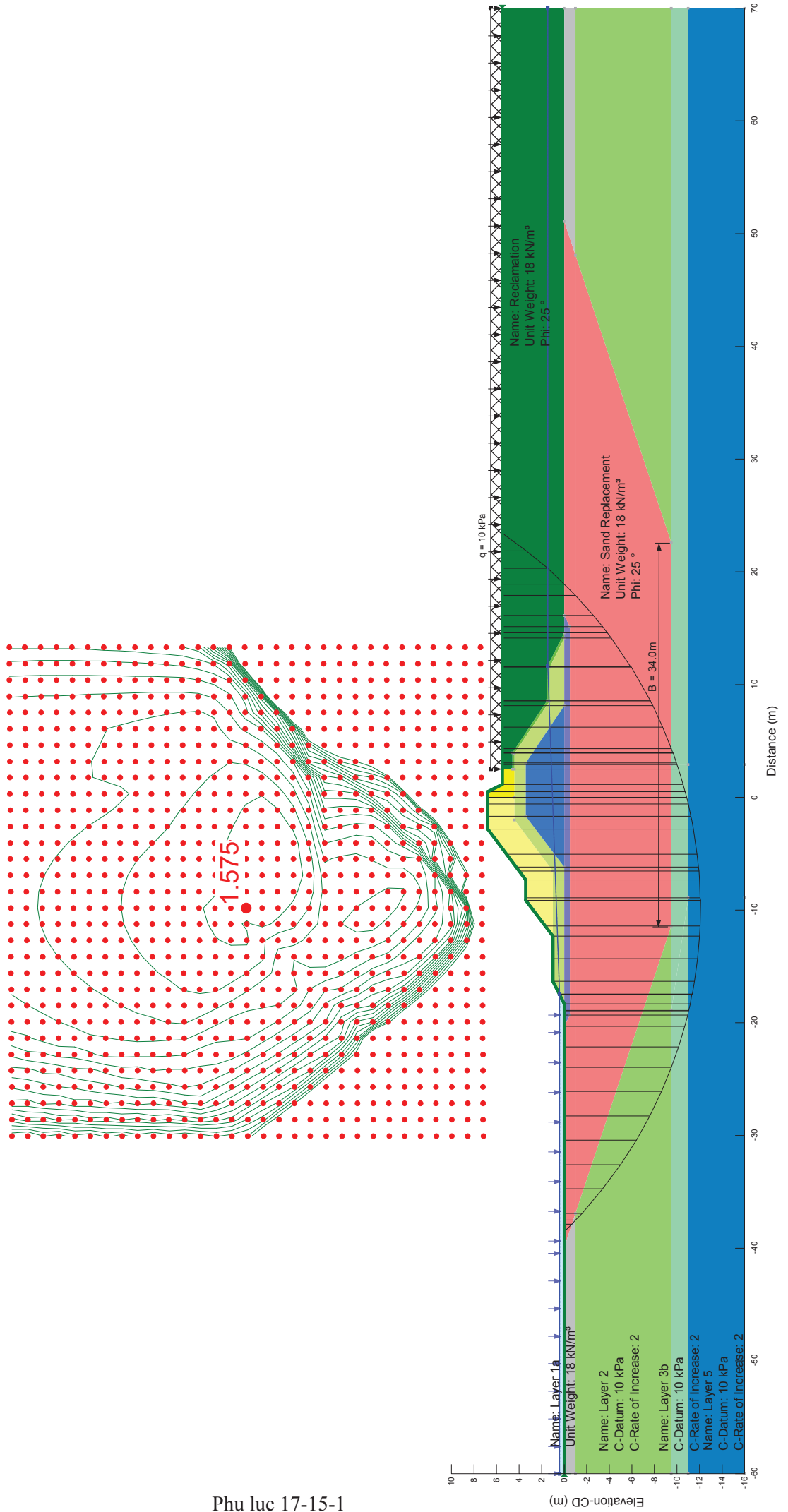
# REVETMENT B\_AFTER 5 YRS - RECOMMENDED (FELLENIOUS METHOD - LONG TERM) Soil Condition at ORB-C



# REVETMENT B\_UNTIL 5 YRS - RECOMMENDED (FELLENIUS METHOD - SHORT TERM) Soil Condition at ORB-C

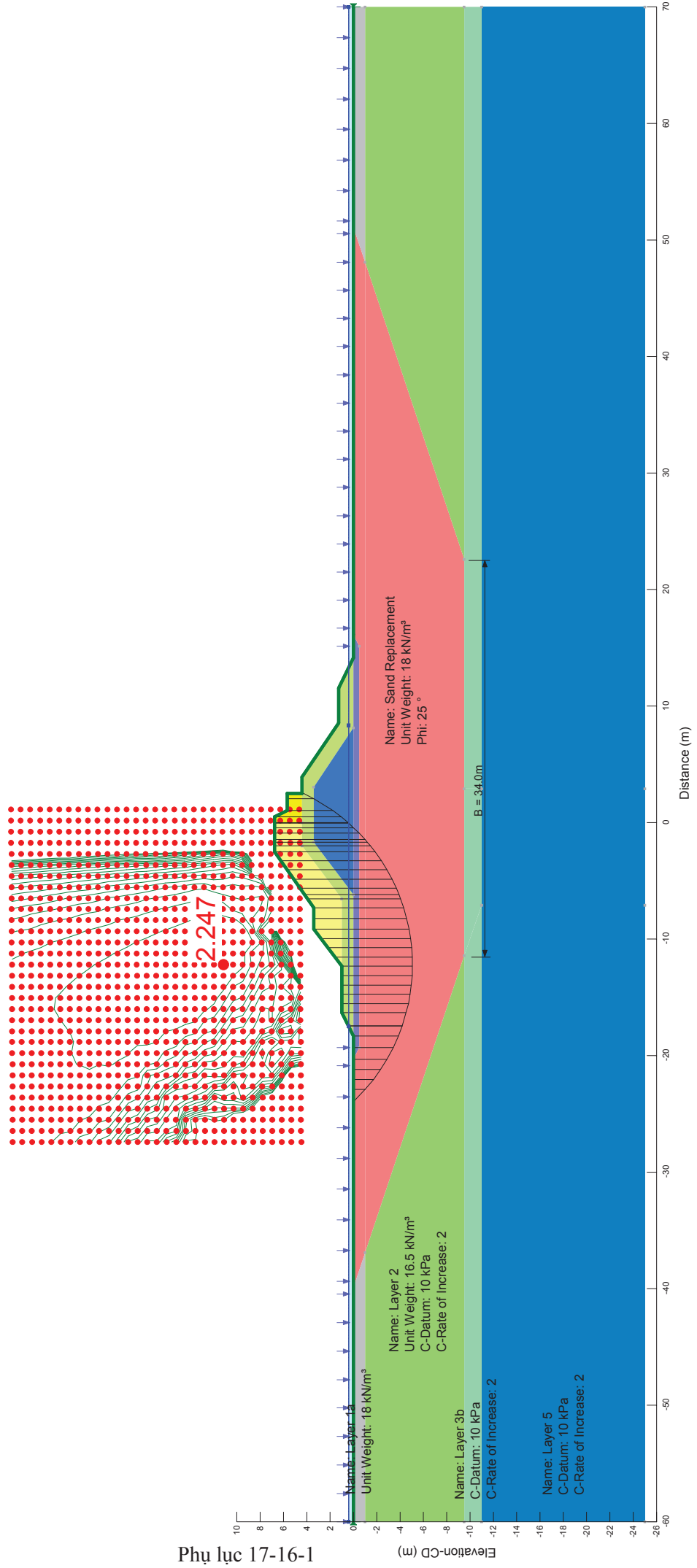


# REVTMENT B\_AFTER 5 YRS - RECOMMENDED (FELLENIUS METHOD - LONG TERM) Soil Condition at ORB-D



# REVETMENT B\_UNTIL 5 YRS - RECOMMENDED (FELLENIUS METHOD - SHORT TERM)

## Soil Condition at ORB-D



Phụ lục 18-1

Hệ thống bảo vệ bằng biện pháp chống ăn mòn ca-tốt  
cho cọc ván thép

1. GENERAL

This specification covers the design of the cathodic protection system for the steel sheet pile of LACH HUYEN PORT PROJECT.

2. CODES AND STANDARDS

- Corrosion Protection and Repair Manual for Port and Harbor Steel Structures(2009) (Coastal Development Institute & Technology, Japan)

3. DESIGN CONDITIONS

(a) Protection Method

Galvanic anode system using aluminum alloy anodes

(b) Seawater Resistivity

Sea water: 35 ohm-cm as average

It is based on the following site survey results

Salinity: Min.8.9, Max.31.8, Ave. 25.5ppm (Middle, High Tide)

Salinity: Min.10.2, Max.31.7, Ave. 22.6ppm (Middle, Low Tide)

Salinity: Min.8.9, Max.31.8, Ave. 24.05ppm

$$S(\%) = 2.320186 \times 10^{-16} \times C_3^4 - 4.83885888 \times 10^{-12} \times C_3^3 + 4.2677569 \times 10^{-8} \times C_3^2 + 5.0714273 \times 10^{-4} \times C_3 - 1.4470907 \times 10^{-3}$$

S(%): Salinity, C(mS/m): Conductivity

Source: The Chemical Society of Japan, Kagaku Binran (Handbook of Chemistry)

(Only Japanese version)

Chloride ion (Cl<sup>-</sup>) (ppm) is calculated with the following formula

Cl<sup>-</sup> (ppm)= Salinity (NaCl) (%) x 10000(ppm/%) x Cl(g/mol) / NaCl(g/mol)

= Salinity (NaCl) (%) x 10000(ppm/%) x 35.45 / 58.44

= Salinity (NaCl) (%) x 6066

Calculation Result

Resistivity: Min.20 ohm.cm, Ave.27 ohm.cm, Max.80 ohm.cm

{Cl<sup>-</sup> (ppm): Min.5400ppm, Ave.14590 ppm, Max.19300 ppm} from salinity

(c) Anode Life : 50 years

(d) Seawater evaluation (clean or polluted)

Criteria of Corrosion Protection and Repair Manual for Port and Harbor Steel Structures (2009)	
Polluted	Cl <sup>-</sup> (ppm): <17800 NH <sub>4</sub> <sup>+</sup> (ppm): >0.1 DO(ppm): <6
Clean	Cl <sup>-</sup> (ppm): >17800 NH <sub>4</sub> <sup>+</sup> (ppm):<0.1 DO(ppm): >6

Data(Middle only)	Judgment
Cl <sup>-</sup> (ppm): Ave.14590 (<17800) NH <sub>4</sub> <sup>+</sup> (ppm): Ave.0.4975(>0.1) DO(ppm): Ave.5.78(<6)	<b>Polluted</b>



Criteria of Corrosion Protection and Repair Manual for Port and Harbor Steel Structures (2009)	
Polluted or Clean	Protective Current Density(A/sq.m)
Clean	In Seawater:0.1, In sea mud:0.02
<b>Polluted</b>	<b>In Seawater:0.13 to 0.15, In sea mud:0.026 to 0.03</b>

(e) Protective Current Density

In Sea water: 0.13 A/m<sup>2</sup> (Initial), 0.065 A/m<sup>2</sup> (Average)

In Sea mud: 0.026 A/m<sup>2</sup> (Initial), 0.013 A/m<sup>2</sup> (Average)

(f) Object to be cathodically protected

Steel Sheet Pile exposed to sea water and sea mud (below L.W.L-1.03m),  
L.W.L.=+0.43m

Note

1) Current drain (Loss current) ; Bare pile connection(+1m to -0.6m) 10% of surface area

2) Coating: Polyethylene or Polyurethane (above L.W.L-1.03m)( Coating damage: negligible)

3) Current speed: Max.0.5m/sec.

4) Floating sand attack to the pile surface: negligible

5) Storm effect: negligible

6) Corrosion by biological bacteria (SRB) in sea mud: negligible

4. STRUCTURE TO BE PROTECTED

Steel Sheet Pile (VL), h=200mm, w=500mm, Surface factor:1.7

Concrete Bottom Level: +1.0m

L.W.L:+0.43m

Coating Bottom Level: -0.6m

Seabed Level: -5m

Pile Toe Level: -14m

5. PROTECTED AREA AND REQUIRED PROTECTIVE CURRENT

(1)Bare Pile Connection in Seawater:

$$346.743 \times 1.7 \times (1+0.6) \times 0.1=94.3\text{sq.m}$$

$$94.3\text{sq.m} \times 0.13\text{A/sq.m} =12.3\text{A}$$

(2)Bare in Seawater:

$$346.743 \times 1.7 \times (5-0.6) =2593.6\text{sq.m}$$

$$2593.6\text{sq.m} \times 0.13\text{A/sq.m}=337.2\text{A}$$

(3)Bare in Sea mud:

$$346.743 \times 1.7 \times (14-5) =5305.2\text{sq.m}$$

$$5305.2\text{sq.m} \times 0.026\text{A/sq.m}=137.9\text{A}$$

Total 487.4A

6. ANODE TO BE USED

Material	Aluminum alloy anode :
Dimension	(255+300) x 280 x 1420
Net weight [kg]	295.6
Gross weight [kg]	312.0
Capacity[Ah/kg]	2600
Closed circuit Potential(mV vs. SCE)	-1050
Output current (Initial) [A/pc.] At 35 ohm.cm	3.5
Output current (Average) [A/pc.]	1.75
Life time[year]	50

7. ANODE QUANTITY

140pcs. (487.4A / 3.5A/pc.)

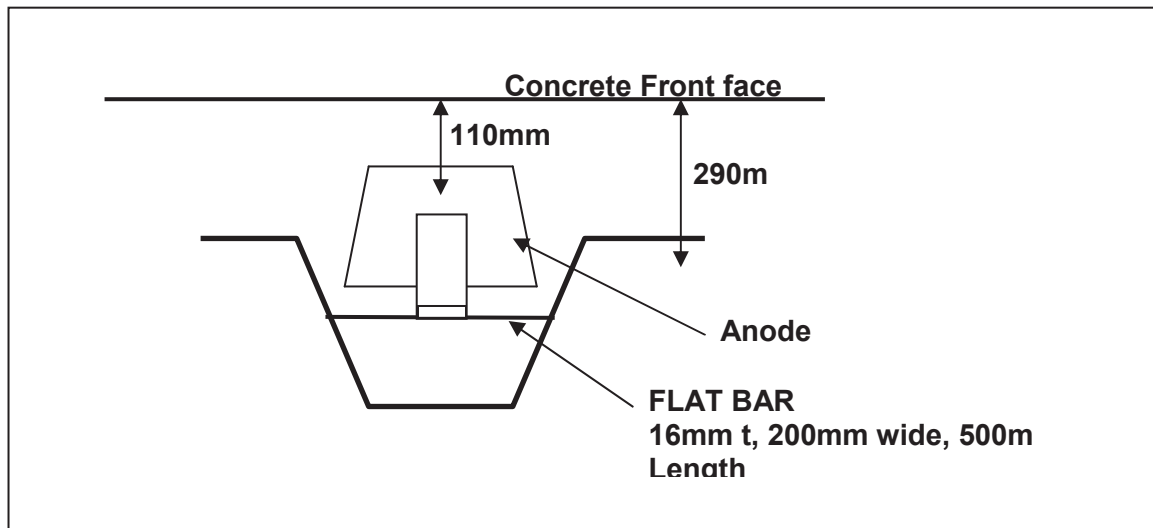
8. POTENTIAL MEASURING TERMINAL

Specification of potential measuring terminal

Material: Stainless Steel SS304

Dimension: 120mm Dia.x 20mm high

Quantity: Total 7 pcs.



Phụ lục 19 (19-1 - 19-17)

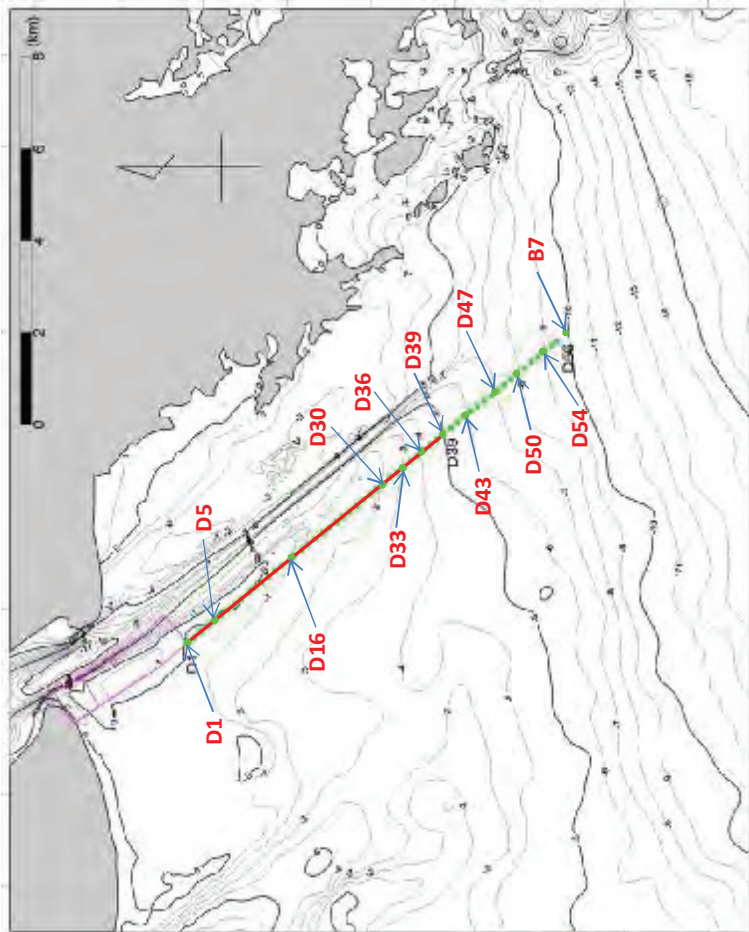
Các tính toán phân tích cho Đê chắn cát

## **APPENDIX: CALCULATION ANALYSIS OF SAND PROTECTION TRAINING DIKE**

### ***Table of Contents***

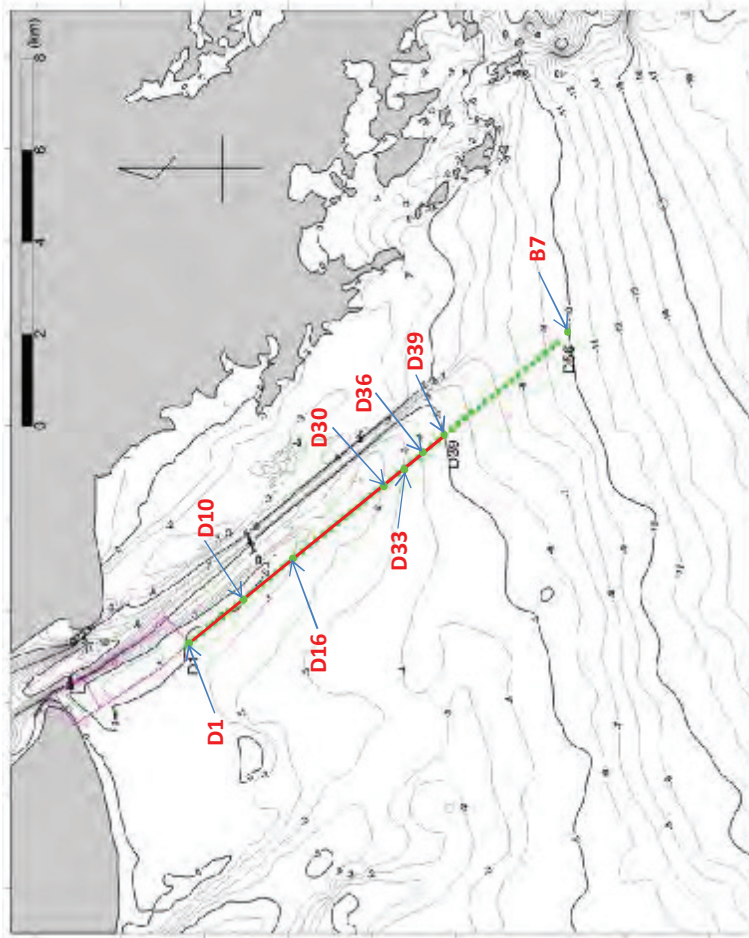
- 1 Summary of Wave Conditions due to HWL
- 2 Summary of Wave Conditions due to LWL
- 3 Stability Coefficient Calculation ( $K_D$ )
- 4 Tetrapod Weight Calculation
- 5 Stability of toe berm in front of vertical impermeable wall structure
- 6 Stability Calculation Due to Wave Force of Sand Protection Training Dike - Case A-1 (HWL)
- 7 Stability Calculation Due to Wave Force of Sand Protection Training Dike - Case A-2 (HWL)
- 8 Stability Calculation Due to Wave Force of Sand Protection Training Dike - Case B-1 (HWL)
- 9 Stability Calculation Due to Wave Force of Sand Protection Training Dike - Case B-2 (HWL)
- 10 Stability Calculation Due to Wave Force of Sand Protection Training Dike - Case A-1 (LWL)
- 11 Stability Calculation Due to Wave Force of Sand Protection Training Dike - Case A-2 (LWL)
- 12 Stability Calculation Due to Wave Force of Sand Protection Training Dike - Case B-1 (LWL)
- 13 Stability Calculation Due to Wave Force of Sand Protection Training Dike - Case B-2 (LWL)
- 14 Summary of Cellular caisson dimension & Safety factor
- 15 Slope Stability Analysis of Sand Protection Training Dike - G.L. -5.0m
- 16 External force calculation of Cellular Caisson
- 17 Reinforcement calculation of Cellular Caisson

1. SUMMARY OF WAVE CONDITIONS (HWL)



Period	Pos	Tide level	Seabed	To	Offshore side					Channel side											
					DIR	Hmax	Beta	L	H1/3	H1/3	Beta	L	Hmax	Beta	L	H1/3	H1/3	Beta	L		
30 years	D5	HWL	0.0	12.8 SSE	3.82	68.52	77.14	2.67	68.52	64.73	ESE	3.51	-72.63	74.06	2.54	-72.63	74.06	2.54	-72.63	74.06	63.23
	D16	HWL	-1.0	12.8 SSE	4.23	68.35	81.06	3.08	68.35	69.51	ESE	4.07	-72.31	79.53	3.00	-72.31	79.53	3.00	-72.31	79.53	68.55
	D30	HWL	-2.0	12.8 SSE	4.76	63.33	85.78	3.58	63.33	74.72	E	4.57	-72.87	84.05	3.47	-72.87	84.05	3.47	-72.87	84.05	73.66
	D33	HWL	-3.0	12.8 SSE	5.37	60.79	90.82	4.08	60.79	79.62	E	5.19	-69.78	89.36	3.99	-69.78	89.36	3.99	-69.78	89.36	78.73
	D36	HWL	-4.0	12.8 SSE	6.04	60.93	96.05	4.64	60.93	84.68	ESE	5.95	-74.63	95.41	4.59	-74.63	95.41	4.59	-74.63	95.41	84.29
	D39	HWL	-5.0	12.8 SSE	6.62	60.95	100.31	4.96	60.95	87.47	ESE	6.53	-73.63	99.67	4.79	-73.63	99.67	4.79	-73.63	99.67	86.05
	D43	HWL	-6.0	12.8 SSE	7.31	60.77	105.13	5.01	60.77	87.89	ESE	7.23	-72.27	104.59	4.85	-72.27	104.59	4.85	-72.27	104.59	86.51
	D47	HWL	-7.0	12.8 SSE	7.93	60.21	109.23	5.07	60.21	88.37	ESE	7.84	-71.22	108.63	4.90	-71.22	108.63	4.90	-71.22	108.63	86.93
	D50	HWL	-8.0	12.8 SSE	8.44	60.17	112.43	5.10	60.17	88.61	ESE	8.20	-69.94	110.93	4.95	-69.94	110.93	4.95	-69.94	110.93	87.39
	D54	HWL	-9.0	12.8 SSE	8.52	59.97	112.94	5.14	59.97	89.00	ESE	8.29	-68.43	111.49	5.00	-68.43	111.49	5.00	-68.43	111.49	87.84
	B7	HWL	-10.0	12.8 SSE	8.63	60.12	113.60	5.06	-66.83	88.29	ESE	8.38	-66.83	112.04	5.19	60.12	112.04	5.19	60.12	112.04	89.39

2. SUMMARY OF WAVE CONDITIONS (LWL)



Period	Pos	Tide level	Seabed	To	Offshore side				Channel side								
					DIR	Hmax	Beta	L	H1/3	H1/3	Beta	L	Hmax	Beta	L	H1/3	H1/3
30 years	D10	LWL	0.0	12.8 SSE	3.14	32.35	70.13	0.93	68.52	38.51	ESE	1.84	-71.19	53.94	0.71	-71.20	33.68
	D16	LWL	-1.0	12.8 SSE	3.73	57.02	76.21	1.48	57.02	48.48	ESE	3.00	-74.02	68.53	1.36	-74.00	46.49
	D30	LWL	-2.0	12.8 SSE	2.87	65.47	67.12	1.95	65.47	55.56	E						
	D33	LWL	-3.0	12.8 SSE	3.34	61.33	72.30	2.43	61.33	61.84	E						
	D36	LWL	-4.0	12.8 SSE	4.01	60.94	78.98	2.99	68.62	68.42	E	3.92	-71.90	78.10	2.93	-71.90	67.80
	D39	LWL	-5.0	12.8 SSE	4.60	60.68	84.32	3.46	68.92	73.48	E	4.49	-69.80	83.39	3.41	-69.80	73.00
	D43	LWL	-6.0	12.8 SSE	5.30	60.37	88.23	4.04	60.37	79.21	E	5.20	-67.35	89.43	3.98	-67.30	78.68
	D47	LWL	-7.0	12.8 SSE	5.92	59.65	95.19	4.55	59.65	83.88	E	5.80	-65.50	94.25	4.48	-65.50	83.30
	D50	LWL	-8.0	12.8 SSE	6.47	59.60	99.27	4.96	59.62	87.44	E	6.40	-73.73	98.77	4.83	-73.73	86.37
	D54	LWL	-9.0	12.8 SSE	7.17	59.43	104.19	5.02	59.43	87.99	ESE	7.09	-72.05	103.64	4.89	-72.05	86.91
	B7	LWL	-10.0	12.8 SSE	7.82	59.72	108.50	5.08	59.72	88.47	ESE	7.73	-70.08	107.93	4.95	-70.08	87.40

**3. STABILITY COEFFICIENT CALCULATION ( $K_D$ )**  
**(According to "The Rock Manual, 2nd edition")**  
**Armour Concrete Unit Layer**

**Stability parameter:**

$$N_s = \frac{H_s}{\Delta D_n} = (K_D \cot \alpha)^{\frac{1}{3}} \quad (Eq. 5.134)$$

$$K_D = \left( \frac{H_s}{\Delta D_n} \right)^3 \frac{1}{\cot \alpha}$$

Where:  $N_s$  - stability parameter

$H_s$  - significant wave height,  $H_{1/3}$  of the incident waves at the toe of the structure (m)

$\Delta$  - relative buoyant density,  $\rho_c/\rho_w - 1$

$\alpha$  - slope angle ( $^\circ$ )

**Surf similarity parameter, using the energy wave period  $T_m$ :**

$$\xi_m = \frac{\tan \alpha}{\sqrt{\frac{2\pi H_s}{g T_m^2}}}$$

**Critical surf similarity parameter:**

$$\xi_{cr} = \left[ \frac{c_{pl}}{c_s} P^{0.31} \sqrt{\tan \alpha} \right]^{\frac{1}{P+0.5}} \quad (Eq. 5.138)$$

For  $\xi_m < \xi_{cr}$  wave are plunging and Equation (5.154) applies:

$$\frac{H_s}{\Delta D_n} = \left( 8.6 \left( \frac{N_{od}}{\sqrt{N}} \right)^{0.5} + 3.94 \right) s_{om}^{0.2} \quad (Eq. 154)$$

For  $\xi_m > \xi_{cr}$  wave are surging and Equation (5.153) applies:

$$\frac{H_s}{\Delta D_n} = \left( 3.75 \left( \frac{N_{od}}{\sqrt{N}} \right)^{0.5} + 0.85 \right) s_{om}^{-0.2} \quad (Eq. 153)$$

Where:  $c_{pl} = 6.2$ , with a standard deviation of  $\sigma = 0.4$

$c_s = 1$ , with a standard deviation of  $\sigma = 0.08$

$P$  - notional permeability of structure,  $0.1 < P < 0.6$  (see Fig. 5.39)

$N$  - number of incident waves at the toe, which depends on the duration of the wave conditions

$s_{om}$  - fictitious wave steepness,  $s_{om} = \frac{2\pi H_s}{g T_m^2}$

$N_{od}$  - the number of displaced unit

$T_m$  - the mean wave period (s)

$\rho_c$ (t/m <sup>3</sup> )	$\rho_w$ (t/m <sup>3</sup> )	$\Delta$ (t/m <sup>3</sup> )	$\alpha$ ( $^\circ$ )	$g$ (m/s <sup>2</sup> )	$P$	$N_{od}$	Damage level	$N$
2.3	1.025	1.24	26.57	9.81	0.5	0.5	5%	3000

### 3. STABILITY COEFFICIENT CALCULATION ( $K_D$ )

Return period	Pos	$H_s$ (m)	$T_m$ (s)	$S_{om}$	$\xi_m$	$\xi_{scr}$	$N_s$	$K_D$
30 yrs Offshore side	D5	2.67	6.30	0.043	2.41	3.54	2.54	<b>8.2</b>
	D16	3.08	6.78	0.043	2.41	3.54	2.54	<b>8.2</b>
	D30	3.58	7.30	0.043	2.41	3.54	2.54	<b>8.2</b>
	D33	4.08	7.80	0.043	2.41	3.54	2.54	<b>8.2</b>
	D36	4.64	8.31	0.043	2.41	3.54	2.54	<b>8.2</b>
	D39	4.96	8.60	0.043	2.41	3.54	2.54	<b>8.2</b>
	D43	5.01	8.64	0.043	2.41	3.54	2.54	<b>8.2</b>
	D47	5.07	8.69	0.043	2.41	3.54	2.54	<b>8.2</b>
	D50	5.10	8.71	0.043	2.41	3.54	2.54	<b>8.2</b>
	D54	5.14	8.75	0.043	2.41	3.54	2.54	<b>8.2</b>
	B7	5.06	8.68	0.043	2.41	3.54	2.54	<b>8.2</b>
30 yrs Channel side	D5	2.54	6.15	0.04	2.41	3.54	2.54	<b>8.2</b>
	D16	3.00	6.68	0.04	2.41	3.54	2.54	<b>8.2</b>
	D30	3.47	7.19	0.04	2.41	3.54	2.54	<b>8.2</b>
	D33	3.99	7.71	0.04	2.41	3.54	2.54	<b>8.2</b>
	D36	4.59	8.27	0.04	2.41	3.54	2.54	<b>8.2</b>
	D39	4.79	8.45	0.04	2.41	3.54	2.54	<b>8.2</b>
	D43	4.85	8.50	0.04	2.41	3.54	2.54	<b>8.2</b>
	D47	4.90	8.54	0.04	2.41	3.54	2.54	<b>8.2</b>
	D50	4.95	8.59	0.04	2.41	3.54	2.54	<b>8.2</b>
	D54	5.00	8.63	0.04	2.41	3.54	2.54	<b>8.2</b>
	B7	5.19	8.79	0.04	2.41	3.54	2.54	<b>8.2</b>



## 4. TETRAPOD WEIGHT CALCULATION

Hudson Formula:

$$W = \frac{\rho_s H_s^3}{K_D \cdot \left( \frac{\rho_s}{\rho_w} - 1 \right)^3 \cot \alpha}$$

Where in:

$\rho_s$  - mass density of armor units (t/m<sup>3</sup>) = 2.3 t/m<sup>3</sup>

$\rho_w$  - mass density of water (t/m<sup>3</sup>) = 1.025t/m<sup>3</sup>

$K_D$  - stability coefficient Tetrapod

$\alpha$  - slope angle 1 : 2

$H_s$  - characteristic wave height

Concrete layer placing Qty. 2

### TETRAPOD WEIGHT CALCULATION

Damage level: 5%

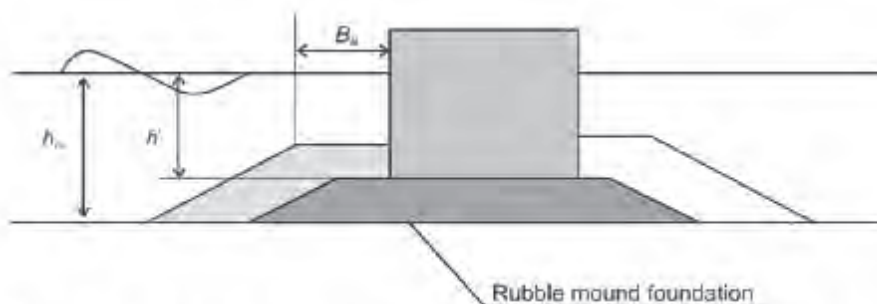
Period	Pos	Seabed Level (m)	Wave height H1/3 (m)	$K_D$	Tetrapod (tons)						
					W (tons)	Type (tons)	Thicknes s (m)	Bs (m)	Row of block placing on top	B (m)	B' (m)
30 year Offshore side	D5	0.0	2.67	8.2	1.38	2.0	1.90	1.30	2	1.60	2.20
	D16	-1.0	3.08	8.2	2.15	3.2	2.20	1.50	2	1.80	2.50
	D30	-2.0	3.58	8.2	3.35	4.0	2.40	1.60	2	2.00	2.80
	D33	-3.0	4.08	8.2	4.97	6.3	2.70	1.90	2	2.30	3.20
	D36	-4.0	4.64	8.2	7.29	8.0	3.00	2.10	2	2.50	3.40
	D39	-5.0	4.96	8.2	8.92	10.0	3.20	2.20	2	2.70	3.70
	D43	-6.0	5.01	8.2	9.20	12.5	3.50	2.40	2	2.90	4.00
	D47	-7.0	5.07	8.2	9.51	12.5	3.50	2.40	2	2.90	4.00
	D50	-8.0	5.10	8.2	9.68	12.5	3.50	2.40	2	2.90	4.00
	D54	-9.0	5.14	8.2	9.95	12.5	3.50	2.40	2	2.90	4.00
	B7	-10.0	5.06	8.2	9.46	12.5	3.50	2.40	2	2.90	4.00
30 year Channel side	D5	0.0	2.54	8.2	1.20	2.0	1.90	1.30	2	1.60	2.20
	D16	-1.0	3.00	8.2	1.97	3.2	2.20	1.50	2	1.80	2.50
	D30	-2.0	3.47	8.2	3.07	4.0	2.40	1.60	2	2.00	2.80
	D33	-3.0	3.99	8.2	4.63	6.3	2.70	1.90	2	2.30	3.20
	D36	-4.0	4.59	8.2	7.08	8.0	3.00	2.10	2	2.50	3.40
	D39	-5.0	4.79	8.2	8.05	10.0	3.20	2.20	2	2.70	3.70
	D43	-6.0	4.85	8.2	8.33	10.0	3.20	2.20	2	2.70	3.70
	D47	-7.0	4.90	8.2	8.58	10.0	3.20	2.20	2	2.70	3.70
	D50	-8.0	4.95	8.2	8.87	10.0	3.20	2.20	2	2.70	3.70
	D54	-9.0	5.00	8.2	9.16	10.0	3.20	2.20	2	2.70	3.70
	B7	-10.0	5.2	8.2	10.22	12.5	3.50	2.40	2	2.90	4.00

## 5. STABILITY OF TOE BERM IN FRONT OF VERTICAL IMPERMEABLE WALL STRUCTURE (According to "The Rock Manual, CIRIA 683")

### 1. Stability parameter due to waves (Madrigal and Valdés, 1995): Two Layers of Armour Stone

$$N_s = \frac{H_s}{\Delta D_{n50}} = \left( 5.8 \frac{h'}{h_m} - 0.6 \right) N_{od}^{0.19} \quad (5.190)$$

$$M_{n50} = \rho_r D_{n50}^3$$



**Figure 5.76** Definition sketch for stability tests described by Madrigal and Valdés (1995)

Where:

- $N_s$  - stability parameter
- $H_s$  - significant wave height,  $H_{1/3}$  of the incident waves at the toe of the structure (m)
- $\Delta$  - relative buoyant density,  $\rho_s/\rho_w - 1$
- $D_{n50}$  - equivalent cube length of median stone (m)
- $h'$  - depth at crest of rubble mound foundation (m)
- $h_m$  - water depth in front of toe berm (m)
- $N_{od}$  - number of units displaced out of the armor layer within a strip width of  $D_{n50}$
- $M_{n50}$  - medium mass of rocks

$\rho_s$ (t/m <sup>3</sup> )	$\rho_w$ (t/m <sup>3</sup> )	$\Delta$ (t/m <sup>3</sup> )	$N_{od}$	Damage Level	HWL (m)
2.65	1.025	1.59	0.50	almost no damage	3.55

Pos	Seabed Level (m)	$H_s$ (m)	$h'$ (m)	$h_m$ (m)	$N_s$	$D_{n50}$ (m)	$M_{n50}$ (kg)	
							Cal.	Selected
D10	0.0	2.67	3.05	3.55	3.84	<b>0.44</b>	222	<b>300</b>
D16	-1.0	3.08	3.05	4.55	2.88	<b>0.67</b>	815	800
D30	-2.0	3.58	4.05	5.55	3.18	<b>0.71</b>	944	1000
D33	-3.0	4.08	5.05	6.55	3.39	<b>0.76</b>	1155	1200
D36	-4.0	4.64	6.05	7.55	3.55	<b>0.82</b>	1483	1500
D39	-5.0	4.96	7.05	8.55	3.67	<b>0.85</b>	1647	1600
D43	-6.0	5.01	8.05	9.55	3.76	<b>0.84</b>	1574	1600
D47	-7.0	5.07	9.05	10.55	3.84	<b>0.83</b>	1533	1600
D50	-8.0	5.10	10.05	11.55	3.90	<b>0.82</b>	1486	1500
D54	-9.0	5.14	11.05	12.55	3.95	<b>0.82</b>	1467	1500
B7	-10.0	5.19	12.05	13.55	4.00	<b>0.82</b>	1458	1500

## 5. STABILITY OF TOE BERM IN FRONT OF VERTICAL IMPERMEABLE WALL STRUCTURE (According to "The Rock Manual, CIRIA 683")

### 2. Calculation of foot protection block (Shigeo TAKAHASHI)

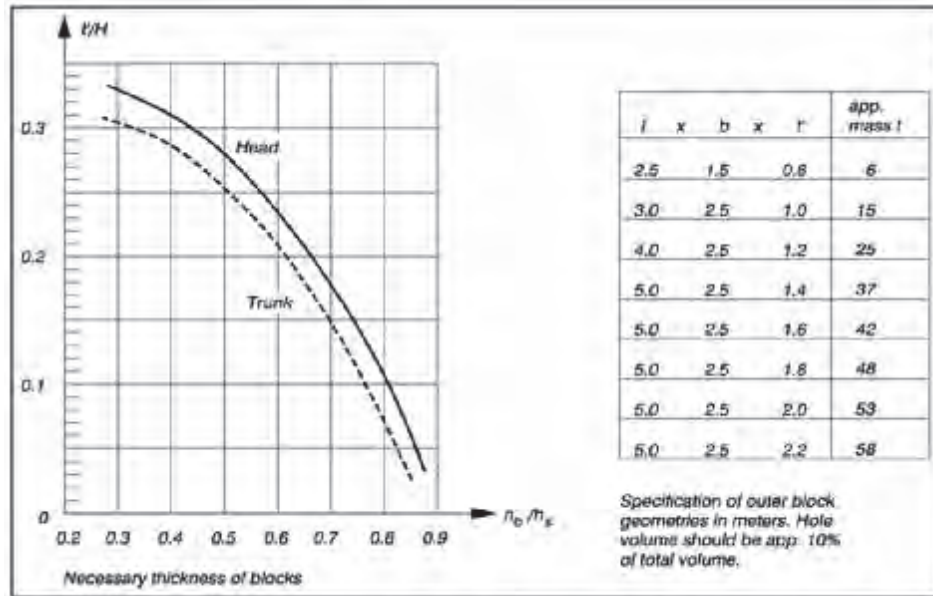


Figure VI-5-53. Design of foot protection blocks according to Japanese practice

Pos	Seabed Level (m)	Bot. Ver. Wall Level (m)	$h_s$ or $h$ (m)	$h_b$ or $d$ (m)	$d/h$	$t'/H_s$	$t'$ (m)	
							Cal.	Select
D16	-1.0	0.5	4.55	2.25	0.49	0.26	0.80	0.8
D30	-2.0	-0.5	5.55	3.25	0.59	0.22	0.79	0.8
D33	-3.0	-1.5	6.55	4.25	0.65	0.19	0.75	0.8
D36	-4.0	-2.5	7.55	5.25	0.70	0.15	0.70	0.8
D39	-5.0	-3.5	8.55	6.25	0.73	0.12	0.60	0.8
D43	-6.0	-4.5	9.55	7.25	0.76	0.11	0.55	0.8
D47	-7.0	-5.5	10.55	8.25	0.78	0.09	0.46	0.8
D50	-8.0	-6.5	11.55	9.25	0.80	0.07	0.36	0.8
D54	-9.0	-7.5	12.55	10.25	0.82	0.06	0.31	0.8
B7 (Head)	-10.0	-8.5	13.55	11.25	0.83	0.08	0.42	0.8

**6 STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - CASE A-1 (HWL)**

STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

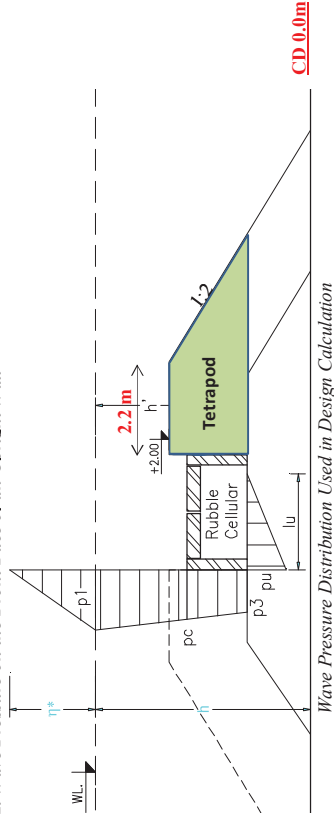
Sand Protection Training Dike - Section for GL upto CD 0.0m

I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	0.0
- Bottom Width of Upright Wall (m):	2.5
- Back Fill Level (CDL):	2.7
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	0.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	2.665
- H <sub>max</sub> , m:	3.821
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	77.14

2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

3. Uplift beneath Upright Wall

$P_3 = P_a = \alpha_3 P_1$

$$I_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	68.519
H <sub>D</sub> (m):	0.366
L <sub>w</sub> (m):	3.821
HWL (CDL):	77.14
Sea bed Level (CDL):	3.55
Crest Level (CDL):	0.0
h (m):	2.0
h' (m):	3.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
4 πh/L:	0.578
2 πh/L:	0.289
λ:	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	3.92
α <sub>1</sub> =	1.048
α <sub>3</sub> =	0.960
(1 + cos β)	1.366
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	4.104
P <sub>1</sub> (t/m <sup>2</sup> ) =	2.80
P <sub>a</sub> (at crest level), (t/m <sup>2</sup> ) =	2.75
P <sub>3</sub> (t/m <sup>2</sup> ) =	2.69

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
I <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	5.44
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	9.10
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Horizontal Effect (Passive Pressure)

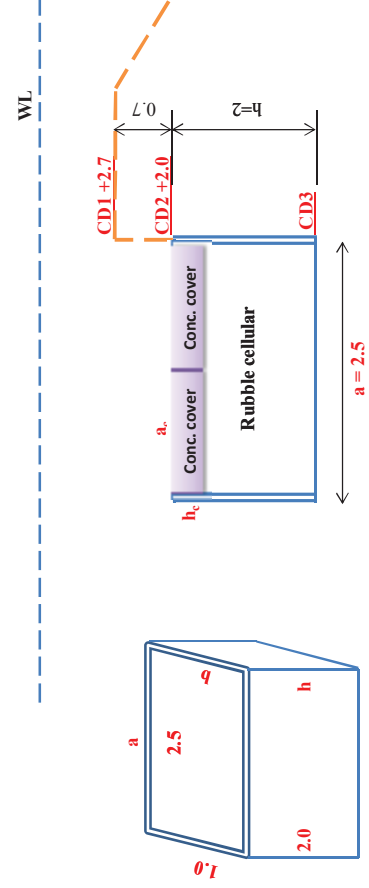
P <sub>p</sub> , t/m	9.43
M <sub>pp</sub> , t.m	6.29

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>con.</sub> (total), t/m	5.97
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	9.10

S.F. (Normal)	0.88
S.F. (with back fill layer effect)	2.05

Mu	0.00
Mp	5.47
Mtetrapod	6.29
Mconc. (=W*t)	7.47
S.F. (Normal)	1.37
S.F. (with back fill layer effect)	OK



CD1	30 years
CD2	+ 2.7 m
CD3	+ 2 m
HWL	0 m
a	+ 3.55 m
h	2.5 m
h <sub>c</sub>	2 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.00 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.24 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	3.41 t/m
W <sub>conc. cover</sub> (1 unit)	0.34 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.26 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	1.88 t/m
ΣW	5.97 t/m

Vertical Effect	
b <sub>back fill layer</sub>	2.2 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	13.23 m <sup>2</sup>
Witetrapod	9.10 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.0
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
K <sub>p</sub> = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	9.43
P <sub>p</sub> , t/m	9.43
γ <sub>p</sub> , m	0.67
M <sub>pp</sub> , t.m	6.29

Rubble Cell	x <sub>sc</sub>
	M (Sc)
	1.25 m
	7.47 t.m/m

P	5.44 t/m
U	0.00 t/m
γ <sub>p</sub>	1.004 m
x <sub>u</sub>	2.500 m
M <sub>p</sub>	5.47 t.m/m
M <sub>u</sub>	0.00 t.m/m
M total	5.47 t.m/m

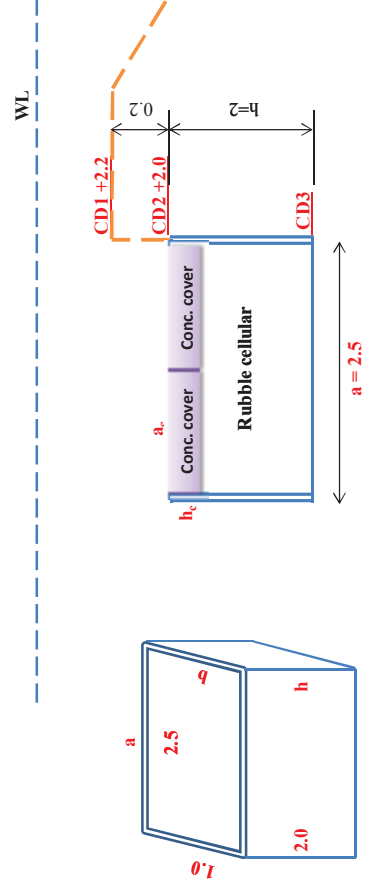
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -1.0m

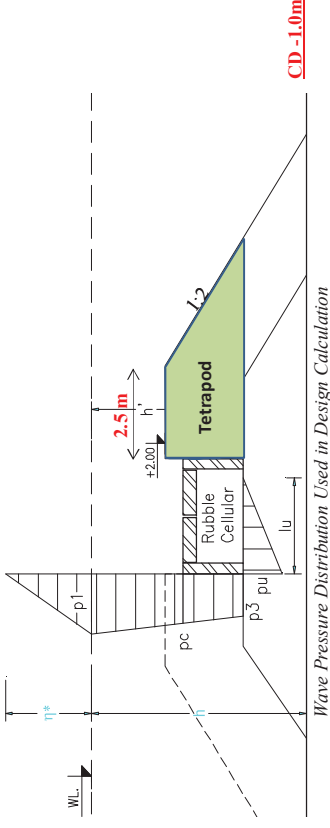
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	0.0
- Bottom Width of Upright Wall (m):	2.5
- Back Fill Level (CDL):	2.2
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-1.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	3.084
- H <sub>max</sub> , m:	4.234
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	81.06



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_4 = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

3. Uplift beneath Upright Wall

$P_5 = P_6 = \alpha_3 P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	68.349
H <sub>D</sub> (m):	0.369
L <sub>w</sub> (m):	4.234
HWL (CDL):	81.06
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-1.0
h (m):	2.0
h' (m):	4.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	3.55
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	0.705
λ:	0.353
	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	4.35
α <sub>1</sub> =	1.025
α <sub>3</sub> =	0.954
(1+cosβ)	1.369
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	4.447
P <sub>1</sub> (t/m <sup>2</sup> ) =	3.04
P <sub>5</sub> (at crest level), (t/m <sup>2</sup> ) =	2.98
P <sub>3</sub> (t/m <sup>2</sup> ) =	2.90

5.3. Uplift beneath Upright Wall

P <sub>5</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	5.89
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	7.11
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	9.43
M <sub>pp</sub> , t.m	6.29

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>con</sub> , (total), t/m	5.97
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	7.11

S.F. (Normal)	0.81
S.F. (with back fill layer effect)	1.66

Mu	0.00
Mp	5.91
Mtetrapod	6.29
Mconc. (=W*t)	7.47
S.F. (Normal)	1.26
S.F. (with back fill layer effect)	OK

CD1	30 years
CD2	+ 2.2 m
CD3	+ 2 m
HWL	0 m
a	+ 3.55 m
h	2.5 m
b	2 m
t	1 m
h <sub>c</sub>	0.20 m
a <sub>c</sub>	0.50 m
b <sub>c</sub>	1.00 m
S <sub>conc. cell</sub> (1 cell)	0.50 m
γ <sub>conc.</sub>	1.24 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc.</sub>	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	3.41 t/m
W <sub>conc. cover</sub> (1 unit)	0.34 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.26 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	1.88 t/m
ΣW	5.97 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	2.5 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	10.34 m <sup>2</sup>
Witetrapod	7.11 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.0
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45°+φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	9.43
Pp, t/m	9.43
γp, m	0.67
M <sub>pp</sub> , t.m	6.29

Rubble Cell	x <sub>sc</sub>
	M (Sc)
	1.25 m
	7.47 t.m/m

P	5.89 t/m
U	0.00 t/m
γ <sub>p</sub>	1.004 m
x <sub>u</sub>	2.500 m
Mp	5.91 t.m/m
Mu	0.00 t.m/m
M total	5.91 t.m/m

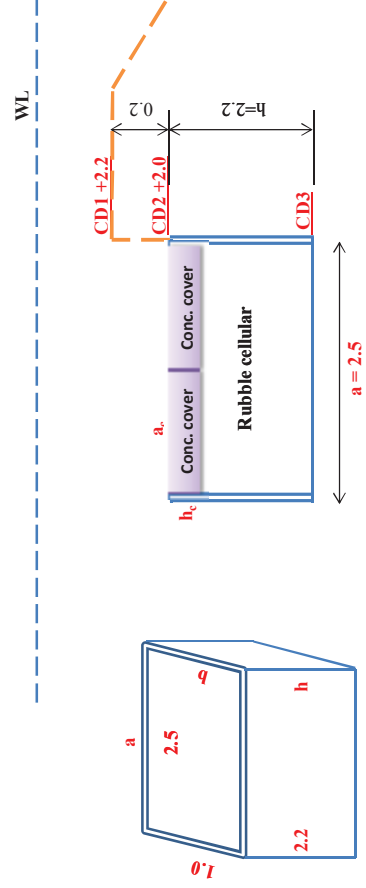
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -2.0m

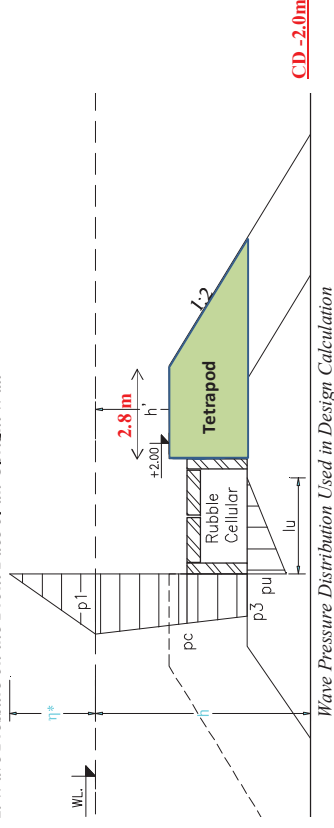
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-0.2
- Bottom Width of Upright Wall (m):	2.5
- Back Fill Level (CDL):	2.2
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-2.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	3.578
- H <sub>max</sub> , m:	4.762
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	85.78



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} (\text{if } -h' \leq 0)$$

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

3. Uplift beneath Upright Wall

$$P_3 = P_u = \alpha_3 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

breaking zone

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	63.333
H <sub>D</sub> (m):	0.449
L (m):	4.762
HWL (CDL):	85.78
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-2.0
h (m):	2.0
h' (m):	5.55
h (m):	3.75
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
4 πh/L:	0.813
2 πh/L:	0.407
λ:	0.999

5.2. Wave pressure on the front face of an Upright wall

η* =	5.17
α <sub>1</sub> =	1.003
α <sub>3</sub> =	0.948
(1 + cos β)	1.449
α <sub>3</sub> λ ρ <sub>0</sub> H <sub>D</sub>	4.893
P <sub>1</sub> (t/m <sup>2</sup> ) =	3.54
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	3.47
P <sub>3</sub> (t/m <sup>2</sup> ) =	3.36

5.3. Uplift beneath Upright Wall

P <sub>3</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	7.51
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	8.58
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	11.41
M <sub>pp</sub> , t.m	8.37

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	6.56
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	8.58

Sliding	S.F. (Normal)	0.70
	S.F. (with back fill layer effect)	1.50

	Mu	0.00
	Mp	8.30
Overturning	Mtetrapod	8.37
	Mconc. (=W*t)	8.20
	S.F. (Normal)	0.99
	S.F. (with back fill layer effect)	OK

	30 years
CD1	+2.2 m
CD2	+2 m
CD3	-0.2 m
HWL	+3.55 m
a	2.5 m
h	2.2 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.00 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.24 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	3.75 t/m
W <sub>conc. cover</sub> (1 unit)	0.34 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.26 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	2.13 t/m
ΣW	6.56 t/m

Vertical Effect	
b <sub>back fill layer</sub>	2.8 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	12.48 m <sup>2</sup>
Witetrapod	8.58 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.2
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	10.37
Pp, t/m	11.41
γp, m	0.73
Mpp, t.m	8.37

Rubble Cell	x <sub>sc</sub>	1.25 m
	M (Sc)	8.20 t.m/m

P	7.51 t/m
U	0.00 t/m
γ <sub>p</sub>	1.106 m
x <sub>u</sub>	2.500 m
Mp	8.30 t.m/m
Mu	0.00 t.m/m
M total	8.30 t.m/m

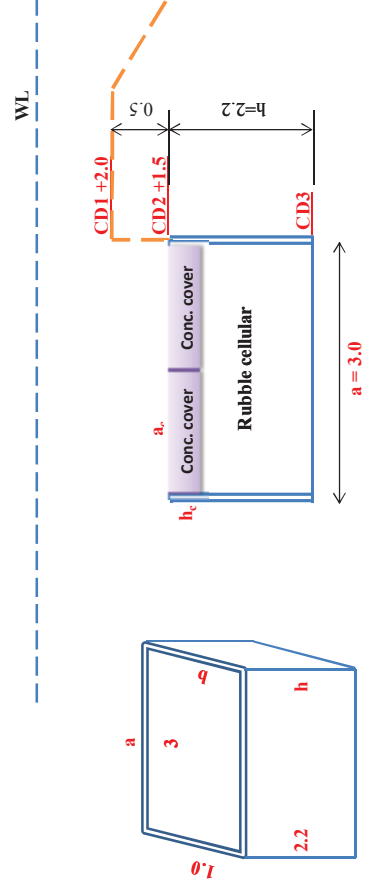
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -3.0m

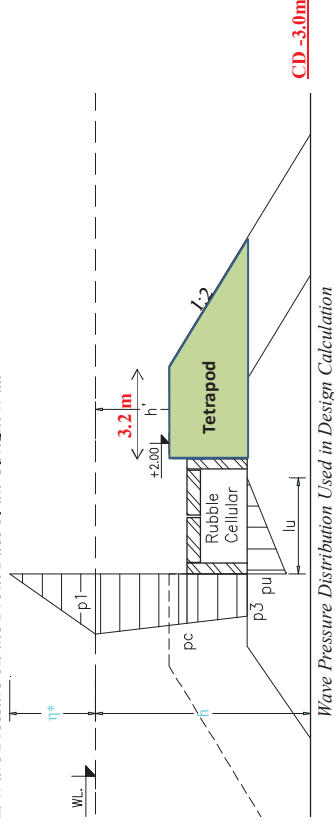
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	1.5
- Bottom Upright Wall Level (CDL):	-0.7
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	2.0
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-3.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.08
- H <sub>max</sub> , m:	5.365
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	90.82



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_4 = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (if \quad h' \leq 0)$$

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

3. Uplift beneath Upright Wall

$$P_5 = P_4 = \alpha_3 P_1$$

$$I_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.793
H <sub>D</sub> (m):	0.488
L <sub>w</sub> (m):	5.365
HWL (CDL):	90.82
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-3.0
h (m):	1.5
h' (m):	6.55
h (m):	4.25
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
4 πh/L:	0.906
2 πh/L:	0.453
λ:	0.999

5.2. Wave pressure on the front face of an Upright wall

η* =	5.98
α <sub>1</sub> =	0.983
α <sub>3</sub> =	0.939
(1 + cos β)	1.488
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	5.400
P <sub>1</sub> (t/m <sup>2</sup> ) =	4.02
P <sub>5</sub> (at crest level), (t/m <sup>2</sup> ) =	3.90
P <sub>3</sub> (t/m <sup>2</sup> ) =	3.77

5.3. Uplift beneath Upright Wall

P <sub>5</sub> (t/m <sup>2</sup> ) =	0.00
I <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	8.44
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	10.95
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	11.41
M <sub>pp</sub> , t.m	8.37

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	7.85
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	10.95

S.F. (Normal)	0.74
S.F. (with back fill layer effect)	1.65

Mu	0.00
Mp	9.33
Mtetrapod	8.37
Mconc. (=W*t)	11.77
S.F. (Normal)	1.26
S.F. (with back fill layer effect)	12.19

CD1	30 years
CD2	+2 m
CD3	+1.5 m
HWL	-0.7 m
a	+3.55 m
h	3 m
b	2.2 m
t	1 m
h <sub>c</sub>	0.20 m
a <sub>c</sub>	0.50 m
b <sub>c</sub>	1.25 m
S <sub>conc. cell</sub> (1 cell)	0.50 m
γ <sub>conc.</sub>	1.44 t/m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cover</sub> (1 unit.)	4.36 t/m
NOS. of cover block	0.43 t/m
S <sub>rubber filling</sub> (1 cell.)	2.00
γ <sub>rubber stone</sub>	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell.)	2.63 t/m
ΣW	7.85 t/m

Vertical Effect	
b <sub>back fill layer</sub>	3.2 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	15.93 m <sup>2</sup>
Wtetrapod	10.95 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.2
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	10.37
Pp, t/m	11.41
γp, m	0.73
M <sub>pp</sub> , t.m	8.37

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	11.77 t.m/m

P	8.44 t/m
U	0.00 t/m
γ <sub>p</sub>	1.106 m
x <sub>u</sub>	3.000 m
Mp	9.33 t.m/m
Mu	0.00 t.m/m
M total	9.33 t.m/m



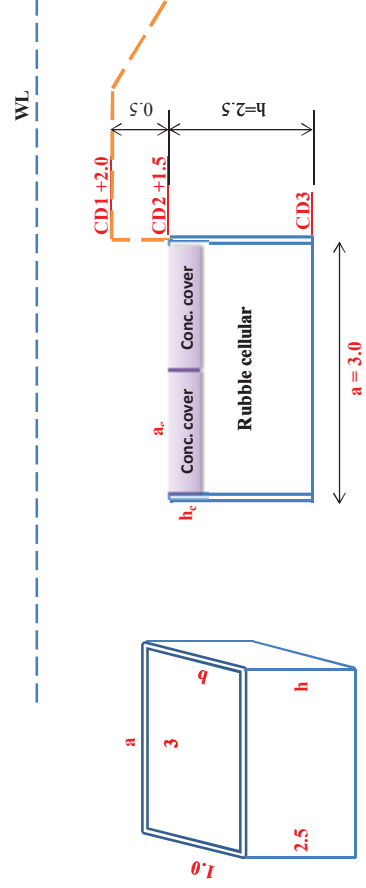
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -4.0m

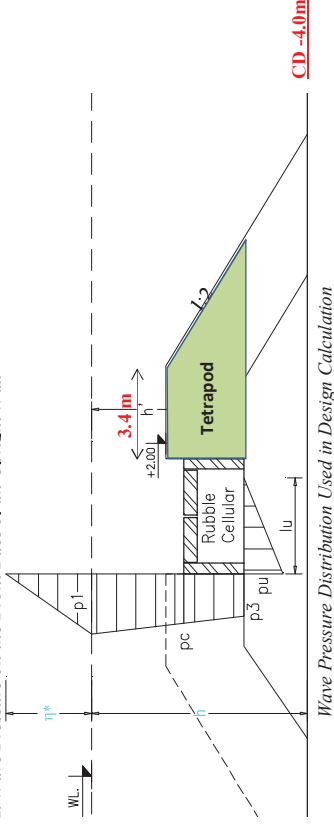
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	1.5
- Bottom Upright Wall Level (CDL):	-1.0
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	2.0
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-4.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.636
- H <sub>max</sub> , m:	6.035
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	96.05



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_4 = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (if \quad h' \leq 0)$$

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

3. Uplift beneath Upright Wall

$$P_5 = P_4 = \alpha_3 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

breaking zone

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.926
H <sub>D</sub> (m):	0.486
L <sub>w</sub> (m):	6.035
HWL (CDL):	96.05
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-4.0
h (m):	1.5
h' (m):	7.55
h (m):	4.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
4 πh/L:	0.988
2 πh/L:	0.494
λ:	0.998

5.2. Wave pressure on the front face of an Upright wall

η* =	6.71
α <sub>1</sub> =	0.965
α <sub>3</sub> =	0.933
(1 + cos β)	1.486
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	5.955
P <sub>1</sub> (t/m <sup>2</sup> ) =	4.42
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	4.29
P <sub>5</sub> (t/m <sup>2</sup> ) =	4.13

5.3. Uplift beneath Upright Wall

P <sub>5</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	10.53
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	13.20
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	14.73
M <sub>pp</sub> , t.m	12.28

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	8.91
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	13.20

Sliding	S.F. (Normal)	0.68
	S.F. (with back fill layer effect)	1.55
	Mu	0.00
	Mp	13.24
Overtopping	Mtetrapod	12.28
	Mconc. (=W*t)	13.36
	S.F. (Normal)	1.01
	S.F. (with back fill layer effect)	13.82

CD1	30 years
CD2	+2 m
CD3	+1.5 m
HWL	-1 m
a	+3.55 m
h	3 m
b	2.5 m
t	1 m
h <sub>c</sub>	0.20 m
a <sub>c</sub>	0.50 m
b <sub>c</sub>	1.25 m
S <sub>conc. cell</sub> (1 cell)	0.50 m
γ <sub>conc.</sub>	1.44 t/m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cover</sub> (1 unit)	4.95 t/m
NOS. of cover block	0.43 t/m
S <sub>rubber filling</sub> (1 cell)	2.00
γ <sub>rubber stone</sub>	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	3.10 t/m
ΣW	8.91 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	3.4 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	19.20 m <sup>2</sup>
Wtetrapod	13.20 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.5
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
γp, m	0.83
Mpp, t.m	12.28

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	13.36 t.m/m

P	10.53 t/m
U	0.00 t/m
γ <sub>p</sub>	1.258 m
x <sub>u</sub>	3.000 m
Mp	13.24 t.m/m
Mu	0.00 t.m/m
M total	13.24 t.m/m

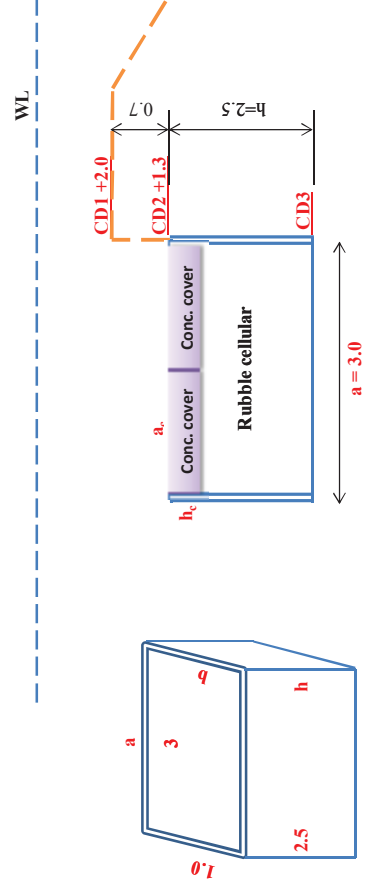
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -5.0m

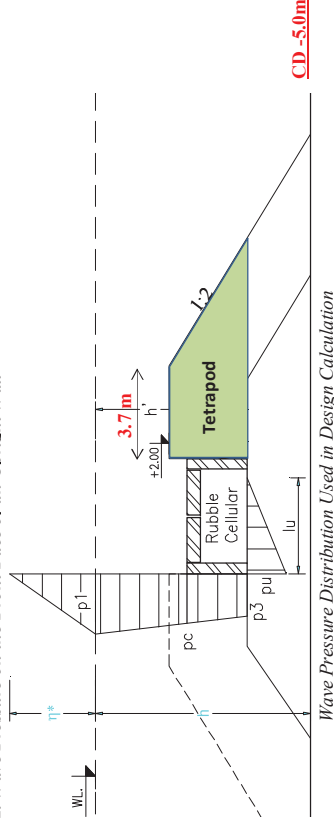
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	1.3
- Bottom Upright Wall Level (CDL):	-1.2
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	2.0
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-5.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.96
- H <sub>max</sub> , m:	6.615
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	100.31



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

3. Uplift beneath Upright Wall

$P_3 = P_a = \alpha_3 P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.947
H <sub>D</sub> (m):	0.486
L (m):	6.615
HWL (CDL):	100.31
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-5.0
h (m):	1.3
h' (m):	8.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	4.75
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.071
λ:	0.536
	0.996

5.2. Wave pressure on the front face of an Upright wall

η* =	7.34
α <sub>1</sub> =	0.946
α <sub>3</sub> =	0.929
(1 + cos β)	1.486
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	6.385
P <sub>1</sub> (t/m <sup>2</sup> ) =	4.74
P <sub>a</sub> (at crest level), (t/m <sup>2</sup> ) =	4.58
P <sub>3</sub> (t/m <sup>2</sup> ) =	4.41

5.3. Uplift beneath Upright Wall

P <sub>a</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	11.24
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	15.18
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	14.73
M <sub>pp</sub> , t.m	12.28

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	8.91
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	15.18

Sliding	S.F. (Normal)	0.63
	S.F. (with back fill layer effect)	1.58

	Mu	0.00
	Mp	14.14
Overturning	Mtetrapod	12.28
	Mconc. (=W*t)	13.36
	S.F. (Normal)	0.94
	S.F. (with back fill layer effect)	7.17

	30 years
CD1	+2 m
CD2	+1.3 m
CD3	-1.2 m
HWL	+3.55 m
a	3 m
h	2.5 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.44 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	4.95 t/m
W <sub>conc. cover</sub> (1 unit)	0.43 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	3.10 t/m
ΣW	8.91 t/m

Vertical Effect	
b <sub>back fill layer</sub>	3.7 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	22.08 m <sup>2</sup>
Wtetrapod	15.18 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.5
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
γp, m	0.83
M <sub>pp</sub> , t.m	12.28

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	13.36 t.m/m

P	11.24 t/m
U	0.00 t/m
γ <sub>n</sub>	1.258 m
x <sub>n</sub>	3.000 m
Mp	14.14 t.m/m
Mu	0.00 t.m/m
M total	14.14 t.m/m

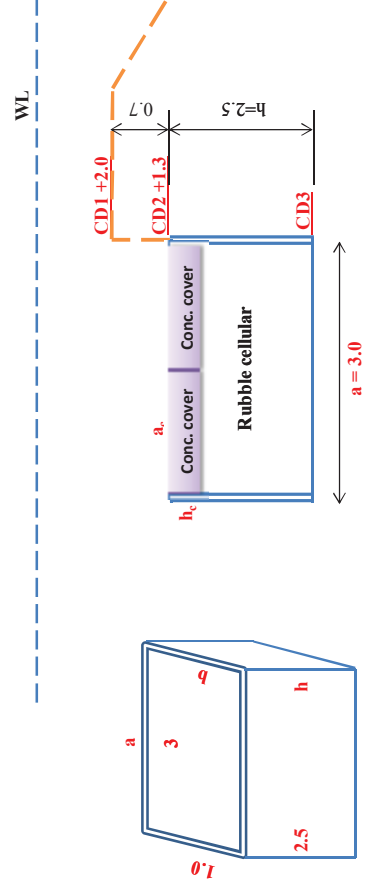
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -6.0m

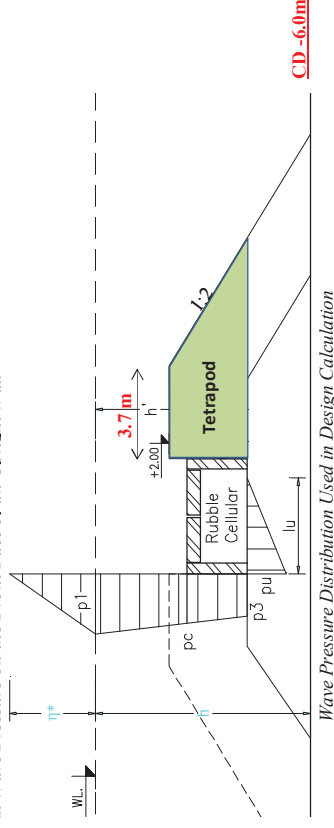
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	1.3
- Bottom Upright Wall Level (CDL):	-1.2
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	2.0
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-6.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	5.01
- H <sub>max</sub> , m:	7.308
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	105.13



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

3. Uplift beneath Upright Wall

$P_3 = P_a = \alpha_3 P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

$H_D = H_{max}$

breaking zone

5. Results of Calculation

5.1. Input data

$\beta$ (deg) =	30 years
$\cos \beta$ :	60.77
H <sub>D</sub> (m):	0.488
L <sub>w</sub> (m):	7.308
HLWL (CDL):	105.13
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-6.0
h (m):	1.3
h' (m):	9.55
$\rho_0$ (t/m <sup>3</sup> ):	4.75
g (m/s <sup>2</sup> ):	1.025
4 $\pi h/L$ :	9.81
2 $\pi h/L$ :	1.142
$\lambda$ :	0.571
	0.991

5.2. Wave pressure on the front face of an Upright wall

$\eta^* =$	8.09
$\alpha_1 =$	0.930
$\alpha_3 =$	0.929
(1 + cos $\beta$ )	1.488
$\alpha_1 \lambda \rho_0 H_D$	6.902
P <sub>1</sub> (t/m <sup>2</sup> ) =	5.14
P <sub>a</sub> (at crest level), (t/m <sup>2</sup> ) =	4.96
P <sub>3</sub> (t/m <sup>2</sup> ) =	4.77

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	12.17
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

$\Sigma W$ (t) =	15.18
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	14.73
M <sub>pp</sub> , t.m	12.28

III. Safety Factor

$\mu$ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	8.91
$\mu$ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	15.18

Sliding	S.F. (Normal)	0.59
	S.F. (with back fill layer effect)	1.46

	Mu	0.00
	Mp	15.31
Overturning	Mtetrapod	12.28
	Mconc. (=W*t)	13.36
	S.F. (Normal)	0.87
	S.F. (with back fill layer effect)	4.41

	30 years
CD1	+2 m
CD2	+1.3 m
CD3	-1.2 m
HWL	+3.55 m
a	3 m
h	2.5 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.44 m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
$\gamma_{conc. cell}$	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	4.95 t/m
W <sub>conc. cover</sub> (1 unit)	0.43 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.56 m <sup>2</sup>
$\gamma_{rubber stone}$	2.6 t/m <sup>3</sup>
$\gamma_{rubber stone}$	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	3.10 t/m
$\Sigma W$	8.91 t/m

Vertical Effect	
b <sub>back fill layer</sub>	3.7 m
Slope	2
$\gamma_{tetrapod}$	2.4 t/m <sup>3</sup>
$\gamma_{tetrapod}$	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	22.08 m <sup>2</sup>
Wtetrapod	15.18 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.5
$\phi$ (deg)	40.0
$\gamma_s$ (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + $\phi/2$ )	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
Wp, m	0.83
Mpp, t.m	12.28

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	13.36 t.m/m

P	12.17 t/m
U	0.00 t/m
$\gamma_h$	1.258 m
x <sub>u</sub>	3.000 m
Mp	15.31 t.m/m
Mu	0.00 t.m/m
M total	15.31 t.m/m

STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)

CASE A-1 Phu kin Tetrapod qua mai

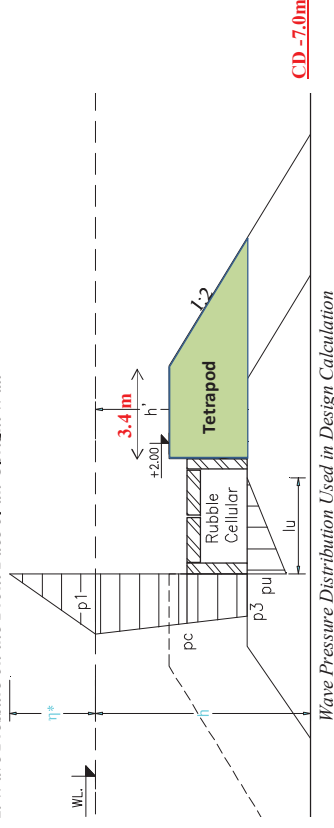
Sand Protection Training Dike - Section for GL upto CD -7.0m

I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-1.5
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	2.8
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-7.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	5.067
- H <sub>max</sub> , m:	7.931
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	109.23

2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_s \rho_w g H_D$

At toe of upright wall:  $P_3 = P_s = \alpha_s P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

$$\alpha_s = 0.6 + \frac{1}{2} \left[ \frac{4\pi h / L}{\sinh \left( \frac{4\pi h}{L} \right)} \right]^2$$

3. Uplift beneath Upright Wall

$P_5 = P_u = \alpha_s P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

$H_D = H_{max}$

breaking zone

5. Results of Calculation

5.1. Input data

$\beta$ (deg) =	30 years
$\cos \beta$ :	60.21
H <sub>D</sub> (m):	0.497
L <sub>w</sub> (m):	7.931
HWL (CDL):	109.23
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-7.0
h (m):	2.0
h' (m):	10.55
$\rho_0$ (t/m <sup>3</sup> ):	5.05
g (m/s <sup>2</sup> ):	1.025
4 $\pi h / L$ :	9.81
2 $\pi h / L$ :	1.214
$\lambda$ :	0.607
	0.989

5.2. Wave pressure on the front face of an Upright wall

$\eta^* =$	8.80
$\alpha_1 =$	0.913
$\alpha_5 =$	0.924
(1 + cos $\beta$ )	1.497
$\alpha_s \lambda \rho_w H_D$	7.335
P <sub>1</sub> (t/m <sup>2</sup> ) =	5.49
P <sub>5</sub> (at crest level), (t/m <sup>2</sup> ) =	5.36
P <sub>3</sub> (t/m <sup>2</sup> ) =	5.07

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	18.26
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

$\Sigma W$ (t) =	22.76
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Horizontal Effect (Passive Pressure)

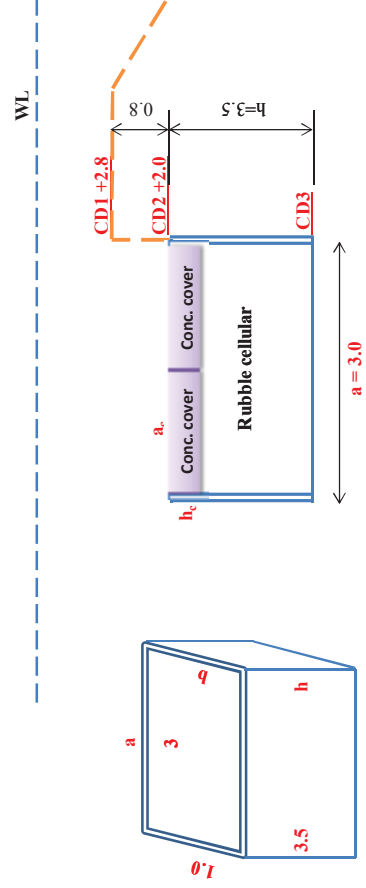
P <sub>p</sub> , t/m	28.87
M <sub>pp</sub> , t.m	33.68

III. Safety Factor

$\mu$ (friction factor between rubble cellular & stone):	0.80
W <sub>con.</sub> (total), t/m	12.43
$\mu$ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	22.76

Sliding	S.F. (Normal)	0.54
	S.F. (with back fill layer effect)	1.42

	Mu	0.00
	Mp	32.24
Overtopping	Mtetrapod	33.68
	Mconc. (=W*t)	18.65
	S.F. (Normal)	0.58
	S.F. (with back fill layer effect)	OK



CD1	30 years
CD2	+2.8 m
CD3	+2 m
HWL	-1.5 m
a	+3.55 m
h	3 m
b	3.5 m
t	1 m
h <sub>c</sub>	0.20 m
a <sub>c</sub>	0.50 m
b <sub>c</sub>	1.25 m
S <sub>conc. cell</sub> (1 cell)	0.50 m
$\gamma_{conc.}$	1.44 t/m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
$\gamma_{conc.}$	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	6.93 t/m
W <sub>conc. cover</sub> (1 unit)	0.43 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.56 m <sup>2</sup>
$\gamma_{rubber stone}$	2.6 t/m <sup>3</sup>
$\gamma_{rubber stone}$	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	4.64 t/m
$\Sigma W$	12.43 t/m

Vertical Effect	
b <sub>back fill layer</sub>	3.4 m
Slope	2
$\gamma_{Tetrapod}$	2.4 t/m <sup>3</sup>
$\gamma_{Tetrapod}$	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	33.11 m <sup>2</sup>
Witetrapod	22.76 t/m
Horizontal Effect (Passive Pressure)	
H (m)	3.5
$\phi$ (deg)	40.0
$\gamma_s$ (t/m <sup>3</sup> )	1.03
K <sub>p</sub> = tan <sup>2</sup> (45° + $\phi$ /2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	16.50
Pp, t/m	28.87
γp, m	1.17
M <sub>pp</sub> , t.m	33.68

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	18.65 t.m/m

P	18.26 t/m
U	0.00 t/m
γ <sub>p</sub>	1.766 m
x <sub>u</sub>	3.000 m
M <sub>p</sub>	32.24 t.m/m
Mu	0.00 t.m/m
M total	32.24 t.m/m

**7 STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - CASE A-2 (HWL)**

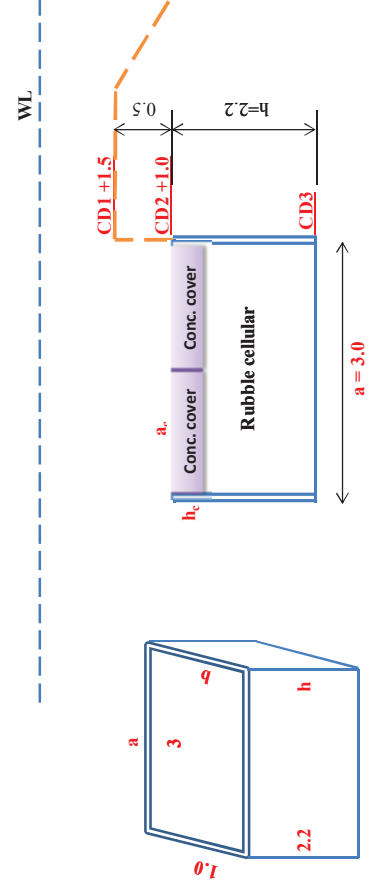
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -3.0m

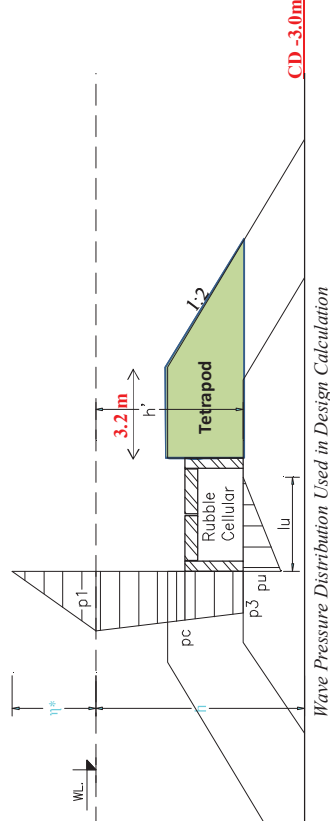
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	1.0
- Bottom Upright Wall Level (CDL):	-1.2
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	1.5
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-3.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.08
- H <sub>max</sub> , m:	5.365
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	90.82



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_w g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

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

3. Uplift beneath Upright Wall

$$P_3 = P_u = \alpha_3 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.793
H <sub>D</sub> (m):	0.488
L <sub>w</sub> (m):	5.365
HWL (CDL):	90.82
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-3.0
h (m):	1.0
h' (m):	6.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	4.75
g (m/s <sup>2</sup> ):	9.81
4 πh/L:	0.906
2 πh/L:	0.453
λ:	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	5.99
α <sub>1</sub> =	0.983
α <sub>3</sub> =	0.931
(1 + cos β)	1.488
α <sub>1</sub> λ ρ <sub>w</sub> H <sub>D</sub>	5.404
P <sub>1</sub> (t/m <sup>2</sup> ) =	4.02
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	3.87
P <sub>3</sub> (t/m <sup>2</sup> ) =	3.74

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	8.38
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	10.95
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	11.41
M <sub>pp</sub> , t.m	8.37

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	7.85
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	10.95

Sliding	S.F. (Normal)	0.75
	S.F. (with back fill layer effect)	1.66

	Mu	0.00
	Mp	9.27
Overturning	Mtetrapod	8.37
	Mconc. (=W*t)	11.77
	S.F. (Normal)	1.27
	S.F. (with back fill layer effect)	13.04

	30 years
CD1	+ 1.5 m
CD2	+ 1 m
CD3	-1.20 m
HWL	+ 3.55 m
a	3 m
h	2.2 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.44 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	4.36 t/m
W <sub>conc. cover</sub> (1 unit)	0.43 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	2.63 t/m
ΣW	7.85 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	3.2 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	15.93 m <sup>2</sup>
Wtetrapod	10.95 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.2
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	10.37
Pp, t/m	11.41
γp, m	0.73
M <sub>pp</sub> , t.m	8.37

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	11.77 t.m/m

P	8.38 t/m
U	0.00 t/m
γ <sub>h</sub>	1.106 m
x <sub>u</sub>	3.000 m
Mp	9.27 t.m/m
Mu	0.00 t.m/m
M total	9.27 t.m/m

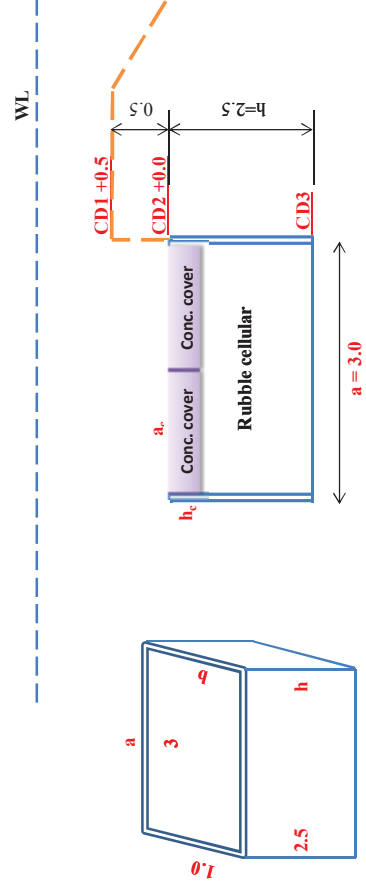
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -4.0m

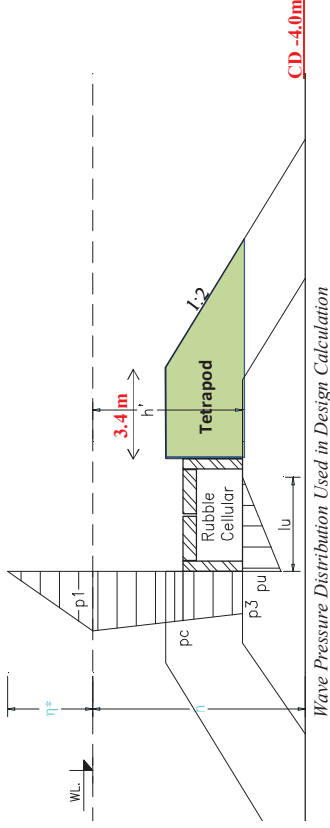
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	0.0
- Bottom Upright Wall Level (CDL):	-2.5
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	0.5
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-4.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.636
- H <sub>max</sub> , m:	6.035
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	96.05



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

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

3. Uplift beneath Upright Wall

$$P_3 = P_u = \alpha_3 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.926
H <sub>D</sub> (m):	0.486
L (m):	6.035
HWL (CDL):	96.05
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-4.0
h (m):	0.0
h' (m):	7.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	6.05
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	0.988
λ:	0.494
	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	6.73
α <sub>1</sub> =	0.965
α <sub>3</sub> =	0.911
(1+cosβ)	1.486
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	5.968
P <sub>1</sub> (t/m <sup>2</sup> ) =	4.43
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	4.20
P <sub>3</sub> (t/m <sup>2</sup> ) =	4.04

5.3. Uplift beneath Upright Wall

P <sub>3</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	10.30
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	13.20
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	14.73
M <sub>pp</sub> , t.m	12.28

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	8.91
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	13.20

S.F. (Normal)	0.69
S.F. (with back fill layer effect)	1.59

Mu	0.00
Mp	12.97
Mtetrapod	12.28
Mconc. (=W*t)	13.36
S.F. (Normal)	1.03
S.F. (with back fill layer effect)	19.37

CD1	30 years
CD2	+ 0.5 m
CD3	+ 0 m
HWL	-2.5 m
a	+ 3.55 m
h	3 m
b	2.5 m
t	1 m
h <sub>c</sub>	0.20 m
a <sub>c</sub>	0.50 m
b <sub>c</sub>	1.25 m
S <sub>conc. cell</sub> (1 cell)	0.50 m
γ <sub>conc.</sub>	1.44 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cover</sub> (1 unit.)	4.95 t/m
NOS. of cover block	0.43 t/m
S <sub>rubber filling</sub> (1 cell.)	2.00
γ <sub>rubber stone</sub>	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell.)	3.10 t/m
ΣW	8.91 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	3.4 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Wtetrapod	19.20 m <sup>2</sup>
Wtetrapod	13.20 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.5
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45°+φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
γp, m	0.83
Mpp, t.m	12.28

Back fill beside conc. Block

x <sub>sc</sub>	1.50 m
M (Sc)	13.36 t.m/m

P	10.30 t/m
U	0.00 t/m
γ <sub>h</sub>	1.258 m
x <sub>u</sub>	3.000 m
Mp	12.97 t.m/m
Mu	0.00 t.m/m
M total	12.97 t.m/m

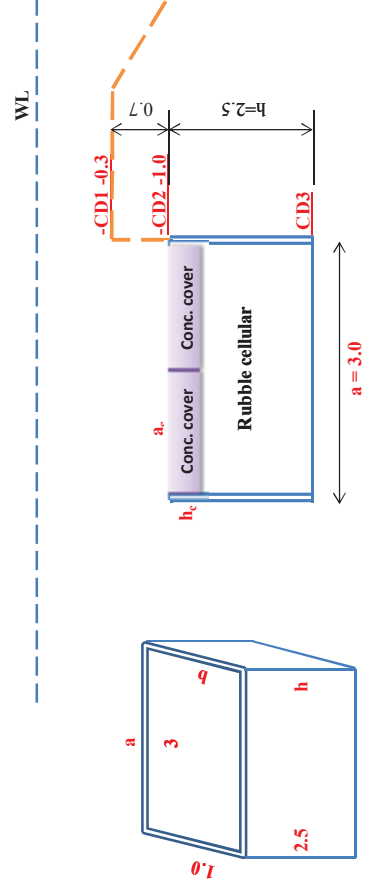
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)

Sand Protection Training Dike - Section for GL upto CD -5.0m

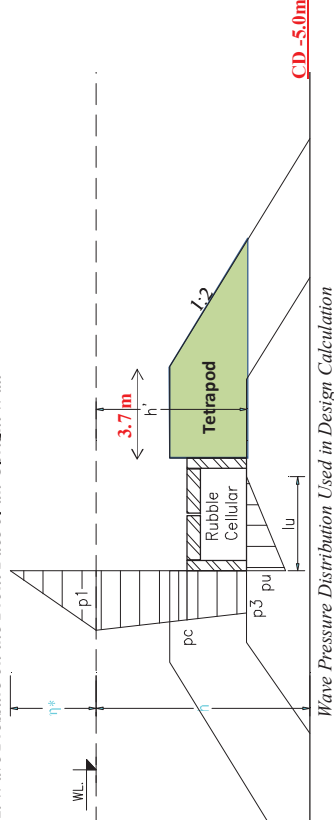
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-1.0
- Bottom Upright Wall Level (CDL):	-3.5
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	-0.3
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-5.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.96
- H <sub>max</sub> , m:	6.615
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	100.31



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (if \quad h' \leq 0)$$

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

3. Uplift beneath Upright Wall

$$P_3 = P_u = \alpha_3 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

breaking zone

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.947
H <sub>D</sub> (m):	0.486
L <sub>w</sub> (m):	6.615
HWL (CDL):	100.31
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-5.0
h (m):	-1.0
h' (m):	8.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	7.05
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.071
λ:	0.536
	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	7.37
α <sub>1</sub> =	0.946
α <sub>3</sub> =	0.894
(1 + cos β)	1.486
α <sub>3</sub> λ ρ <sub>0</sub> H <sub>D</sub>	6.413
P <sub>1</sub> (t/m <sup>2</sup> ) =	4.76
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	4.44
	4.26

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	10.87
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	15.18
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	14.73
M <sub>pp</sub> , t.m	12.28

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	8.91
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	15.18

S. F. (Normal)	0.66
S. F. (with back fill layer effect)	1.63

Mu	0.00
Mp	13.69
Mtetrapod	12.28
Mconc. (=W*t)	13.36
S. F. (Normal)	0.98
S. F. (with back fill layer effect)	9.48

	30 years
CD1	-0.3 m
CD2	-1 m
CD3	-3.5 m
HWL	+ 3.55 m
a	3 m
h	2.5 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.44 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	4.95 t/m
W <sub>conc. cover</sub> (1 unit)	0.43 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	3.10 t/m
ΣW	8.91 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	3.7 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	22.08 m <sup>2</sup>
Witetrapod	15.18 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.5
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
γp, m	0.83
M <sub>pp</sub> , t.m	12.28

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	13.36 t.m/m

P	10.87 t/m
U	0.00 t/m
γ <sub>p</sub>	1.259 m
x <sub>u</sub>	3.000 m
Mp	13.69 t.m/m
Mu	0.00 t.m/m
M total	13.69 t.m/m



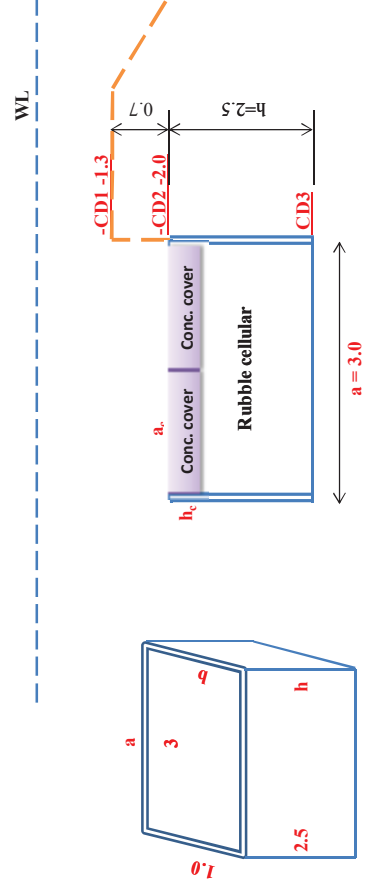
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -6.0m

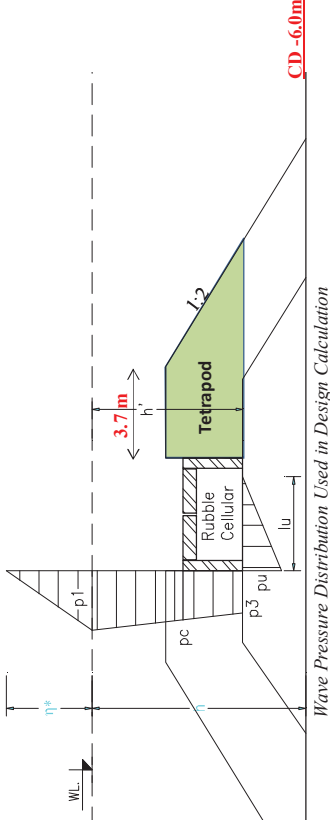
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-2.0
- Bottom Upright Wall Level (CDL):	-4.5
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	-1.3
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-6.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	5.01
- H <sub>max</sub> , m:	7.308
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	105.13



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (if \quad h' \leq 0)$$

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

3. Uplift beneath Upright Wall

$$P_3 = P_u = \alpha_3 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

breaking zone

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.77
H <sub>D</sub> (m):	0.488
L (m):	7.308
HWL (CDL):	105.13
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-6.0
h (m):	-2.0
h' (m):	9.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	8.05
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.142
λ:	0.571
	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	8.16
α <sub>1</sub> =	0.930
α <sub>3</sub> =	0.879
(1+cosβ)	1.488
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	6.963
P <sub>1</sub> (t/m <sup>2</sup> ) =	5.18
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	4.75
P <sub>3</sub> (t/m <sup>2</sup> ) =	4.56

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	11.63
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	15.18
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	14.73
M <sub>pp</sub> , t.m	12.28

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>con</sub> , (total), t/m	8.91
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	15.18

Sliding	S.F. (Normal)	0.61
	S.F. (with back fill layer effect)	1.53

	Mu	0.00
	Mp	14.64
Overtopping	Mtetrapod	12.28
	Mconc. (=W*t)	13.36
	S.F. (Normal)	0.91
	S.F. (with back fill layer effect)	5.65

	30 years
CD1	-1.3 m
CD2	-2 m
CD3	-4.5 m
HWL	+ 3.55 m
a	3 m
h	2.5 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.44 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	4.95 t/m
W <sub>conc. cover</sub> (1 unit)	0.43 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	3.10 t/m
ΣW	8.91 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	3.7 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	22.08 m <sup>2</sup>
Wtetrapod	15.18 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.5
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45°+φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
γp, m	0.83
M <sub>pp</sub> , t.m	12.28

Back fill beside conc. Block

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	13.36 t.m/m

P	11.63 t/m
U	0.00 t/m
γ <sub>h</sub>	1.259 m
x <sub>u</sub>	3.000 m
Mp	14.64 t.m/m
Mu	0.00 t.m/m
M total	14.64 t.m/m

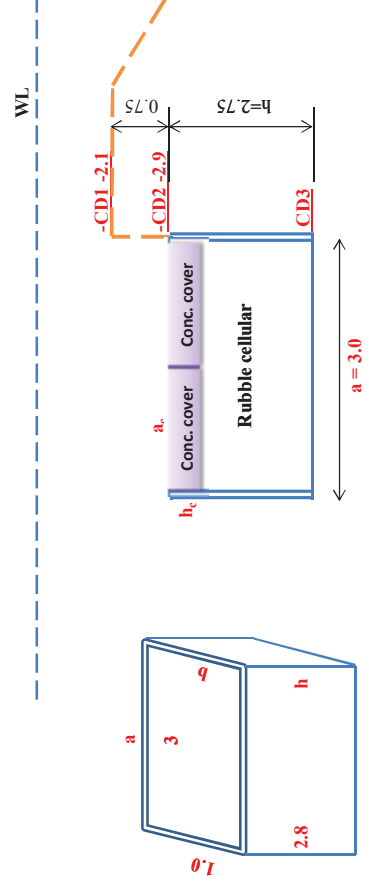
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)

Sand Protection Training Dike - Section for GL upto CD -7.0m

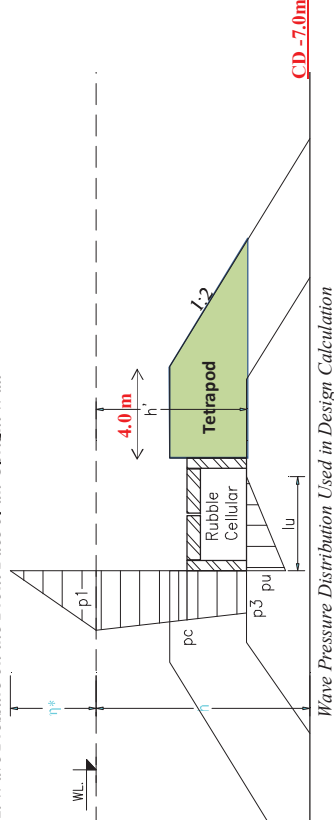
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-2.9
- Bottom Upright Wall Level (CDL):	-5.6
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	-2.1
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-7.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	5.067
- H <sub>max</sub> , m:	7.931
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	109.23



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

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

3. Uplift beneath Upright Wall

$$P_3 = P_u = \alpha_3 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

breaking zone

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.21
H <sub>D</sub> (m):	0.497
L <sub>w</sub> (m):	7.931
HWL (CDL):	109.23
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-7.0
h (m):	-2.9
h' (m):	10.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	9.15
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.214
λ:	0.607
	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	8.90
α <sub>1</sub> =	0.913
α <sub>3</sub> =	0.862
(1 + cos β)	1.497
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	7.420
P <sub>1</sub> (t/m <sup>2</sup> ) =	5.55
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	5.02
P <sub>3</sub> (t/m <sup>2</sup> ) =	4.78

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	13.48
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	18.05
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	17.82
M <sub>pp</sub> , t.m	16.34

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>con</sub> , (total), t/m	9.79
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	18.05

Sliding	S.F. (Normal)	0.58
	S.F. (with back fill layer effect)	1.52
	Mu	0.00
	Mp	18.68
Overturning	Mtetrapod	16.34
	Mconc. (=W*t)	14.68
	S.F. (Normal)	0.79
	S.F. (with back fill layer effect)	6.28

	30 years
CD1	-2.1 m
CD2	-2.85 m
CD3	-5.6 m
HWL	+3.55 m
a	3 m
h	2.75 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.44 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	5.45 t/m
W <sub>conc. cover</sub> (1 unit)	0.43 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	3.48 t/m
ΣW	9.79 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	4 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	26.25 m <sup>2</sup>
Witetrapod	18.05 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.8
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	12.96
Pp, t/m	17.82
γp, m	0.92
M <sub>pp</sub> , t.m	16.34

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	14.68 t.m/m

P	13.48 t/m
U	0.00 t/m
γ <sub>p</sub>	1.386 m
x <sub>u</sub>	3.000 m
Mp	18.68 t.m/m
Mu	0.00 t.m/m
M total	18.68 t.m/m

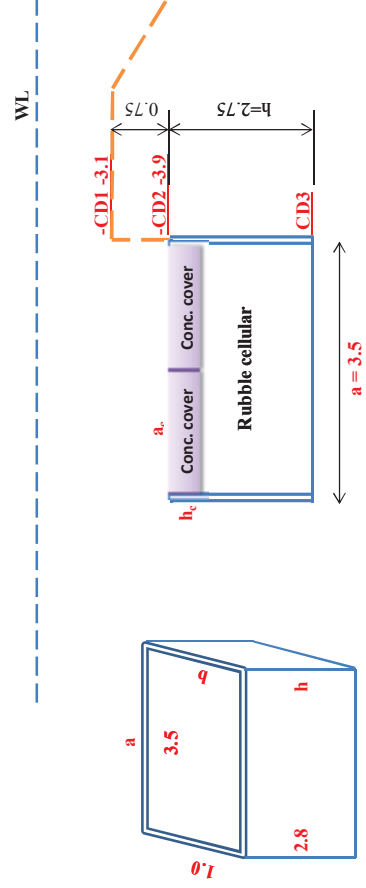
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)

Sand Protection Training Dike - Section for GL upto CD -8.0m

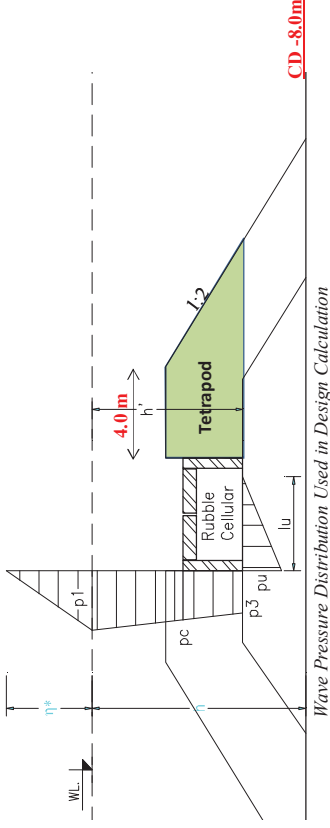
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-3.9
- Bottom Upright Wall Level (CDL):	-6.6
- Bottom Width of Upright Wall (m):	3.5
- Back Fill Level (CDL):	-3.1
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-8.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	5.096
- H <sub>max</sub> , m:	8.44
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	112.43



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

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

3. Uplift beneath Upright Wall

$P_3 = P_u = \alpha_3 P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

breaking zone

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.17
H <sub>D</sub> (m):	0.497
L (m):	8.44
HWL (CDL):	112.43
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-8.0
h (m):	-3.9
h' (m):	11.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	10.15
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.291
λ:	0.645
	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	9.48
α <sub>1</sub> =	0.895
α <sub>3</sub> =	0.844
(1+cosβ)	1.497
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	7.743
P <sub>1</sub> (t/m <sup>2</sup> ) =	5.80
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	5.14
P <sub>3</sub> (t/m <sup>2</sup> ) =	4.89

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	13.79
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	18.05
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	17.82
M <sub>pp</sub> , t.m	16.34

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>con.</sub> (total), t/m	11.39
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	18.05

S. F. (Normal)	0.66
S. F. (with back fill layer effect)	1.58

Mu	0.00
Mp	19.12
Mtetrapod	16.34
Mconc. (=W*t)	19.92
S. F. (Normal)	1.04
S. F. (with back fill layer effect)	7.16

CD1	30 years
CD2	-3.1 m
CD3	-3.85 m
HWL	-6.6 m
a	+3.55 m
h	3.5 m
b	2.75 m
t	1 m
h <sub>c</sub>	0.20 m
a <sub>c</sub>	0.50 m
b <sub>c</sub>	1.50 m
S <sub>conc. cell</sub> (1 cell)	0.50 m
γ <sub>conc.</sub>	1.64 t/m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	6.20 t/m
W <sub>conc. cover</sub> (1 unit)	0.52 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.86 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	4.15 t/m
ΣW	11.39 t/m

Vertical Effect	
b <sub>back fill layer</sub>	4 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
W <sub>Tetrapod</sub>	26.25 m <sup>2</sup>
W <sub>Tetrapod</sub>	18.05 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.8
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45°+φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	12.96
Pp, t/m	17.82
γp, m	0.92
M <sub>pp</sub> , t.m	16.34

x <sub>sc</sub>	1.75 m
M (Sc)	19.92 t.m/m

P	13.79 t/m
U	0.00 t/m
γ <sub>h</sub>	1.386 m
x <sub>u</sub>	3.500 m
Mp	19.12 t.m/m
Mu	0.00 t.m/m
M total	19.12 t.m/m

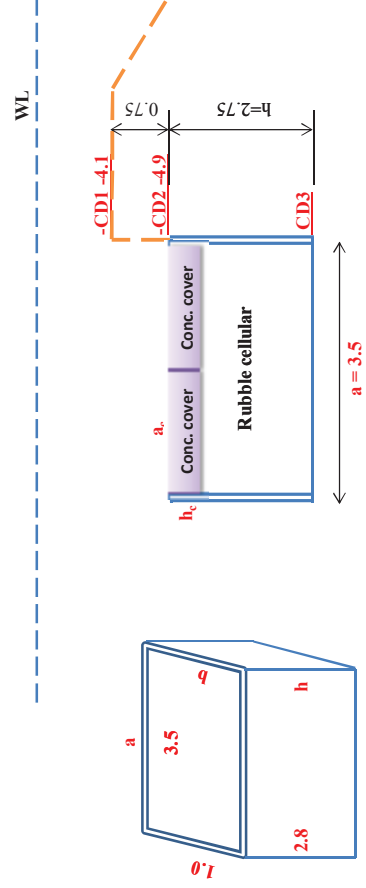
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -9.0m

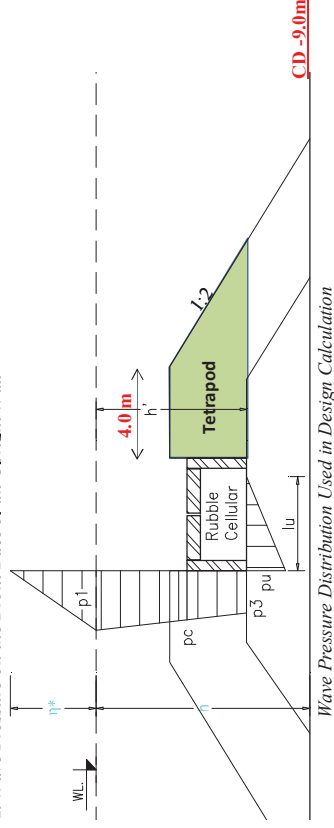
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-4.9
- Bottom Upright Wall Level (CDL):	-7.6
- Bottom Width of Upright Wall (m):	3.5
- Back Fill Level (CDL):	-4.1
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-9.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	5.143
- H <sub>max</sub> , m:	8.522
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	112.94



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_w g H_0$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

3. Uplift beneath Upright Wall

$P_3 = P_u = \alpha_3 P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>0</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	59.97
H <sub>0</sub> (m):	0.500
L <sub>w</sub> (m):	8.522
HWL (CDL):	112.94
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-9.0
h (m):	-4.9
h' (m):	12.55
h (m):	11.15
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
4 πh/L:	1.396
2 πh/L:	0.698
λ:	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	9.59
α <sub>1</sub> =	0.871
α <sub>3</sub> =	0.820
(1 + cos β)	1.500
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>0</sub>	7.609
P <sub>1</sub> (t/m <sup>2</sup> ) =	5.71
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	4.93
P <sub>3</sub> (t/m <sup>2</sup> ) =	4.68

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	13.22
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	18.05
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	17.82
M <sub>pp</sub> , t.m	16.34

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>con.</sub> (total), t/m	11.39
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	18.05

Sliding	S.F. (Normal)	0.69
	S.F. (with back fill layer effect)	1.64

	Mu	0.00
	Mp	18.34
Overturning	Mtetrapod	16.34
	Mconc. (=W*t)	19.92
	S.F. (Normal)	1.09
	S.F. (with back fill layer effect)	9.95

	30 years
CD1	-4.1 m
CD2	-4.85 m
CD3	-7.6 m
HWL	+3.55 m
a	3.5 m
h	2.75 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.50 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.64 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	6.20 t/m
W <sub>conc. cover</sub> (1 unit)	0.52 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.86 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	4.15 t/m
ΣW	11.39 t/m

Vertical Effect	
b <sub>back fill layer</sub>	4 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	26.25 m <sup>2</sup>
Wtetrapod	18.05 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.8
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	12.96
Pp, t/m	17.82
γp, m	0.92
M <sub>pp</sub> , t.m	16.34

Rubble Cell	x <sub>sc</sub>	1.75 m
	M (Sc)	19.92 t.m/m

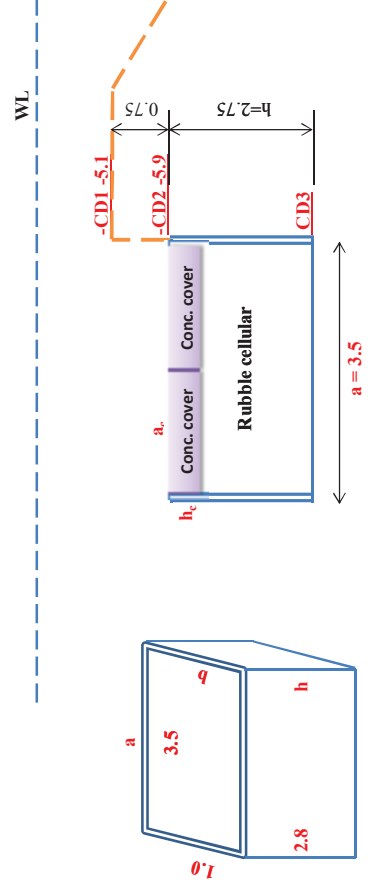
P	13.22 t/m
U	0.00 t/m
γ <sub>h</sub>	1.387 m
x <sub>u</sub>	3.500 m
Mp	18.34 t.m/m
Mu	0.00 t.m/m
M total	18.34 t.m/m

STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)  
Sand Protection Training Dike - Section for GL upto CD -10.0m

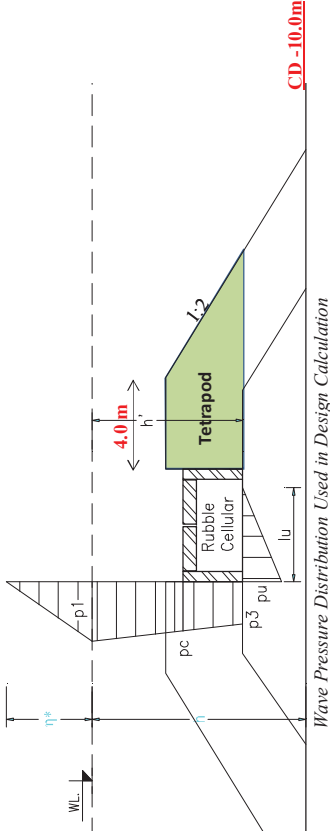
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-5.9
- Bottom Upright Wall Level (CDL):	-8.6
- Bottom Width of Upright Wall (m):	3.5
- Back Fill Level (CDL):	-5.1
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-10.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	5.058
- H <sub>max</sub> , m:	8.629
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	113.60



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

3. Uplift beneath Upright Wall

$P_3 = P_a = \alpha_3 P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation  
*breaking zone*

H<sub>D</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.12
H <sub>D</sub> (m):	0.498
L <sub>w</sub> (m):	8.629
HWL (CDL):	113.60
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-10.0
h (m):	-5.9
h' (m):	13.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	12.15
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.499
λ:	0.749
	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	9.70
α <sub>1</sub> =	0.848
α <sub>3</sub> =	0.796
(1 + cos β)	1.498
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	7.503
P <sub>1</sub> (t/m <sup>2</sup> ) =	5.62
P <sub>a</sub> (at crest level), (t/m <sup>2</sup> ) =	4.73
P <sub>3</sub> (t/m <sup>2</sup> ) =	4.48

5.3. Uplift beneath Upright Wall

P <sub>a</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	12.66
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	18.05
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	17.82
M <sub>pp</sub> , t.m	16.34

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>con.</sub> (total), t/m	11.39
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	18.05

Sliding	S.F. (Normal)	0.72
	S.F. (with back fill layer effect)	1.72
	Mu	0.00
	Mp	17.57
Overturning	Mtetrapod	16.34
	Mconc. (=W*t)	19.92
	S.F. (Normal)	1.13
	S.F. (with back fill layer effect)	16.12

	30 years
CD1	-5.1 m
CD2	-5.85 m
CD3	-8.6 m
HWL	+3.55 m
a	3.5 m
h	2.75 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.50 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.64 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	6.20 t/m
W <sub>conc. cover</sub> (1 unit)	0.52 t/m
Nos. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.86 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	4.15 t/m
ΣW	11.39 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	4 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	26.25 m <sup>2</sup>
Witetrapod	18.05 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.8
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	12.96
Pp, t/m	17.82
γp, m	0.92
Mpp, t.m	16.34

Back fill beside conc. Block

x <sub>sc</sub>	1.75 m
M (Sc)	19.92 t.m/m

P	12.66 t/m
U	0.00 t/m
γ <sub>h</sub>	1.388 m
x <sub>u</sub>	3.500 m
Mp	17.57 t.m/m
Mu	0.00 t.m/m
M total	17.57 t.m/m

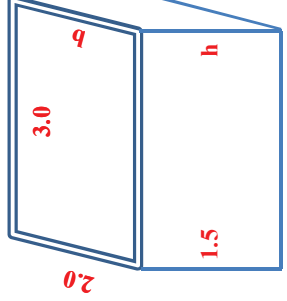
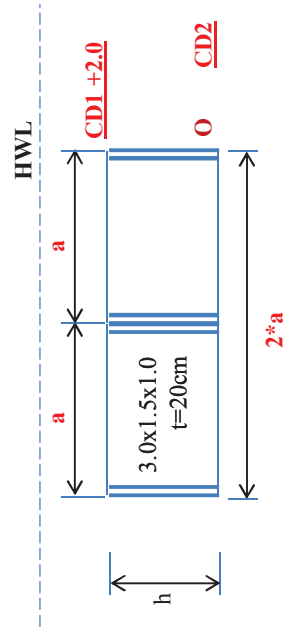
**8 STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - CASE B-1 (HWL)**

**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD 0.0m**

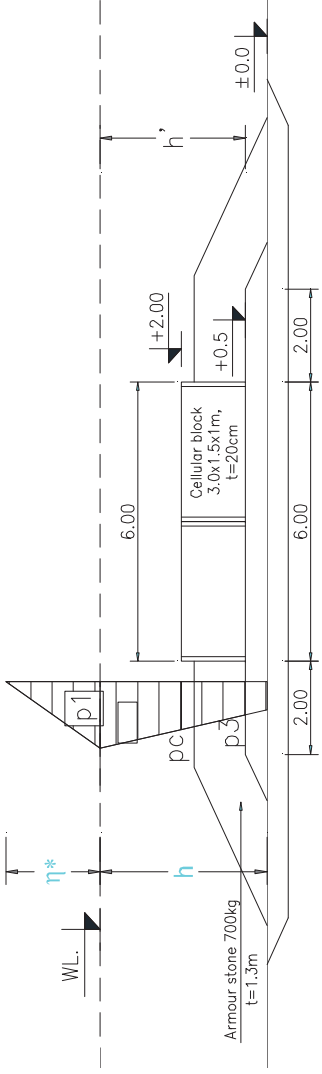
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	0.5
- Bottom Width of Upright Wall (m):	6.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	0.0
- Significant Wave Height ( $H_{1/3}$ ), m:	2.665
- $H_{max}$ , m:	3.821
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	77.14



**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (\text{if } -h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
*breaking zone*

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	68.52
$H_D$ (m):	0.366
$L$ (m):	3.821
HWL (CDL):	77.14
Sea bed Level (CDL):	3.55
Crest Level (CDL):	0.0
$h$ (m):	2.0
$h'$ (m):	3.6
$\rho_0$ (t/m <sup>3</sup> ):	3.05
$g$ (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	0.578
$\lambda$ :	0.289
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	3.92
$\alpha_1 =$	1.048
$\alpha_3 =$	0.965
$(1 + \cos \beta)$	1.366
$\alpha_1 \lambda \rho_0 H_D$	4.104
$P_1$ (t/m <sup>2</sup> ) =	2.80
$P_u$ (at crest level), (t/m <sup>2</sup> ) =	2.75
$P_3$ (t/m <sup>2</sup> ) =	2.71

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to bottom Elev.}}$ (t/m) =	4.09
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
$W_{con. \text{ cell. (total), t/m}}$	9.99

Sliding	S.F. at CD2 Elev.	1.95
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Overturnin	$M_{p \text{ at O}}$	3.09
g	$M_{conc. \text{ cell at O}} (=W*t)$	29.96
	S.F. at O	9.70

CD1	30 years
CD2	+2 m
HWL	0.5 m
a	+ 3.55 m
h	3 m
b	1.5 m
t	2.00 m
$S_{conc. \text{ cell (1 cell.)}}$	0.20 m
$\gamma_{conc.}$	1.84 m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
$W_{conc. \text{ cell (1 cell.)}}$	1.375 t/m <sup>3</sup>
$S_{rubber \text{ filling (1 cell.)}}$	3.80 t/m
$\gamma_{rubber \text{ stone}}$	4.16 m <sup>2</sup>
$\gamma_{rubber \text{ stone}}$	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
$W_{rubber \text{ filling (1 cell.)}}$	37%
Nos. of cellular block	6.19 t/m
$x_o$	2.00 unit
$M_o$	3.00 m
	29.96 tm/m

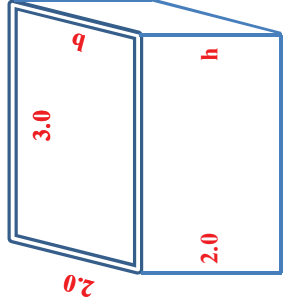
Wave	$P_{up \text{ to bottom Elev.}}$	4.09 t/m
	$\gamma_{p \text{ upto bottom Elev.}}$	0.754 m
	$M_{p \text{ at O}}$	3.09 t.m/m

**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)**  
**Sand Protection Training Dike - Section for GL upto CD -1.0m**

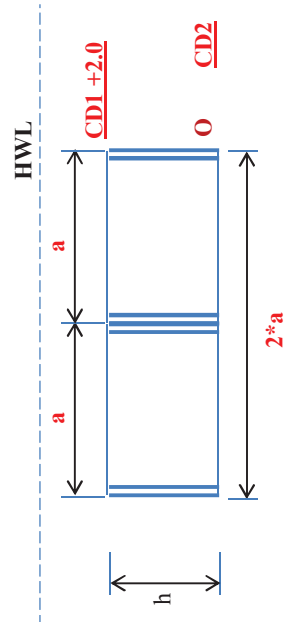
**I. Wave Force under Wave Crest**

**1. Input data:**

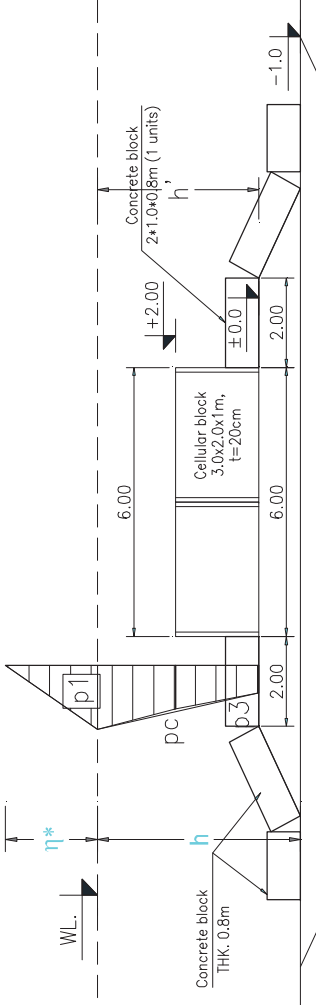
- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	0.0
- Bottom Width of Upright Wall (m):	6.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-1.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	3.084
- H <sub>max</sub> , m:	4.234
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	81.06



Bellow sketch for reference only



**2. Wave Pressure on the Front Face of an Upright Wall**



**Wave Pressure Distribution Used in Design Calculation**

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (\text{if } -h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	68.35
$H_D$ (m):	0.369
$L$ (m):	4.234
HWL (CDL):	81.06
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-1.0
$h$ (m):	2.0
$h'$ (m):	4.6
$\rho_0$ (t/m <sup>3</sup> ):	3.55
$g$ (m/s <sup>2</sup> ):	1.025
$4 \pi h / L$ :	9.81
$2 \pi h / L$ :	0.705
$\lambda$ :	0.353
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	4.35
$\alpha_1 =$	1.025
$\alpha_3 =$	0.954
$(1 + \cos \beta)$	1.369
$\alpha_1 \lambda \rho_0 H_D$	4.447
$p_1$ (t/m <sup>2</sup> ) =	3.04
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	2.98
$p_3$ (t/m <sup>2</sup> ) =	2.90

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	5.89
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
$W_{\text{con. cell. (total), t/m}}$	13.32

Sliding	S.F. at CD2 Elev.	1.81
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Overturnin	$M_{p \text{ at O}}$	5.93
g	$M_{\text{conc. cell at O}} (=W^*t)$	59.92
	S.F. at O	10.10

CD1	30 years
CD2	+2 m
HWL	0 m
a	+ 3.55 m
h	3 m
b	2 m
t	2.00 m
	0.20 m
$S_{\text{conc. cell (1 cell.)}}$	1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$	1.375 t/m <sup>3</sup>
$W_{\text{conc. cell (1 cell.)}}$	5.06 t/m
$S_{\text{rubber filling (1 cell.)}}$	4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$	2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
$W_{\text{rubber filling (1 cell.)}}$	8.26 t/m
Nos. of cellular block	2.00 unit
$x_{\text{O}}$	4.50 m
$M_{\text{O}}$	59.92 tm/m

Wave	$P_{\text{up to bottom Elev.}}$	5.89 t/m
	$\gamma_{p \text{ upto bottom Elev.}}$	1.008 m
	$M_{p \text{ at O}}$	5.93 t.m/m

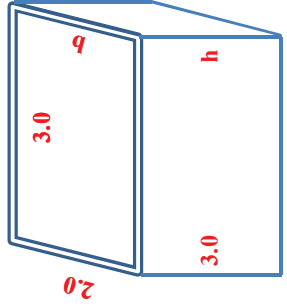


**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD -2.0m**

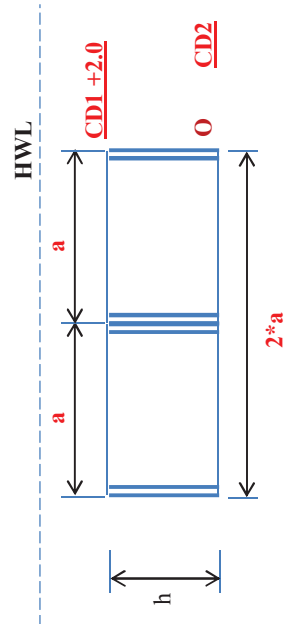
**I. Wave Force under Wave Crest**

**1. Input data:**

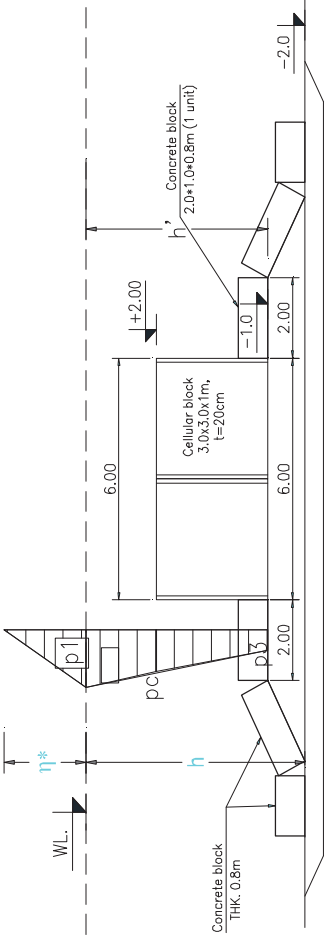
- Wave Return Period:	<b>30 years</b>
- Tide Level - HWL (CDL):	<b>3.55</b>
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-1.0
- Bottom Width of Upright Wall (m):	6.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	<b>-2.0</b>
- Significant Wave Height (H <sub>1/3</sub> ), m:	3.578
- H <sub>max</sub> , m:	4.762
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	85.78



*Bellow sketch for reference only*



**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (\text{if } -h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
*breaking zone*

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	63.333
$H_D$ (m):	0.449
$L$ (m):	4.762
$L$ (m):	85.78
HWL (CDL):	3.55
Sea bed Level (CDL):	-2.0
Crest Level (CDL):	2.0
$h$ (m):	5.6
$h'$ (m)	4.55
$\rho_0$ (t/m <sup>3</sup> ):	1.025
$g$ (m/s <sup>2</sup> ):	9.81
$4\pi h/L$ :	0.813
$2\pi h/L$ :	0.407
$\lambda$	1.000

*Upright wall*

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	5.17
$\alpha_1 =$	1.003
$\alpha_3 =$	0.937
$(1 + \cos \beta)$	1.449
$\alpha_1 \lambda \rho_0 H_D$	4.896
$P_1$ (t/m <sup>2</sup> ) =	3.55
$P_c$ (at crest level), (t/m <sup>2</sup> ) =	3.47
$P_3$ (t/m <sup>2</sup> ) =	3.32

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	10.19
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	<b>0.80</b>
$W_{\text{con. cell. (total), t/m}}$	19.97

<b>Sliding</b>	<b>1.6</b>
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$M_{p \text{ at O}}$	15.45
$M_{\text{conc. cell at O}} (=W^*t)$	89.88
<b>S.F. at O</b>	<b>5.82</b>

<b>CD1</b>	30 years
<b>CD2</b>	+2 m
<b>HWL</b>	-1 m
<b>a</b>	+ 3.55 m
<b>h</b>	3.0 m
<b>b</b>	3.0 m
<b>t</b>	2.0 m
<b>S<sub>conc. cell</sub> (1 cell.)</b>	0.2 m
$\gamma_{\text{conc.}}$	1.84 t/m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4 t/m <sup>3</sup>
$W_{\text{conc. cell}} (1 \text{ cell.})$	1.375 t/m <sup>3</sup>
$W_{\text{rubber filling}} (1 \text{ cell.})$	7.59 t/m
$S_{\text{rubber filling}}$	4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$	2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$	1.575 t/m <sup>3</sup>
<b>void ratio</b>	37%
$W_{\text{rubber filling}} (1 \text{ cell.})$	12.38 t/m
<b>Nos. of cellular block</b>	<b>2.00</b> unit
$x_o$	4.50 m
$M_o$	89.88 tm/m

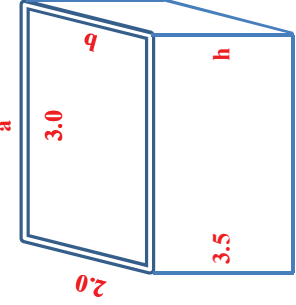
<b>Wave</b>	$P_{\text{up to bottom Elev.}}$	10.19 t/m
	$\gamma_{p \text{ upto bottom Elev.}}$	1.516 m
	$M_{p \text{ at O}}$	15.45 t.m/m

**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD-2.0m - From Sta. 1+750-2+950**

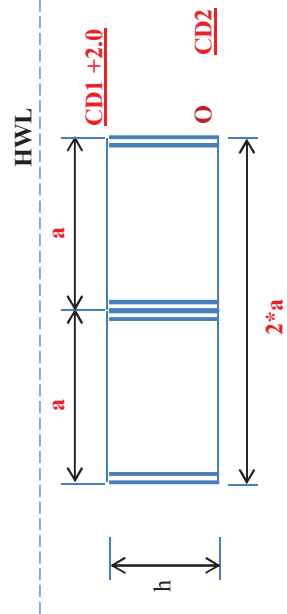
**I. Wave Force under Wave Crest**

**1. Input data:**

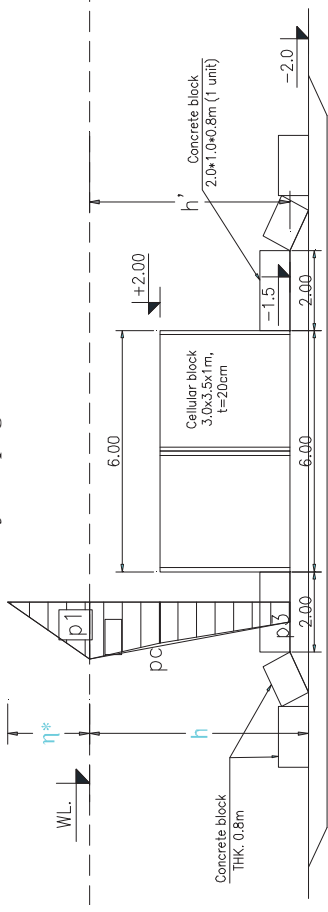
- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-1.5
- Bottom Width of Upright Wall (m):	6.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-2.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	3.578
- H <sub>max</sub> , m:	4.762
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	85.78



Bellow sketch for reference only



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

H<sub>D</sub> = H<sub>max</sub>  
 breaking zone

**5. Results of Calculation**

**5.1. Input data**

β (deg) =	30 years
cos β:	63.333
H <sub>D</sub> (m):	0.449
L (m):	4.762
HWL (CDL):	85.78
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-2.0
h (m):	2.0
h' (m):	5.6
ρ <sub>0</sub> (t/m <sup>3</sup> ):	5.05
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	0.813
λ:	0.407
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

η* =	5.17
α <sub>1</sub> =	1.003
α <sub>3</sub> =	0.930
(1+cosβ)	1.449
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	4.896
P <sub>1</sub> (t/m <sup>2</sup> ) =	3.55
P <sub>c</sub> (at crest level), (t/m <sup>2</sup> ) =	3.47
P <sub>3</sub> (t/m <sup>2</sup> ) =	3.30

**5.3. Total wave pressure on Upright Wall**

P <sub>up to bottom Elev.</sub> (t/m) =	11.84
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**II. Safety Factor**

μ (friction factor between conc. cell. stone & stone):	0.80
W <sub>con. cell.</sub> (total), t/m	23.30

Sliding	S.F. at CD2 Elev.	1.6
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Overturnin	Mp <sub>atO</sub>	20.98
g	M <sub>conc. cell.atO</sub> (=W*t)	104.86
	S.F. at O	5.00

CD1	30 years
CD2	+2 m
HWL	-1.5 m
a	+ 3.55 m
h	3 m
b	3.5 m
t	2.00 m
S <sub>conc. cell</sub> (1 cell.)	0.20 m
γ <sub>conc.</sub>	1.84 t/m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)	1.375 t/m <sup>3</sup>
S <sub>rubber filling</sub> (1 cell.)	8.86 t/m
γ <sub>rubber stone</sub>	4.16 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
W <sub>rubber filling</sub> (1 cell.)	37%
Nos. of cellular block	14.45 t/m
X <sub>O</sub>	2.00 unit
M <sub>O</sub>	4.50 m
	104.86 tm/m

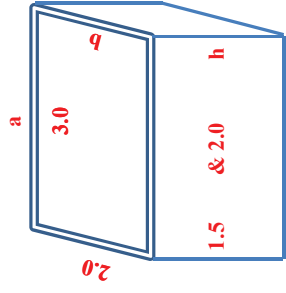
Wave	P <sub>up to bottom Elev.</sub>	11.84 t/m
	Y <sub>p upto bottom Elev.</sub>	1.771 m
	Mp <sub>atO</sub>	20.98 t.m/m

**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD -3.0m**

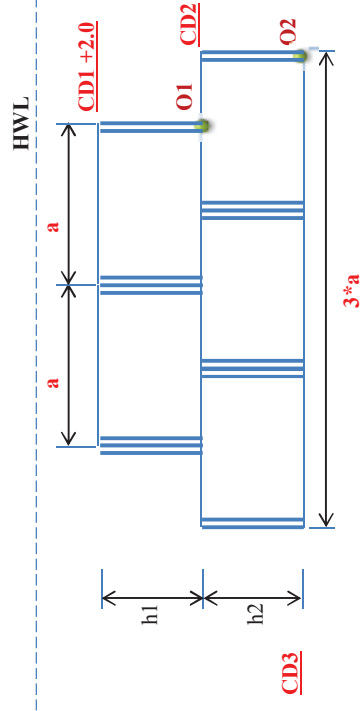
**I. Wave Force under Wave Crest**

**I. Input data:**

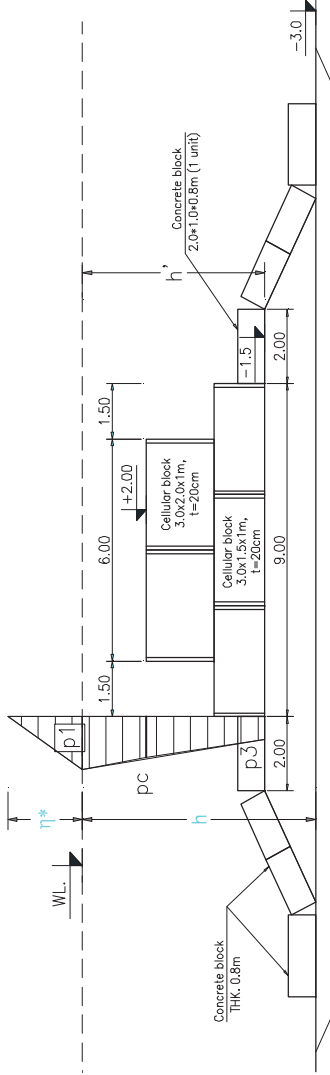
- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-1.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-3.0
- Significant Wave Height ( $H_{1/3}$ ), m:	4.08
- $H_{max}$ , m:	5.365
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	90.82



*Below sketch for reference only*



**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_w g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$

*breaking zone*

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	60.793
$H_D$ (m):	0.488
$L$ (m):	5.365
HWL (CDL):	90.82
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-2.5
$h$ (m):	2.0
$h'$ (m):	6.05
$\rho_0$ (t/m <sup>3</sup> ):	5.05
$g$ (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	0.837
$\lambda$ :	0.419
	1.000 <i>Upright wall</i>

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	5.99
$\alpha_1 =$	0.998
$\alpha_3 =$	0.932
$(1 + \cos \beta)$	1.488
$\alpha_1 \lambda \rho_w H_D$	5.488
$p_1$ (t/m <sup>2</sup> ) =	4.08
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	4.00
$p_{O1}$ (at CD2 level), (t/m <sup>2</sup> ) =	3.89
$p_3$ (t/m <sup>2</sup> ) =	3.80

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to CD2 Elev.}}$ (t/m) =	7.69
$P_{up \text{ to bottom Elev.}}$ (t/m) =	13.65

**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W1 con. cell. (total), t/m	13.32
W2 con. cell. (total), t/m	14.98

Sliding	<b>S.F. at CD2 Elev. 1.38</b>
	<b>S.F. at CD3 Elev. 1.66</b>

$M_{p \text{ at O1}}$	7.75
$M_{conc. \text{ cell. at O1}} (=W^*t)$	59.92
Overturning	<b>S.F. at O1 7.73</b>
$M_{p \text{ at O2}}$	24.17
$M_{conc. \text{ cell. at O2}} (=W^*t)$	168.77
	<b>S.F. at O2 6.98</b>

CD1	30 years
CD2	+2 m
CD3	+0 m
HWL	-1.50 m
$a$	+3.55 m
$h_1$	3 m
$h_2$	2 m
$b$	1.5 m
$t$	2.00 m
$S_{conc. \text{ cell}} (1 \text{ cell.})$	0.20 m
$S_{conc. \text{ cell}}$	1.84 m <sup>2</sup>
$\gamma'_{conc.}$	2.4 t/m <sup>3</sup>
$\gamma'_{conc.}$	1.375 t/m <sup>3</sup>
W1 $_{conc. \text{ cell}} (1 \text{ cell.})$	5.06 t/m
W2 $_{conc. \text{ cell}} (1 \text{ cell.})$	3.80 t/m
$S_{rubber \text{ filling}} (1 \text{ cell.})$	4.16 m <sup>2</sup>
$\gamma_{rubber \text{ stone}}$	2.6 t/m <sup>3</sup>
$\gamma_{rubber \text{ stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
W1 $_{rubber \text{ filling}} (1 \text{ cell.})$	8.26 t/m
W2 $_{rubber \text{ filling}} (1 \text{ cell.})$	6.19 t/m
Nos. of cellular block	2.00 unit
$X_{O1}$	4.50 m
$M_{O1}$	59.92 tm/m
Nos. of cellular block	3.00 unit
$X_{O2}$	6.00 m
$M_{O2}$	168.77 tm/m

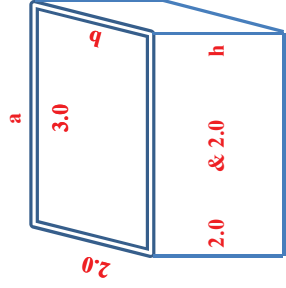
$P_{up \text{ to CD2 Elev.}}$	7.69 t/m
$\gamma'_{p \text{ upto CD2 Elev.}}$	1.01
$M_{p \text{ at O1}}$	7.75 t.m/m
$P_{up \text{ to bottom Elev.}}$	13.65 t/m
$\gamma'_{p \text{ upto bottom Elev.}}$	1.771 m
$M_{p \text{ at O2}}$	24.17 t.m/m

**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD -3.0m**

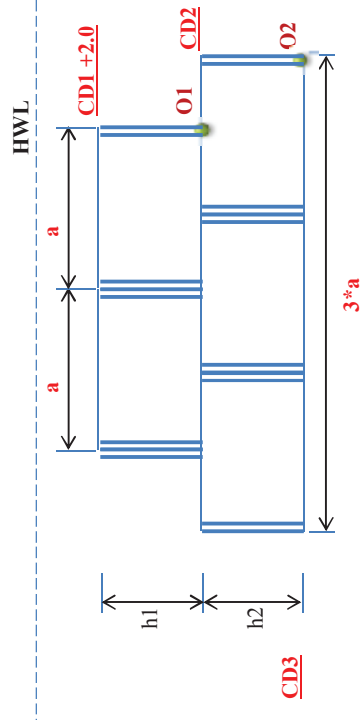
**I. Wave Force under Wave Crest**

**I. Input data:**

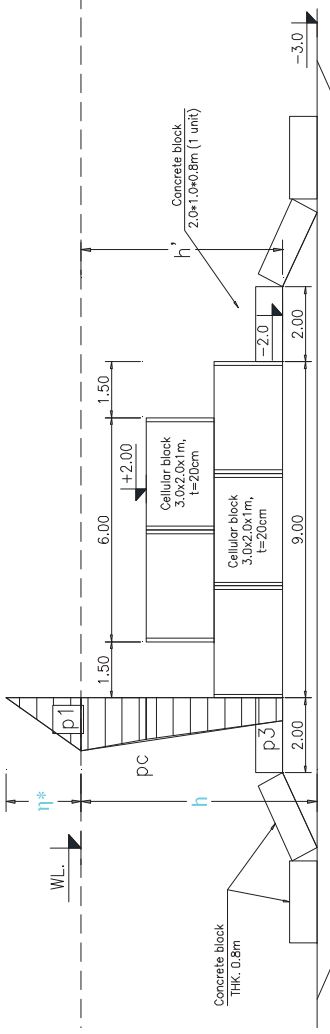
- Wave Return Period:	<b>30 years</b>
- Tide Level - HWL (CDL):	<b>3.55</b>
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-2.0
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	<b>-3.0</b>
- Significant Wave Height ( $H_{1/3}$ ), m:	4.08
- $H_{max}$ , m:	5.365
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	90.82



*Below sketch for reference only*



**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0gH_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$

*breaking zone*

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	60.793
$H_D$ (m):	0.488
L (m):	5.365
HWL (CDL):	90.82
Sea bed Level (CDL):	<b>-3.0</b>
Crest Level (CDL):	2.0
h (m):	6.55
h' (m):	5.55
$\rho_0$ (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
$4\pi h/L$ :	0.906
$2\pi h/L$ :	0.453
$\lambda$ :	1.000 <i>Upright wall</i>

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	5.99
$\alpha_1 =$	0.983
$\alpha_3 =$	0.920
(1+cos $\beta$ )	1.488
$\alpha_1 \lambda \rho_0 H_D$	5.405
$p_1$ (t/m <sup>2</sup> ) =	4.02
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	3.93
$p_{O1}$ (at CD2 level), (t/m <sup>2</sup> ) =	3.82
$p_3$ (t/m <sup>2</sup> ) =	3.70

**5.3. Total wave pressure on Upright Wall**

$P_{up.to CD2 Elev.}$ (t/m) =	7.51
$P_{up.to bottom Elev.}$ (t/m) =	15.26

**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	<b>0.80</b>
W1con. cell. (total), t/m	13.32
W2con. cell. (total), t/m	19.97

Sliding	<b>S.F. at CD2 Elev. 1.42</b>
	<b>S.F. at CD3 Elev. 1.74</b>

Overturning	$M_{p.at O1}$	7.58
	$M_{conc. cell. at O1 (=W*t)}$	59.92
	<b>S.F. at O1</b>	<b>7.90</b>
	$M_{p.at O2}$	30.95
	$M_{conc. cell. at O2 (=W*t)}$	199.73
	<b>S.F. at O2 6.45</b>	

CD1	30 years
CD2	+2 m
CD3	+0 m
HWL	+2 m
a	+ 3.55 m
h1	3 m
h2	2 m
b	2.00 m
t	0.20 m
$S_{conc. cell.}$ (1 cell.)	1.84 m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
$\gamma'_{conc.}$	1.375 t/m <sup>3</sup>
W1 <sub>conc. cell.</sub> (1 cell.)	5.06 t/m
W2 <sub>conc. cell.</sub> (1 cell.)	5.06 t/m
$S_{rubber filling}$ (1 cell.)	4.16 m <sup>2</sup>
$\gamma_{rubber stone}$	2.6 t/m <sup>3</sup>
$\gamma'_{rubber stone}$	1.575 t/m <sup>3</sup>
void ratio	37%
W1 <sub>rubber filling</sub> (1 cell.)	8.26 t/m
W2 <sub>rubber filling</sub> (1 cell.)	8.26 t/m
Nos. of cellular block	2.00 unit
$X_{O1}$	4.50 m
$M_{O1}$	59.92 tm/m
Nos. of cellular block	3.00 unit
$X_{O2}$	6.00 m
$M_{O2}$	199.73 tm/m

Wave	$P_{up.to CD2 Elev.}$	7.51 t/m
	$\gamma'_{p up.to CD2 Elev.}$	1.01
	$M_{p at O1}$	7.58 tm/m
	$P_{up.to bottom Elev.}$	15.26 t/m
	$\gamma'_{p up.to bottom Elev.}$	2.028 m
	$M_{p at O2}$	30.95 tm/m

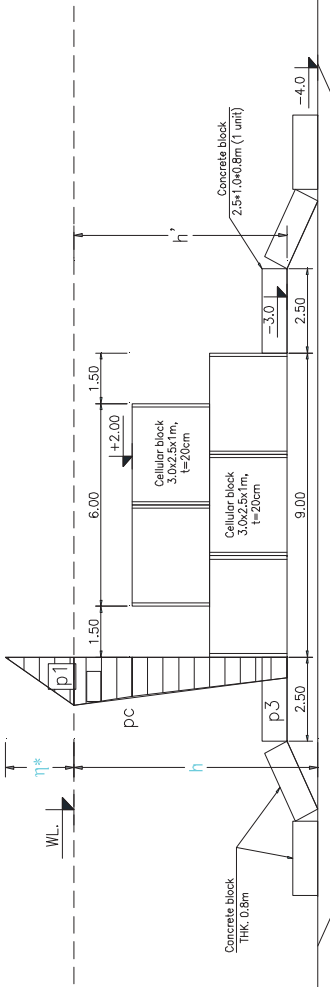
**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD -3.0m**

**I. Wave Force under Wave Crest**

**I. Input data:**

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-3.0
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-4.0
- Significant Wave Height ( $H_{1/3}$ ), m:	4.636
- $H_{max}$ , m:	6.035
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	96.05

**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0gH_D$

At toe of upright wall:  $P_3 = P_u = \alpha_5 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_5 = 1 + \frac{h'}{\eta^*} \text{ (if } h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	60.926
$H_D$ (m):	0.486
$L$ (m):	6.035
HWL (CDL):	96.05
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-4.0
$h$ (m):	2.0
$h'$ (m):	7.55
$\rho_0$ (t/m <sup>3</sup> ):	6.55
$g$ (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	0.988
$\lambda$ :	0.494
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	6.73
$\alpha_1 =$	0.965
$\alpha_3 =$	0.904
$(1 + \cos \beta)$	1.486
$\alpha_1 \lambda \rho_0 H_D$	5.968
$p_1$ (t/m <sup>2</sup> ) =	4.43
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	4.33
$p_{O1}$ (at CD2 level), (t/m <sup>2</sup> ) =	4.17
$p_3$ (t/m <sup>2</sup> ) =	4.01

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to CD2 Elev.}}$ (t/m) =	10.22
$P_{up \text{ to bottom Elev.}}$ (t/m) =	20.85

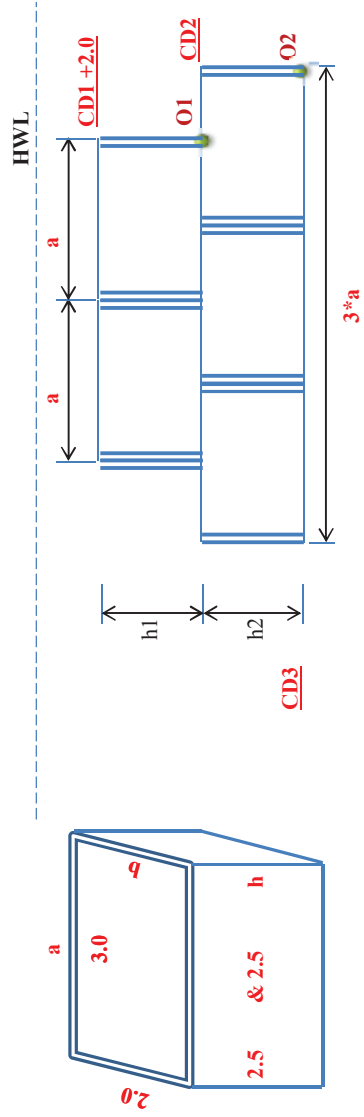
**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W1 con. cell. (total), t/m	16.64
W2 con. cell. (total), t/m	24.97

Sliding	<b>S.F. at CD2 Elev. 1.30</b>
	<b>S.F. at CD3 Elev. 1.60</b>

Overturning	$M_{p \text{ at O1}}$	12.91
	$M_{\text{conc. cell. at O1}} (=W*u)$	74.90
	<b>S.F. at O1</b>	<b>5.80</b>
	$M_{p \text{ at O2}}$	53.01
	$M_{\text{conc. cell. at O2}} (=W*u)$	249.67
	<b>S.F. at O2</b>	<b>4.71</b>

*Bellow sketch for reference only*



CD1	+2	30 years
CD2	-0.50	
CD3	-3.00	
HWL	+3.55	
a	3	
h1	2.5	
h2	2.5	
t	2.00	
t	0.20	
$S_{\text{conc. cell.}}$ (1 cell.)	1.84	m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4	t/m <sup>3</sup>
$\gamma'_{\text{conc.}}$	1.375	t/m <sup>3</sup>
W1 conc. cell. (1 cell.)	6.33	t/m
W2 conc. cell. (1 cell.)	6.33	t/m
$S_{\text{rubber filling}}$ (1 cell.)	4.16	m <sup>2</sup>
$\gamma_{\text{rubber stone}}$	2.6	t/m <sup>3</sup>
$\gamma'_{\text{rubber stone}}$	1.575	t/m <sup>3</sup>
void ratio	37%	
W1 rubber filling (1 cell.)	10.32	t/m
W2 rubber filling (1 cell.)	10.32	t/m
Nos. of cellular block	2.00	unit
$X_{O1}$	4.50	m
$M_{O1}$	74.90	tm/m
Nos. of cellular block	3.00	unit
$X_{O2}$	6.00	m
$M_{O2}$	249.67	tm/m

Wave	$P_{up \text{ to CD2 Elev.}}$	10.22	t/m
	$\gamma_p$ upto CD2 Elev.	1.26	
	$M_p$ at O1	12.91	t.m/m
	$P_{up \text{ to bottom Elev.}}$	20.85	t/m
	$\gamma_p$ upto bottom Elev.	2.542	m
	$M_p$ at O2	53.01	t.m/m

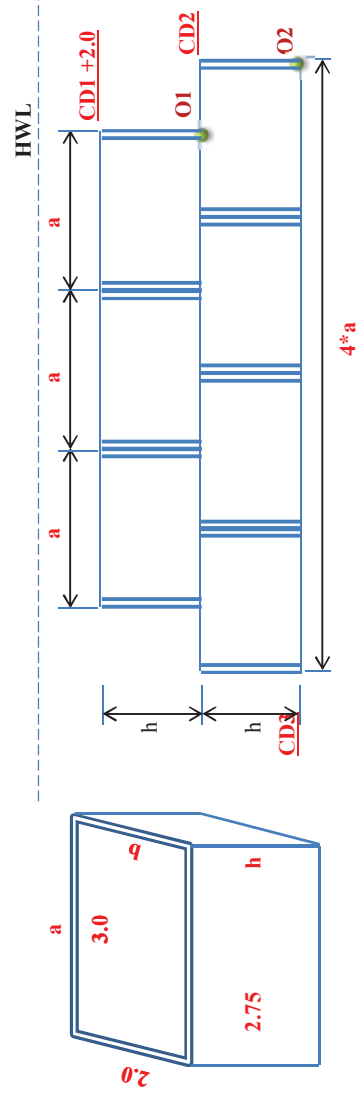
**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)**  
**Sand Protection Training Dike - Section for GL upto CD -5.0m**

**I. Wave Force under Wave Crest**

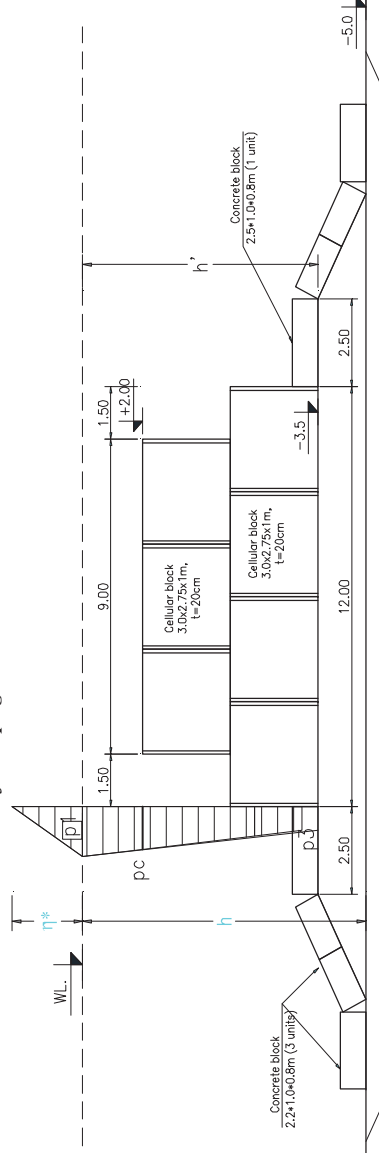
**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-3.5
- Bottom Width of Upright Wall (m):	12.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-5.0
- Significant Wave Height ( $H_{1/3}$ ), m:	4.96
- $H_{max}$ , m:	6.615
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	100.31

*Bellow sketch for reference only*



**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0gH_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3P_1$

$$\lambda = \exp\left[-10\left(\frac{h}{L}\right)^{1.5}\left(1 - \frac{h'}{h}\right)^5\right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (if \ h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$

*breaking zone*

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	60.947
$H_D$ (m):	0.486
L (m):	6.615
HWL (CDL):	100.31
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-5.0
h (m):	2.0
h' (m):	8.55
$\rho_0$ (t/m <sup>3</sup> ):	7.05
g (m/s <sup>2</sup> ):	1.025
4 $\pi h/L$ :	9.81
2 $\pi h/L$ :	1.071
$\lambda$ :	0.536
	1.000

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^*$ =	7.37
$\alpha_1$ =	0.946
$\alpha_3$ =	0.894
$(1 + \cos \beta)$ :	1.486
$\alpha_1 \lambda \rho_0 H_D$ :	6.413
$P_1$ (t/m <sup>2</sup> ) =	4.76
$P_c$ (at crest level), (t/m <sup>2</sup> ) =	4.65
$P_{O1}$ (at CD2 level), (t/m <sup>2</sup> ) =	4.46
$P_3$ (t/m <sup>2</sup> ) =	4.26

**5.3. Total wave pressure on Upright Wall**

$P_{up\ to\ CD2\ Elev.}$ (t/m) =	11.99
$P_{up\ to\ bottom\ Elev.}$ (t/m) =	24.51

**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
Wcon. cell. (total), t/m	64.08

Sliding	<b>S.F. at CD2 Elev. 1.83</b>
	<b>S.F. at CD3 Elev. 2.09</b>

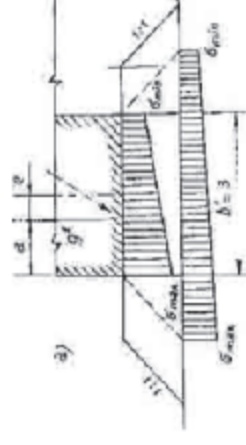
Overturning	$M_{p\ at\ O1}$	16.66
	$M_{conc.\ cell.\ at\ O1 (=W^*t)}$	123.58
	<b>S.F. at O1</b>	<b>7.42</b>
g	$M_{p\ at\ O2}$	68.66
	$M_{conc.\ cell.\ at\ O2 (=W^*t)}$	384.49
	<b>S.F. at O2</b>	<b>5.60</b>

CD1	30 years
CD2	+ 2 m
CD3	-0.8 m
HWL	-3.5 m
a	+ 3.55 m
h	3 m
b	2.75 m
t	2.00 m
S <sub>conc. cell</sub> (1 cell.)	0.20 m
$\gamma_{conc.}$	1.84 m <sup>2</sup>
$\gamma'_{conc.}$	2.4 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)	1.375 t/m <sup>3</sup>
S <sub>rubber filling</sub> (1 cell.)	6.96 t/m
$\gamma_{rubber\ stone}$	4.16 m <sup>2</sup>
$\gamma'_{rubber\ stone}$	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
W <sub>rubber filling</sub> (1 cell.)	37%
Nos. of cellular block	11.35 t/m
$X_{O1}$	3.00 unit
$M_{O1}$	4.50 m
Nos. of cellular block	123.58 tm/m
$X_{O2}$	4.00 unit
$M_{O2}$	6.00 m
	384.49 tm/m

Wave	$P_{up\ to\ CD2\ Elev.}$	11.99 t/m
	$\gamma_{up\ to\ CD2\ Elev.}$	1.39
	$M_{p\ at\ O1}$	16.66 t.m/m
	$P_{up\ to\ bottom\ Elev.}$	24.51 t/m
	$\gamma_{up\ to\ bottom\ Elev.}$	2.801 m
	$M_{p\ at\ O2}$	68.66 t.m/m

**Bearing capacity of soft soil under foundation**

a = 4.93 m  
 b = 12 m  
 b/3 = 4 m  
 Checking condition  $a \geq b/3$ :  
 c = 1.07 m



$\sigma_{max} = 8.20\ t/m^2 = 0.82\ kg/cm^2$   
 $\sigma_{min} = 2.48\ t/m^2 = 0.25\ kg/cm^2$   
 - ( - ) rubberstone = 6.0 kg/cm<sup>2</sup>  
 = 6.0 kg/cm<sup>2</sup> **OK**  
 = 0.82 kg/cm<sup>2</sup> **OK**  
 = 0.25 kg/cm<sup>2</sup> **OK**

soft soil = 2.0 kg/cm<sup>2</sup>  
 $b' = 12\ m$   
 $hn = 1.5\ m$   
 $\gamma_{c-k} = 1.575\ t/m^3$   
 $\sigma'_{max} = 8.92\ t/m^2 = 0.89\ kg/cm^2$   
 $\sigma'_{min} = 4.35\ t/m^2 = 0.43\ kg/cm^2$   
**OK**  
**OK**

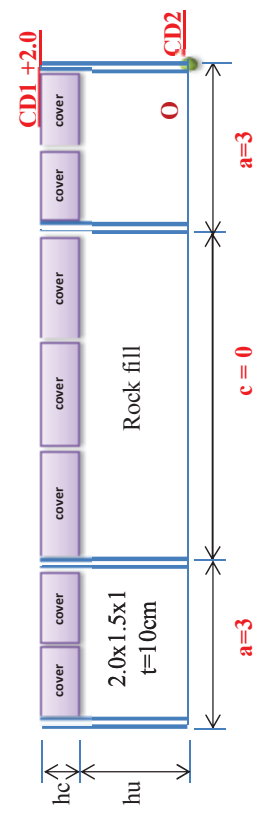
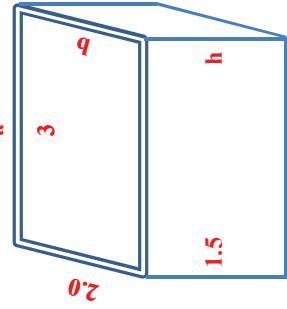
**9 STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - CASE B-2 (HWL)**

**D. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD 0.0m**  
(Clause 5.5.2 - OCDI)

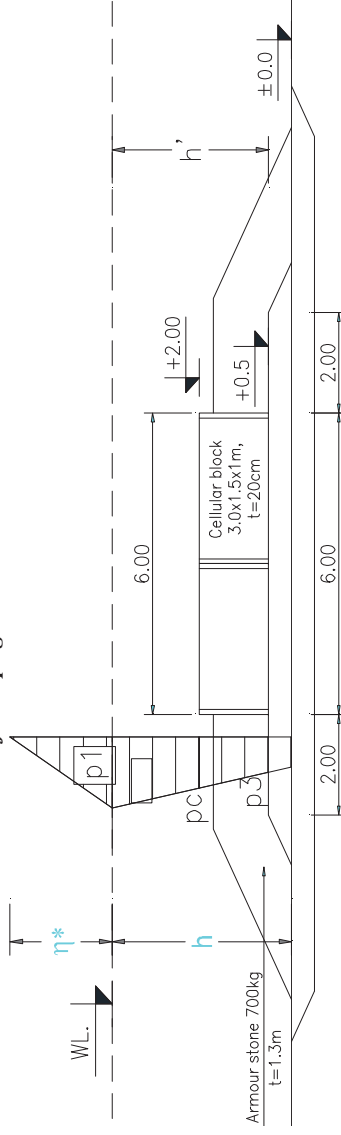
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	0.5
- Bottom Width of Upright Wall (m):	6.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	0.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	2.665
- H <sub>max</sub> , m:	3.821
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	77.14



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_w g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{h} \eta^* \quad (\text{if } h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	68.52
$H_D$ (m):	0.366
$L$ (m):	3.821
HWL (CDL):	77.14
Sea bed Level (CDL):	3.55
Crest Level (CDL):	0.0
$h$ (m):	2.0
$h'$ (m):	3.55
$\rho_0$ (t/m <sup>3</sup> ):	3.05
$g$ (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	0.578
$\lambda$ :	0.289
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	3.92
$\alpha_1 =$	1.048
$\alpha_3 =$	0.965
$(1 + \cos \beta)$	1.366
$\alpha_1 \lambda \rho_0 H_D$	4.104
$P_1$ (t/m <sup>2</sup> ) =	2.80
$P_c$ (at crest level), (t/m <sup>2</sup> ) =	2.75
$P_3$ (t/m <sup>2</sup> ) =	2.71

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	4.09
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
$W_{\text{con. cell.}}$ (total), t/m	7.92
$W_{\text{conc. cover}}$ (total), t/m	3.58
$W_{\text{core rubble at bottom}}$ , t/m	0.00

**Sliding**

<b>S.F. at bottom Elev.</b>	<b>2.25</b>
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**Overturnin g**

$M_{p \text{ at O}}$	3.08
$M_{\text{conc. cell at O2}} (=W*t)$	55.09
<b>S.F. at O</b>	<b>17.89</b>

<b>CDI</b>	30 years
<b>CD2</b>	+2 m
<b>HWL</b>	+0.5 m
<b>a</b>	+3.55 m
<b>h</b>	3 m
<b>hu</b>	1.5 m
<b>b</b>	1.00 m
<b>t</b>	2.00 m
<b>c</b>	0.20 m
<b>hc</b>	0.00 m
<b>a<sub>e, side</sub></b>	0.50 m
<b>a<sub>e, center</sub></b>	2.6 m
$S_{\text{conc. cell}}$ (1 cell.)	0.00 m
$S_{\text{conc.}}$	1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4 t/m <sup>3</sup>
$\gamma'_{\text{conc.}}$	1.375 t/m <sup>3</sup>
$W_{\text{conc. cell}}$ (1 cell.)	3.80 t/cell.
$S_{\text{rubble filling}}$ (1 cell.)	4.16 m <sup>2</sup>
$\gamma_{\text{rubble stone}}$	2.6 t/m <sup>3</sup>
$\gamma'_{\text{rubble stone}}$	1.575 t/m <sup>3</sup>
<b>void ratio</b>	37%
$W_{\text{rubble filling}}$ (1 cell.)	4.13 t/cell.
$W_{\text{conc. cover side}}$	1.79 t/m
$W_{\text{conc. cover center}}$	0.00 t/m
$W_{\text{core rubble at bottom}}$	0.00 t/m
<b>Nos. of cellular block</b>	<b>2.00</b> unit
$x_{O1}$	3.00 m
$M_{O1}$	55.09 tm/m

**Upper Layer**

<b>Wave</b>	
$P_{\text{up to bottom Elev.}}$	4.09 t/m
$y_{p \text{ upto bottom Elev.}}$	0.752 m
$M_{p \text{ at O2}}$	3.08 t.m/m

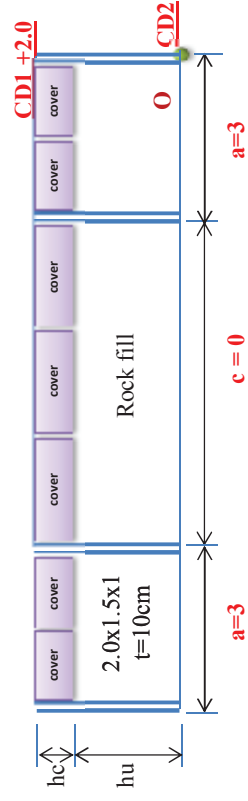
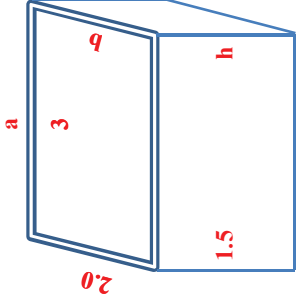


**E. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -1.0m**  
(Clause 5.5.2 - OCDI)

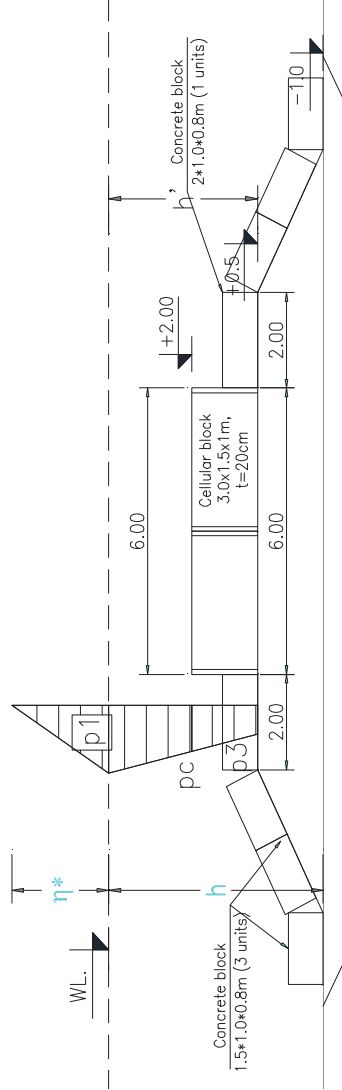
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall (CDL):	0.5
- Bottom Width of Upright Wall (m):	6.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-1.0
- Significant Wave Height ( $H_{1/3}$ ), m:	3.084
- $H_{max}$ , m:	4.234
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	81.06



**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	68.35
$H_D$ (m):	0.369
L (m):	4.234
HWL (CDL):	81.06
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-1.0
h (m):	2.0
h' (m):	4.55
$\rho_0$ (t/m <sup>3</sup> ):	3.05
g (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	0.705
$\lambda$ :	0.353
$\lambda$ :	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	4.35
$\alpha_1 =$	1.025
$\alpha_3 =$	0.960
$(1 + \cos \beta)$	1.369
$\alpha_1 \lambda \rho_0 H_D$	4.447
$P_1$ (t/m <sup>2</sup> ) =	3.04
$P_c$ (at crest level), (t/m <sup>2</sup> ) =	2.98
$P_3$ (t/m <sup>2</sup> ) =	2.92

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to bottom Elev. (t/m)}}$ =	4.43
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W <sub>conc. cell. (total)</sub> , t/m	7.92
W <sub>conc. cover (total)</sub> , t/m	3.58
W <sub>conc. rubble at bottom</sub> , t/m	0.00

**Sliding**

<b>S.F. at bottom Elev.</b>	<b>2.08</b>
$M_{p \text{ at O}}$	3.33
$M_{\text{conc. cell. at O2}} (=W^*t)$	55.09
<b>S.F. at O</b>	<b>16.53</b>

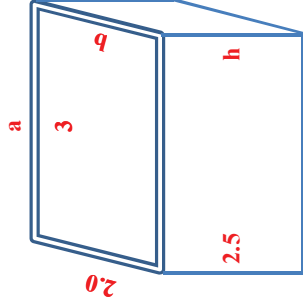
CDI	30 years	+ 2 m
CD2		+ 0.5 m
HWL		+ 3.55 m
a		3 m
h		1.5 m
hu		1.00 m
b		2.00 m
t		0.20 m
c		0.00 m
hc		0.50 m
a <sub>c side</sub>		2.6 m
a <sub>c center</sub>		0.00 m
S <sub>conc. cell (1 cell.)</sub>		1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$		2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$		1.375 t/m <sup>3</sup>
W <sub>conc. cell (1 cell.)</sub>		3.80 t/cell.
S <sub>rubber filling (1 cell.)</sub>		4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$		2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$		1.575 t/m <sup>3</sup>
void ratio		37%
W <sub>rubber filling (1 cell.)</sub>		4.13 t/cell.
W <sub>conc. cover side</sub>		1.79 t/m
W <sub>conc. cover center</sub>		0.00 t/m
W <sub>conc. rubble at bottom</sub>		0.00 t/m
Nos. of cellular block		2.00 unit
X <sub>O1</sub>		3.00 m
M <sub>O1</sub>		55.09 tm/m
Upper Layer		
Wave		
P <sub>up to bottom Elev.</sub>		4.43 t/m
Y <sub>p upto bottom Elev.</sub>		0.753 m
M <sub>p at O2</sub>		3.33 t.m/m

**F. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -2.0m**  
(Clause 5.5.2 - OCDI)

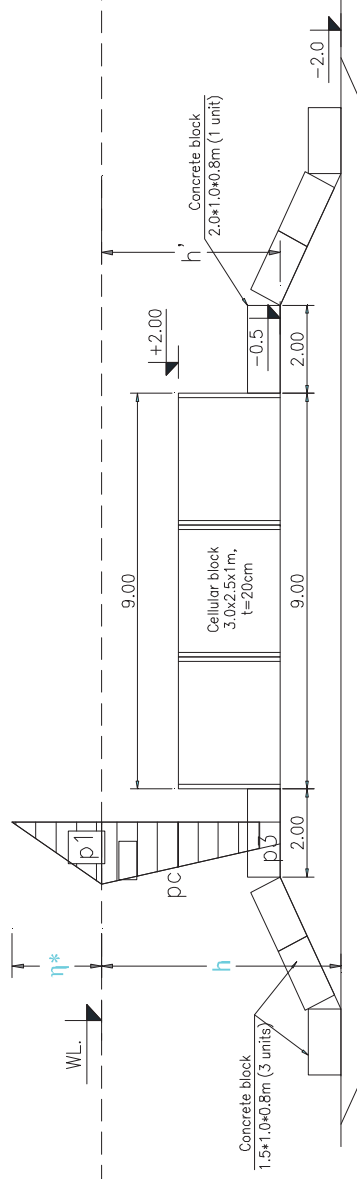
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-0.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-2.0
- Significant Wave Height ( $H_{1/3}$ ), m:	3.578
- $H_{max}$ , m:	4.762
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	85.78



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	63.333
$H_D$ (m):	0.449
L (m):	4.762
HWL (CDL):	85.78
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-2.0
h (m):	2.0
h' (m):	5.55
$\rho_0$ (t/m <sup>3</sup> ):	4.05
g (m/s <sup>2</sup> ):	1.025
4 $\pi h / L$ :	9.81
2 $\pi h / L$ :	0.813
$\lambda$ :	0.407
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	5.17
$\alpha_1 =$	1.003
$\alpha_3 =$	0.944
$(1 + \cos \beta)$	1.449
$\alpha_1 \lambda \rho_0 H_D$	4.896
$p_1$ (t/m <sup>2</sup> ) =	3.55
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	3.47
$p_3$ (t/m <sup>2</sup> ) =	3.35

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	8.52
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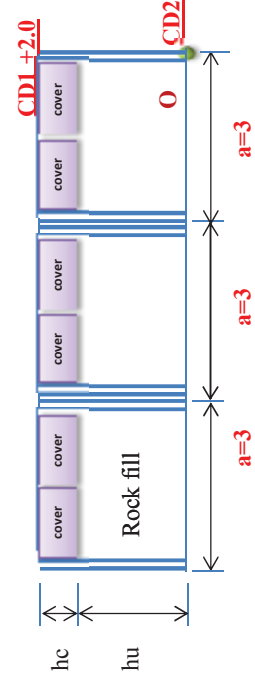
**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
Wconc. cell. (total), t/m	21.87
Wconc. cover (total), t/m	5.36

Sliding	<b>S.F. at bottom Elev.</b>	2.56
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Overturnin g	$M_{p \text{ at } O}$	10.71
	$M_{\text{conc. cell. at } O2} (=W*t)$	220.97
	<b>S.F. at O</b>	20.62

Sketch of detail of super structure  
HWL



CD1	30 years	+2	m
CD2		-0.5	m
HWL		+ 3.55	m
a		3	m
h		2.5	m
hu		2.00	m
b		2.00	m
t		0.20	m
hc		0.50	m
a inside		2.6	m
S <sub>conc. cell</sub> (1 cell.)		1.84	m <sup>2</sup>
$\gamma_{\text{conc.}}$		2.4	t/m <sup>3</sup>
$\gamma_{\text{conc.}}$		1.375	t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)		6.33	t/cell.
S <sub>rubber filling</sub> (1 cell.)		4.16	m <sup>2</sup>
$\gamma_{\text{rubber stone}}$		2.6	t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$		1.575	t/m <sup>3</sup>
void ratio		37%	
W <sub>rubber filling</sub> (1 cell.)		8.26	t/cell.
W <sub>conc. cover side</sub>		1.79	t/m

Upper Layer	Nos. of cellular block	3.00	unit
	$X_{O1}$	4.50	m
	$M_{O1}$	220.97	tm/m

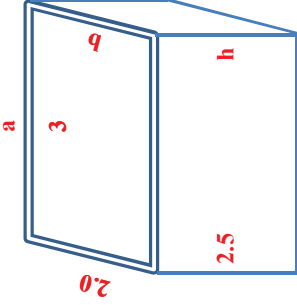
Wave	$P_{\text{up to bottom Elev.}}$	8.52	t/m
	$\gamma_{p \text{ upto bottom Elev.}}$	1.258	m
	$M_{p \text{ at } O2}$	10.71	t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

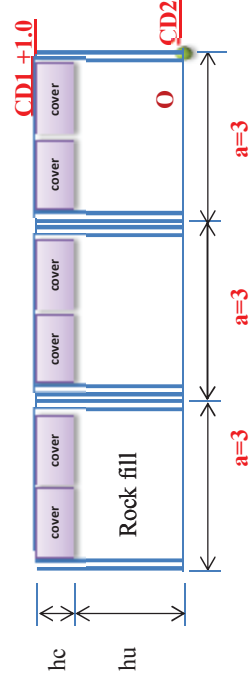
**I. Wave Force under Wave Crest**

**1. Input data:**

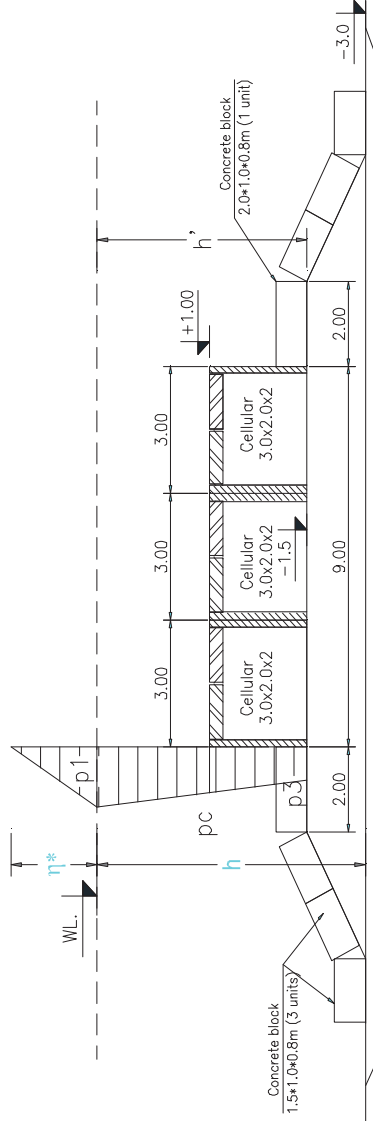
- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	1.0
- Bottom Upright Wall Level (CDL):	-1.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-3.0
- Significant Wave Height ( $H_{1/3}$ ), m:	4.08
- $H_{max}$ , m:	5.365
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	90.82



Sketch of detail of super structure  
HWL



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	60.793
$H_D$ (m):	0.488
L (m):	5.365
HWL (CDL):	90.82
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-3.0
h (m):	1.0
h' (m):	6.55
$\rho_0$ (t/m <sup>3</sup> ):	5.05
g (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	4.906
$\lambda$ :	0.453
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	5.99
$\alpha_1 =$	0.983
$\alpha_3 =$	0.927
$(1 + \cos \beta)$	1.488
$\alpha_1 \lambda \rho_0 H_D$	5.405
$p_1$ (t/m <sup>2</sup> ) =	4.02
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	3.87
$p_3$ (t/m <sup>2</sup> ) =	3.73

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	9.50
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
Wconc. cell. (total), t/m	21.87
Wconc. cover (total), t/m	5.36

Sliding **S.F. at bottom Elev. = 2.29**

Overturnin **Mp at O = 11.95**

**Mconc. cell. at O2 (=W\*t) = 220.97**

**S.F. at O = 18.49**

CD1	30 years	+1	m
CD2		-1.5	m
HWL		+3.55	m
a		3	m
h		2.5	m
hu		2.00	m
b		2.00	m
t		0.20	m
hc		0.50	m
a inside		2.6	m
Sconc. cell (1 cell.)		1.84	m <sup>2</sup>
$\gamma_{\text{conc.}}$		2.4	t/m <sup>3</sup>
$\gamma_{\text{conc.}}$		1.375	t/m <sup>3</sup>
W conc. cell (1 cell.)		6.33	t/cell.
S rubber filling (1 cell.)		4.16	m <sup>2</sup>
$\gamma_{\text{rubber stone}}$		2.6	t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$		1.575	t/m <sup>3</sup>
void ratio		37%	
W rubber filling (1 cell.)		8.26	t/cell.
W conc. cover side		1.79	t/m

Upper Layer	Nos. of cellular block	3.00	unit
	$X_{O1}$	4.50	m
	$M_{O1}$	220.97	tm/m

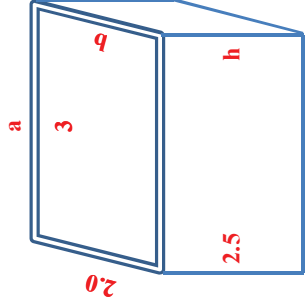
Wave	$P_{\text{up to bottom Elev.}}$	9.50	t/m
	$\gamma_p$ upto bottom Elev.	1.258	m
	$M_{p \text{ at } O2}$	11.95	t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

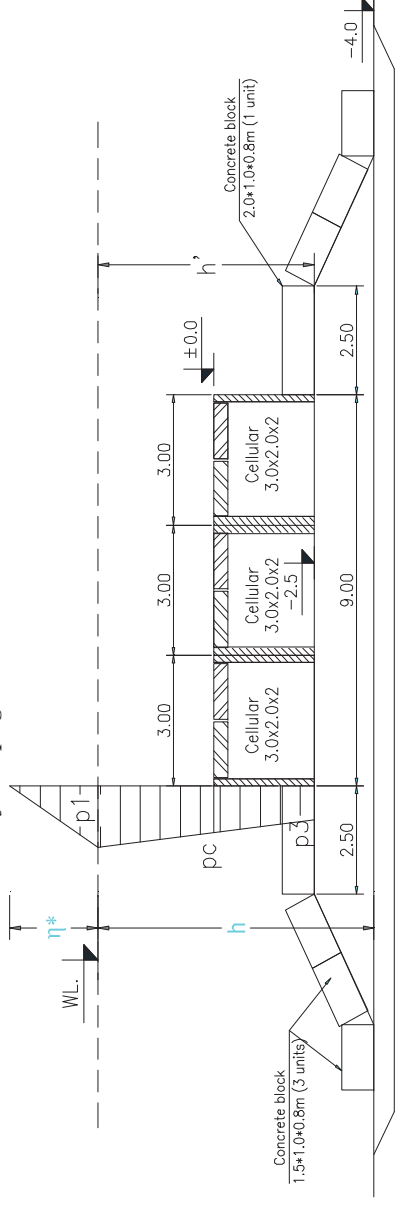
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	0.43
- Crest Level (CDL):	0.0
- Bottom Upright Wall Level (CDL):	-2.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-4.0
- Significant Wave Height ( $H_{1/3}$ ), m:	4.636
- $H_{max}$ , m:	6.035
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	96.05



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	60.926
$H_D$ (m):	0.486
L (m):	6.035
HWL (CDL):	96.05
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-4.0
h (m):	0.0
h' (m):	4.43
$\rho_0$ (t/m <sup>3</sup> ):	2.93
g (m/s <sup>2</sup> ):	1.025
4 $\pi h / L$ :	9.81
2 $\pi h / L$ :	0.580
$\lambda$ :	0.290
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	6.73
$\alpha_1 =$	1.048
$\alpha_3 =$	0.973
(1 + $\cos \beta$ )	1.486
$\alpha_1 \lambda \rho_0 H_D$	6.480
$p_1$ (t/m <sup>2</sup> ) =	4.81
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	4.80
$p_3$ (t/m <sup>2</sup> ) =	4.69

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	11.85
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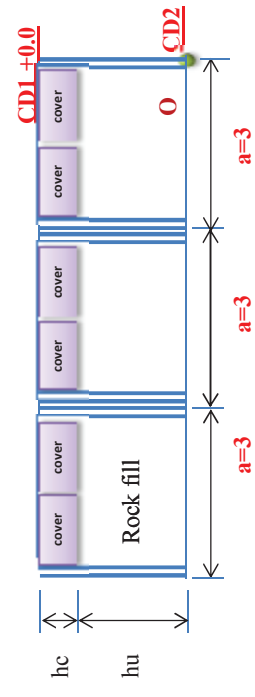
**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W <sub>con. cell.</sub> (total), t/m	21.87
W <sub>conc. cover</sub> (total), t/m	5.36

Sliding	S.F. at bottom Elev.	1.84
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Overturnin g	$M_{p \text{ at } O}$	14.87
	$M_{\text{conc. cell. at } O2} (=W*t)$	220.97
	S.F. at O	14.86

Sketch of detail of super structure  
HWL



CD1	30 years	+0	m
CD2		-2.5	m
HWL		+0.43	m
a		3	m
h		2.5	m
hu		2.00	m
b		2.00	m
t		0.20	m
hc		0.50	m
a inside		2.6	m
S <sub>conc. cell</sub> (1 cell.)		1.84	m <sup>2</sup>
$\gamma_{\text{conc.}}$		2.4	t/m <sup>3</sup>
$\gamma_{\text{conc.}}$		1.375	t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)		6.33	t/cell.
S <sub>rubber filling</sub> (1 cell.)		4.16	m <sup>2</sup>
$\gamma_{\text{rubber stone}}$		2.6	t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$		1.575	t/m <sup>3</sup>
void ratio		37%	
W <sub>rubber filling</sub> (1 cell.)		8.26	t/cell.
W <sub>conc. cover side</sub>		1.79	t/m

Upper Layer	Nos. of cellular block	3.00	unit
	X <sub>O1</sub>	4.50	m
	M <sub>O1</sub>	220.97	tm/m

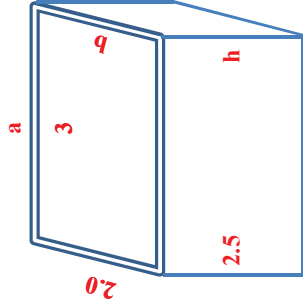
Wave	P <sub>up to bottom Elev.</sub>	11.85	t/m
	$\gamma_p$ upto bottom Elev.	1.255	m
	M <sub>p at O2</sub>	14.87	t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

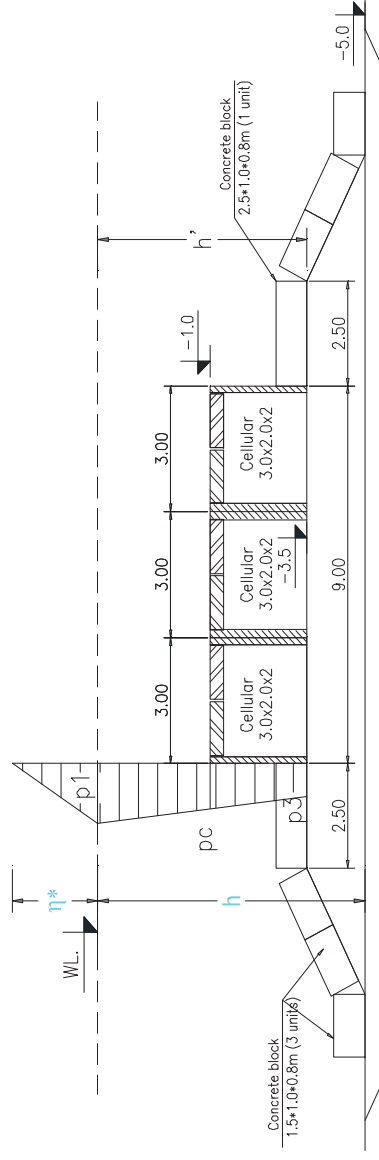
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-1.0
- Bottom Upright Wall Level (CDL):	-3.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-5.0
- Significant Wave Height ( $H_{1/3}$ ), m:	4.96
- $H_{max}$ , m:	6.615
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	100.31



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{h} \eta^* \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	60.947
$H_D$ (m):	0.486
L (m):	6.615
HWL (CDL):	100.31
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-5.0
h (m):	-1.0
h' (m):	8.55
$\rho_0$ (t/m <sup>3</sup> ):	7.05
g (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	4.905
$\lambda$ :	0.536
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	7.37
$\alpha_1 =$	0.946
$\alpha_3 =$	0.894
$(1 + \cos \beta)$	1.486
$\alpha_1 \lambda \rho_0 H_D$	6.413
$p_1$ (t/m <sup>2</sup> ) =	4.76
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	4.44
$p_3$ (t/m <sup>2</sup> ) =	4.26

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	10.87
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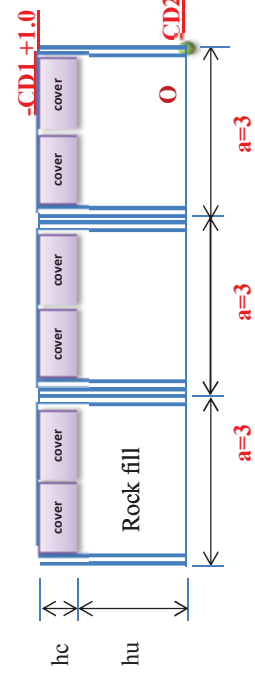
**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
Wconc. cell. (total), t/m	21.87
Wconc. cover (total), t/m	5.36

Sliding	S.F. at bottom Elev.	2.00
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Overturnin g	$M_{p \text{ at } O}$	13.69
	$M_{\text{conc. cell. at } O2} (=W*t)$	220.97
	S.F. at O	16.15

Sketch of detail of super structure  
HWL



CD1	30 years	-1 m
CD2		-3.5 m
HWL		+ 3.55 m
a		3 m
h		2.5 m
hu		2.00 m
b		2.00 m
t		0.20 m
hc		0.50 m
a inside		2.6 m
$S_{\text{conc. cell (1 cell.)}}$		1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$		2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$		1.375 t/m <sup>3</sup>
W conc. cell (1 cell.)		6.33 t/cell.
$S_{\text{rubber filling (1 cell.)}}$		4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$		2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$		1.575 t/m <sup>3</sup>
void ratio		37%
W rubber filling (1 cell.)		8.26 t/cell.
W conc. cover side		1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	$X_{O1}$	4.50 m
	$M_{O1}$	220.97 tm/m

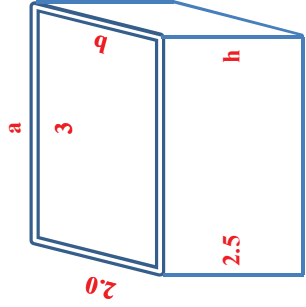
Wave	$P_{\text{up to bottom Elev.}}$	10.87 t/m
	$\gamma_{p \text{ upto bottom Elev.}}$	1.259 m
	$M_{p \text{ at } O2}$	13.69 t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OC DI)

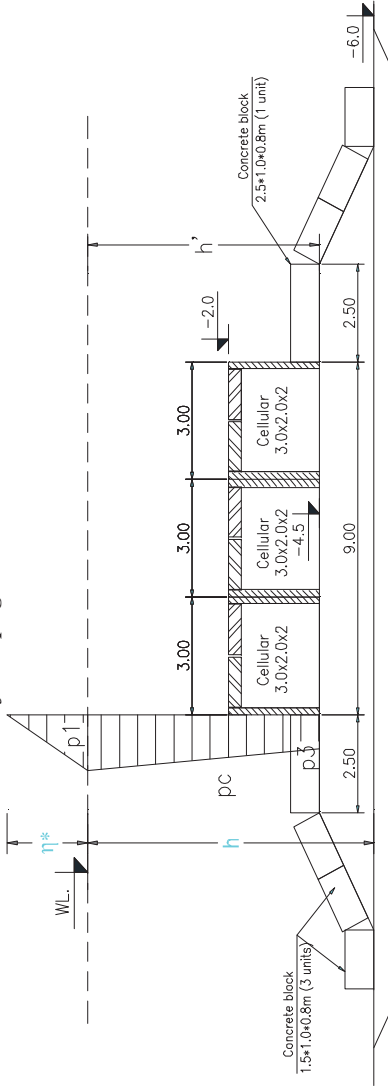
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-2.0
- Bottom Upright Wall Level (CDL):	-4.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-6.0
- Significant Wave Height ( $H_{1/3}$ ), m:	5.01
- $H_{max}$ , m:	7.308
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	105.13



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	60.772
$H_D$ (m):	0.488
L (m):	7.308
HWL (CDL):	105.13
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-6.0
h (m):	-2.0
h' (m):	9.55
$\rho_0$ (t/m <sup>3</sup> ):	8.05
g (m/s <sup>2</sup> ):	1.025
4 $\pi h / L$ :	9.81
2 $\pi h / L$ :	1.142
$\lambda$ :	0.571
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	8.16
$\alpha_1 =$	0.930
$\alpha_3 =$	0.879
(1+cos $\beta$ )	1.488
$\alpha_1 \lambda \rho_0 H_D$	6.963
$p_1$ (t/m <sup>2</sup> ) =	5.18
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	4.75
$p_3$ (t/m <sup>2</sup> ) =	4.56

**5.3. Total wave pressure on Upright Wall**

$P_{up}$ to bottom Elev. (t/m) =	11.63
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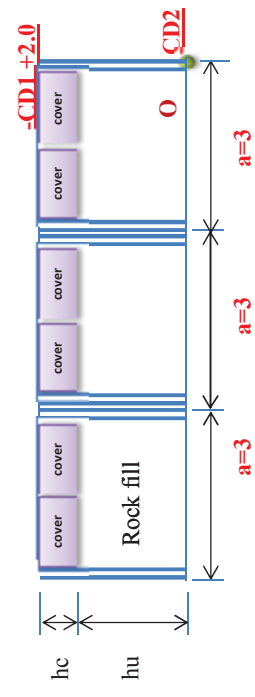
**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W <sub>con. cell.</sub> (total), t/m	21.87
W <sub>conc. cover</sub> (total), t/m	5.36

Sliding	S.F. at bottom Elev.	1.87
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Overturnin g	$M_{p,at O}$	14.64
	$M_{conc. cell. at O2} (=W*t)$	220.97
	S.F. at O	15.09

Sketch of detail of super structure HWL



CD1	30 years	-2 m
CD2		-4.5 m
HWL		+ 3.55 m
a		3 m
h		2.5 m
hu		2.00 m
b		2.00 m
t		0.20 m
hc		0.50 m
a inside		2.6 m
S <sub>conc. cell</sub> (1 cell.)		1.84 m <sup>2</sup>
$\gamma_{conc.}$		2.4 t/m <sup>3</sup>
$\gamma_{conc.}$		1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)		6.33 t/cell.
S <sub>rubber filling</sub> (1 cell.)		4.16 m <sup>2</sup>
$\gamma_{rubber stone}$		2.6 t/m <sup>3</sup>
$\gamma_{rubber stone}$		1.575 t/m <sup>3</sup>
void ratio		37%
W <sub>rubber filling</sub> (1 cell.)		8.26 t/cell.
W <sub>conc. cover side</sub>		1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	X <sub>O1</sub>	4.50 m
	M <sub>O1</sub>	220.97 tm/m

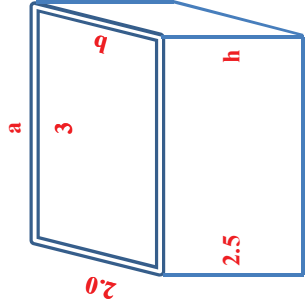
Wave	P <sub>up</sub> to bottom Elev.	11.63 t/m
	$\gamma_p$ upto bottom Elev.	1.259 m
	M <sub>p</sub> at O2	14.64 t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

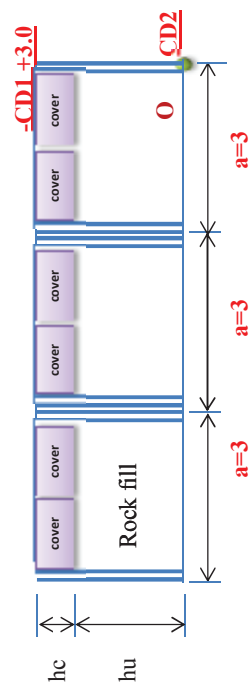
**I. Wave Force under Wave Crest**

**1. Input data:**

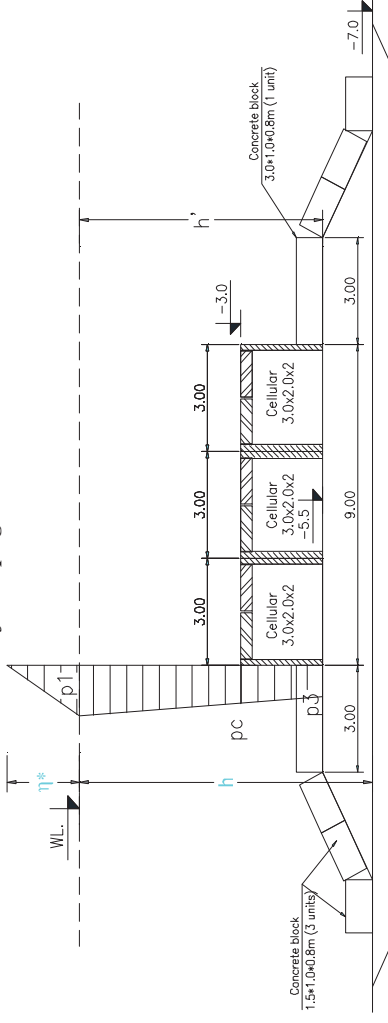
- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-3.0
- Bottom Upright Wall Level (CDL):	-5.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-7.0
- Significant Wave Height ( $H_{1/3}$ ), m:	5.067
- $H_{max}$ , m:	7.931
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	109.23



Sketch of detail of super structure  
HWL



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0gH_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	60.21
$H_D$ (m):	0.497
$L$ (m):	7.931
HWL (CDL):	109.23
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-7.0
$h$ (m):	-3.0
$h'$ (m):	10.55
$\rho_0$ (t/m <sup>3</sup> ):	9.05
$g$ (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	4.905
$\lambda$ :	0.607
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	8.90
$\alpha_1 =$	0.913
$\alpha_3 =$	0.863
$(1 + \cos \beta)$	1.497
$\alpha_1 \lambda \rho_0 H_D$	7.421
$p_1$ (t/m <sup>2</sup> ) =	5.55
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	5.00
$p_3$ (t/m <sup>2</sup> ) =	4.79

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to bottom Elev.}}$ (t/m) =	12.25
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
$W_{con. \text{ cell.}}$ (total), t/m	21.87
$W_{conc. \text{ cover}}$ (total), t/m	5.36

Sliding **S.F. at bottom Elev.** **1.78**

$M_{p \text{ at } O}$	15.42
$M_{conc. \text{ cell. at } O2}$ (=W*t)	220.97
<b>S.F. at O</b>	<b>14.33</b>

CD1	30 years
CD2	-3 m
HWL	-5.5 m
a	+ 3.55 m
h	3 m
hu	2.5 m
hu	2.00 m
b	2.00 m
t	0.20 m
hc	0.50 m
a inside	2.6 m
$S_{conc. \text{ cell}}$ (1 cell.)	1.84 m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
$\gamma_{conc.}$	1.375 t/m <sup>3</sup>
$W_{conc. \text{ cell}}$ (1 cell.)	6.33 t/cell.
$S_{rubber \text{ filling}}$ (1 cell.)	4.16 m <sup>2</sup>
$\gamma_{rubber \text{ stone}}$	2.6 t/m <sup>3</sup>
$\gamma_{rubber \text{ stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
$W_{rubber \text{ filling}}$ (1 cell.)	8.26 t/cell.
$W_{conc. \text{ cover side}}$	1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	$X_{O1}$	4.50 m
	$M_{O1}$	220.97 tm/m

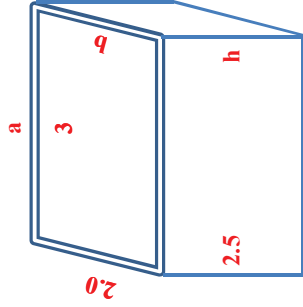
Wave	$P_{up \text{ to bottom Elev.}}$	12.25 t/m
	$\gamma_p$ upto bottom Elev.	1.259 m
	$M_{p \text{ at } O2}$	15.42 t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

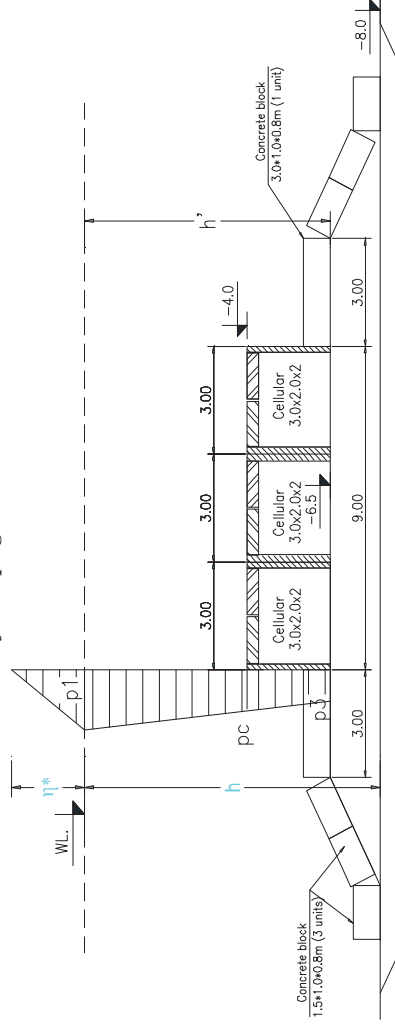
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-4.0
- Bottom Upright Wall Level (CDL):	-6.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-8.0
- Significant Wave Height ( $H_{1/3}$ ), m:	5.096
- $H_{max}$ , m:	8.44
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	112.43



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	60.174
$H_D$ (m):	0.497
L (m):	8.44
HWL (CDL):	112.43
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-8.0
h (m):	-4.0
h' (m):	11.55
$\rho_0$ (t/m <sup>3</sup> ):	10.05
g (m/s <sup>2</sup> ):	1.025
4 $\pi h / L$ :	9.81
2 $\pi h / L$ :	1.291
$\lambda$ :	0.645
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	9.48
$\alpha_1 =$	0.895
$\alpha_3 =$	0.846
$(1 + \cos \beta)$	1.497
$\alpha_1 \lambda \rho_0 H_D$	7.743
$p_1$ (t/m <sup>2</sup> ) =	5.80
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	5.12
$p_3$ (t/m <sup>2</sup> ) =	4.90

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	12.53
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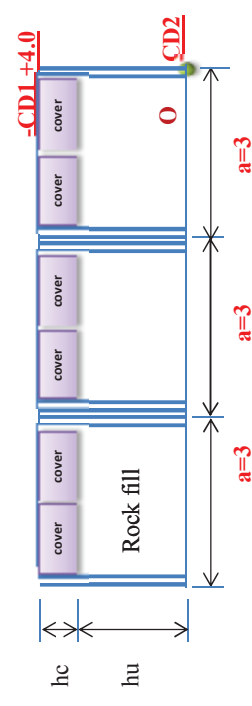
**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
Wconc. cell. (total), t/m	21.87
Wconc. cover (total), t/m	5.36

Sliding	S.F. at bottom Elev.	1.74
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Overturning	$M_{p \text{ at } O}$	15.78
g	$M_{\text{conc. cell. at } O2} (=W*t)$	220.97
	S.F. at O	14.00

Sketch of detail of super structure  
HWL



CD1	30 years
CD2	-4 m
HWL	-6.5 m
a	+ 3.55 m
h	3 m
hu	2.5 m
hu	2.00 m
b	2.00 m
t	0.20 m
hc	0.50 m
a inside	2.6 m
S <sub>conc. cell (1 cell.)</sub>	1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$	1.375 t/m <sup>3</sup>
W <sub>conc. cell (1 cell.)</sub>	6.33 t/cell.
S <sub>rubber filling (1 cell.)</sub>	4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$	2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling (1 cell.)</sub>	8.26 t/cell.
W <sub>conc. cover side</sub>	1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	$X_{O1}$	4.50 m
	$M_{O1}$	220.97 tm/m

Wave	$P_{\text{up to bottom Elev.}}$	12.53 t/m
	$\gamma_{p \text{ upto bottom Elev.}}$	1.259 m
	$M_{p \text{ at } O2}$	15.78 t.m/m

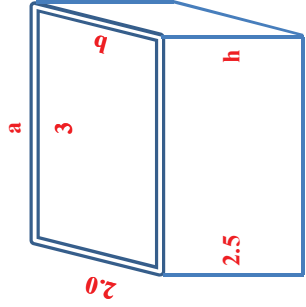


**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

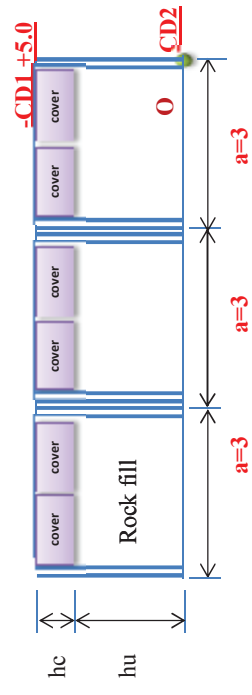
**I. Wave Force under Wave Crest**

**1. Input data:**

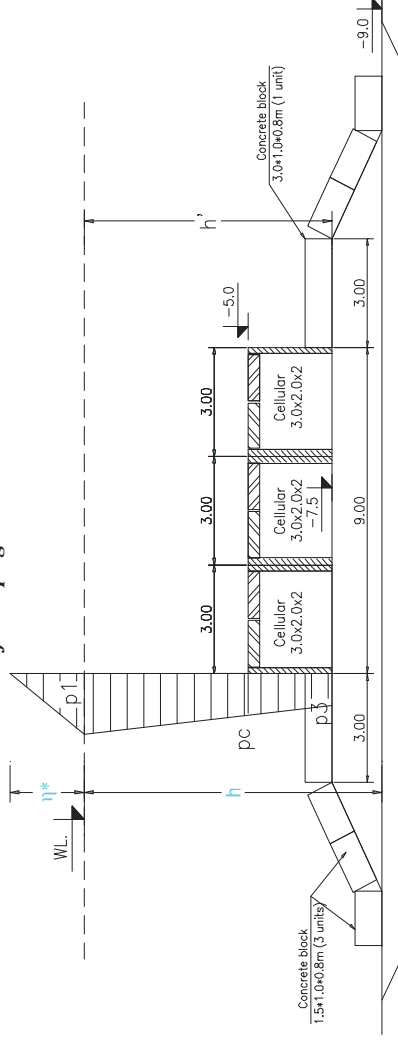
- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-5.0
- Bottom Upright Wall Level (CDL):	-7.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-9.0
- Significant Wave Height ( $H_{1/3}$ ), m:	5.143
- $H_{max}$ , m:	8.522
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	112.94



Sketch of detail of super structure  
HWL



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	59.967
$H_D$ (m):	0.500
$L$ (m):	8.522
HWL (CDL):	112.94
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-9.0
$h$ (m):	-5.0
$h'$ (m):	12.55
$\rho_0$ (t/m <sup>3</sup> ):	11.05
$g$ (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	1.396
$\lambda$ :	0.698
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	9.59
$\alpha_1 =$	0.871
$\alpha_3 =$	0.822
$(1 + \cos \beta)$	1.500
$\alpha_1 \lambda \rho_0 H_D$	7.609
$p_1$ (t/m <sup>2</sup> ) =	5.71
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	4.92
$p_3$ (t/m <sup>2</sup> ) =	4.69

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	12.02
$\mu$ (friction factor between conc. cell. stone & stone):	0.80
$W_{\text{con. cell.}}$ (total), t/m	21.87
$W_{\text{conc. cover}}$ (total), t/m	5.36

**II. Safety Factor**

Sliding	S.F. at bottom Elev.	1.8
Overturnin g	$M_{p \text{ at } O}$	15.14
	$M_{\text{conc. cell. at } O2}$ (=W*t)	220.97
	S.F. at O	14.60

CD1	30 years
CD2	-5 m
HWL	-7.5 m
a	+ 3.55 m
h	3 m
hu	2.5 m
hu	2.00 m
b	2.00 m
t	0.20 m
hc	0.50 m
a inside	2.6 m
$S_{\text{conc. cell}}$ (1 cell.)	1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$	1.375 t/m <sup>3</sup>
$W_{\text{conc. cell}}$ (1 cell.)	6.33 t/cell.
$S_{\text{rubber filling}}$ (1 cell.)	4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$	2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
$W_{\text{rubber filling}}$ (1 cell.)	8.26 t/cell.
$W_{\text{conc. cover side}}$	1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	$X_{O1}$	4.50 m
	$M_{O1}$	220.97 tm/m

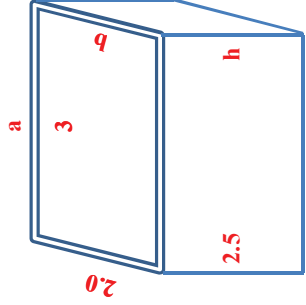
Wave	$P_{\text{up to bottom Elev.}}$	12.02 t/m
	$\gamma_{p \text{ upto bottom Elev.}}$	1.260 m
	$M_{p \text{ at } O2}$	15.14 t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

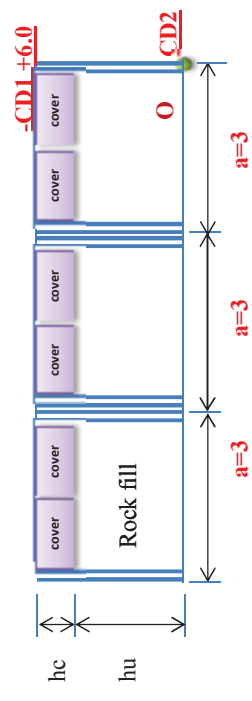
**I. Wave Force under Wave Crest**

**1. Input data:**

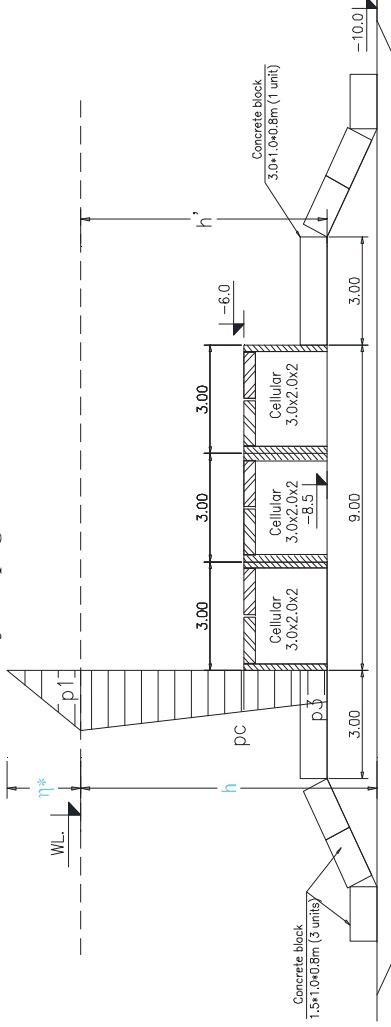
- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	-6.0
- Bottom Upright Wall Level (CDL):	-8.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-10.0
- Significant Wave Height ( $H_{1/3}$ ), m:	5.058
- $H_{max}$ , m:	8.629
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	113.60



Sketch of detail of super structure  
HWL



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{h} \eta^* \quad (\text{if } -h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	60.118
$H_D$ (m):	0.498
L (m):	8.629
HWL (CDL):	113.60
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-10.0
h (m):	-6.0
h' (m):	13.55
$\rho_0$ (t/m <sup>3</sup> ):	12.05
g (m/s <sup>2</sup> ):	1.025
4 $\pi h / L$ :	9.81
2 $\pi h / L$ :	4.999
$\lambda$ :	0.749
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	9.70
$\alpha_1 =$	0.848
$\alpha_3 =$	0.798
$(1 + \cos \beta)$	1.498
$\alpha_1 \lambda \rho_0 H_D$	7.504
$p_1$ (t/m <sup>2</sup> ) =	5.62
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	4.72
$p_3$ (t/m <sup>2</sup> ) =	4.48

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	11.51
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
Wconc. cell. (total), t/m	21.87
Wconc. cover (total), t/m	5.36

Sliding	S.F. at bottom Elev.	1.89
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Overturning	$M_{p \text{ at } O}$	14.51
	$M_{\text{conc. cell. at } O2} (=W*t)$	220.97
	S.F. at O	15.23

CD1	30 years	-6 m
CD2		-8.5 m
HWL		+ 3.55 m
a		3 m
h		2.5 m
hu		2.00 m
b		2.00 m
t		0.20 m
hc		0.50 m
a inside		2.6 m
S <sub>conc. cell (1 cell.)</sub>		1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$		2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$		1.375 t/m <sup>3</sup>
W <sub>conc. cell (1 cell.)</sub>		6.33 t/cell.
S <sub>rubber filling (1 cell.)</sub>		4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$		2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$		1.575 t/m <sup>3</sup>
void ratio		37%
W <sub>rubber filling (1 cell.)</sub>		8.26 t/cell.
W <sub>conc. cover side</sub>		1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	X <sub>O1</sub>	4.50 m
	M <sub>O1</sub>	220.97 tm/m

Wave	P <sub>up to bottom Elev.</sub>	11.51 t/m
	$\gamma_p$ upto bottom Elev.	1.261 m
	M <sub>p at O2</sub>	14.51 t.m/m

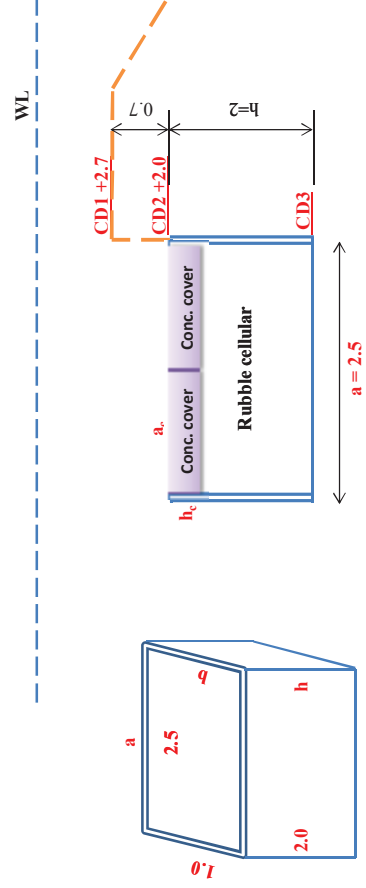
**10 STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - CASE A-1 (LWL)**

STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)  
Sand Protection Training Dike - Section for GL upto CD 0.0m

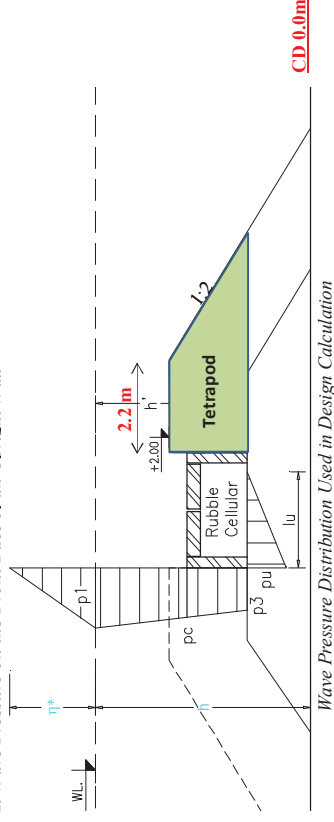
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	0.43
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	0.0
- Bottom Width of Upright Wall (m):	2.5
- Back Fill Level (CDL):	2.7
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	0.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	0.93
- H <sub>max</sub> , m:	3.14
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	70.13



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} (\text{if } -h' \leq 0)$$

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

3. Uplift beneath Upright Wall

$$P_3 = P_u = \alpha_3 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	32.35
H <sub>D</sub> (m):	0.845
L (m):	3.14
LWL (CDL):	70.13
Sea bed Level (CDL):	0.43
Crest Level (CDL):	0.0
h (m):	2.0
h' (m):	0.43
ρ <sub>0</sub> (t/m <sup>3</sup> ):	0.43
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	0.077
λ:	0.039
	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	4.34
α <sub>1</sub> =	1.099
α <sub>3</sub> =	0.999
(1 + cos β)	1.845
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	3.537
P <sub>1</sub> (t/m <sup>2</sup> ) =	3.26
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	3.27
P <sub>3</sub> (t/m <sup>2</sup> ) =	3.26

5.3. Uplift beneath Upright Wall

P <sub>3</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	6.53
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	9.10
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	9.43
M <sub>pp</sub> , t.m	6.29

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	5.97
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	9.10

Sliding	S.F. (Normal)	0.73
	S.F. (with back fill layer effect)	1.71
	Mu	0.00
	Mp	6.54
Overtopping	Mtetrapod	6.29
	Mconc. (=W*t)	7.47
	S.F. (Normal)	1.14
	S.F. (with back fill layer effect)	29.82

	30 years
CD1	+ 2.7 m
CD2	+ 2 m
CD3	0 m
LWL	+ 0.43 m
a	2.5 m
h	2 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.00 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.24 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc.</sub>	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	3.41 t/m
W <sub>conc. cover</sub> (1 unit)	0.34 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.26 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	1.88 t/m
ΣW	5.97 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	2.2 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	13.23 m <sup>2</sup>
Wtetrapod	9.10 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.0
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	9.43
Pp, t/m	9.43
γp, m	0.67
M <sub>pp</sub> , t.m	6.29

Back fill beside conc. Block

Rubble Cell	x <sub>sc</sub>	1.25 m
	M (Sc)	7.47 t.m/m

P	6.53 t/m
U	0.00 t/m
γ <sub>p</sub>	1.001 m
x <sub>u</sub>	2.500 m
Mp	6.54 t.m/m
Mu	0.00 t.m/m
M total	6.54 t.m/m

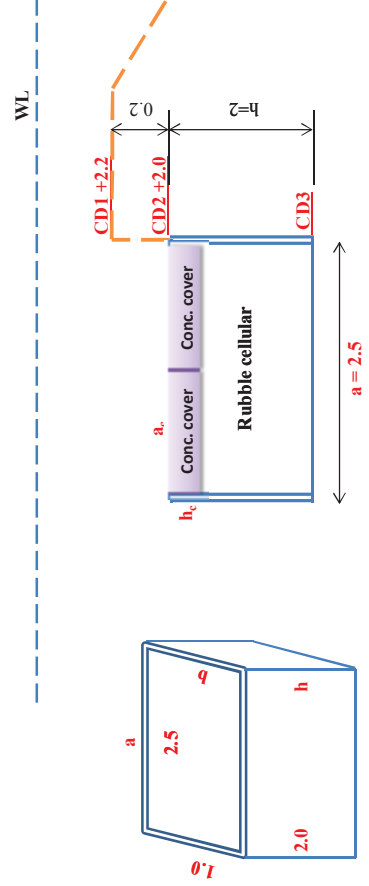
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -1.0m

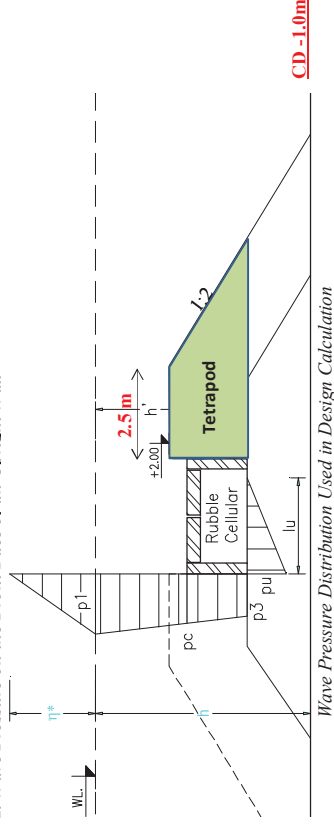
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	0.0
- Bottom Width of Upright Wall (m):	2.5
- Back Fill Level (CDL):	2.2
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-1.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	1.48
- H <sub>max</sub> , m:	3.727
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	76.21



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_4 = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (if \quad h' \leq 0)$$

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

3. Uplift beneath Upright Wall

$$P_5 = P_4 = \alpha_3 P_1$$

$$I_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

breaking zone

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	57.021
H <sub>D</sub> (m):	0.544
L (m):	3.727
LWL (CDL):	76.21
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-1.0
h (m):	2.0
h' (m):	4.55
h (m):	3.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
4 πh/L:	0.750
2 πh/L:	0.375
λ:	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	4.32
α <sub>1</sub> =	1.016
α <sub>3</sub> =	0.948
(1 + cos β)	1.544
α <sub>3</sub> λ ρ <sub>0</sub> H <sub>D</sub>	3.881
P <sub>1</sub> (t/m <sup>2</sup> ) =	3.00
P <sub>2</sub> (at crest level), (t/m <sup>2</sup> ) =	2.93
P <sub>3</sub> (t/m <sup>2</sup> ) =	2.84

5.3. Uplift beneath Upright Wall

P <sub>5</sub> (t/m <sup>2</sup> ) =	0.00
I <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	5.77
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	7.11
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	9.43
M <sub>pp</sub> , t.m	6.29

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	5.97
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	7.11

S.F. (Normal)	0.83
S.F. (with back fill layer effect)	1.69

Mu	0.00
Mp	5.80
Mtetrapod	6.29
Mconc. (=W*t)	7.47
S.F. (Normal)	1.29
S.F. (with back fill layer effect)	OK

CD1	30 years
CD2	+ 2.2 m
CD3	+ 2 m
LWL	0 m
a	+ 3.55 m
h	2.5 m
b	2 m
t	1 m
h <sub>c</sub>	0.20 m
a <sub>c</sub>	0.50 m
b <sub>c</sub>	1.00 m
S <sub>conc. cell</sub> (1 cell)	0.50 m
γ <sub>conc.</sub>	1.24 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cover</sub> (1 unit)	3.41 t/m
NOS. of cover block	0.34 t/m
S <sub>rubber filling</sub> (1 cell)	2.00
γ <sub>rubber stone</sub>	1.26 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	1.88 t/m
ΣW	5.97 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	2.5 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	10.34 m <sup>2</sup>
Wtetrapod	7.11 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.0
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	9.43
Pp, t/m	9.43
γp, m	0.67
M <sub>pp</sub> , t.m	6.29

Rubble Cell	x <sub>sc</sub>	1.25 m
	M (Sc)	7.47 t.m/m

P	5.77 t/m
U	0.00 t/m
γ <sub>p</sub>	1.005 m
x <sub>u</sub>	2.500 m
Mp	5.80 t.m/m
Mu	0.00 t.m/m
M total	5.80 t.m/m

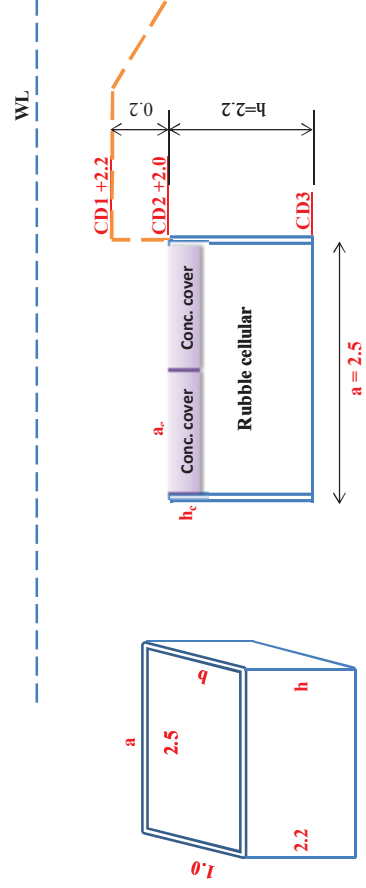
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -2.0m

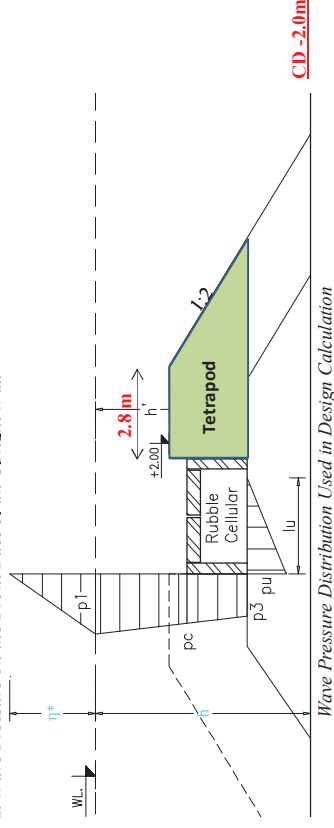
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-0.2
- Bottom Width of Upright Wall (m):	2.5
- Back Fill Level (CDL):	2.2
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-2.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	1.952
- H <sub>max</sub> , m:	2.87
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	67.12



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_w g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (if \quad h' \leq 0)$$

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

3. Uplift beneath Upright Wall

$$P_3 = P_u = \alpha_3 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

breaking zone

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	65.468
H <sub>D</sub> (m):	0.415
L (m):	2.87
LWL (CDL):	67.12
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-2.0
h (m):	2.0
h' (m):	5.55
h' (m):	3.75
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
4 πh/L:	1.039
2 πh/L:	0.520
λ:	0.999

5.2. Wave pressure on the front face of an Upright wall

η* =	3.04
α <sub>1</sub> =	0.953
α <sub>3</sub> =	0.918
(1 + cos β)	1.415
α <sub>3</sub> λ ρ <sub>0</sub> H <sub>D</sub>	2.801
P <sub>1</sub> (t/m <sup>2</sup> ) =	1.98
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	1.92
P <sub>3</sub> (t/m <sup>2</sup> ) =	1.82

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	4.11
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	8.58
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	11.41
M <sub>pp</sub> , t.m	8.37

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	6.56
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	8.58

Sliding	S.F. (Normal)	1.28
	S.F. (with back fill layer effect)	2.74

	Mu	0.00
	Mp	4.56
Overturning	Mtetrapod	8.37
	Mconc. (=W*t)	8.20
	S.F. (Normal)	1.80
	S.F. (with back fill layer effect)	OK

	30 years
CD1	+2.2 m
CD2	+2 m
CD3	-0.2 m
LWL	+3.55 m
a	2.5 m
h	2.2 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.00 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.24 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cover</sub> (1 unit)	3.75 t/m
W <sub>conc. cover</sub> (1 unit)	0.34 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.26 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	2.13 t/m
ΣW	6.56 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	2.8 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	12.48 m <sup>2</sup>
Wtetrapod	8.58 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.2
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	10.37
Pp, t/m	11.41
γp, m	0.73
Mpp, t.m	8.37

Rubble Cell	x <sub>sc</sub>	1.25 m
	M (Sc)	8.20 t.m/m

P	4.11 t/m
U	0.00 t/m
γ <sub>p</sub>	1.109 m
x <sub>u</sub>	2.500 m
Mp	4.56 t.m/m
Mu	0.00 t.m/m
M total	4.56 t.m/m

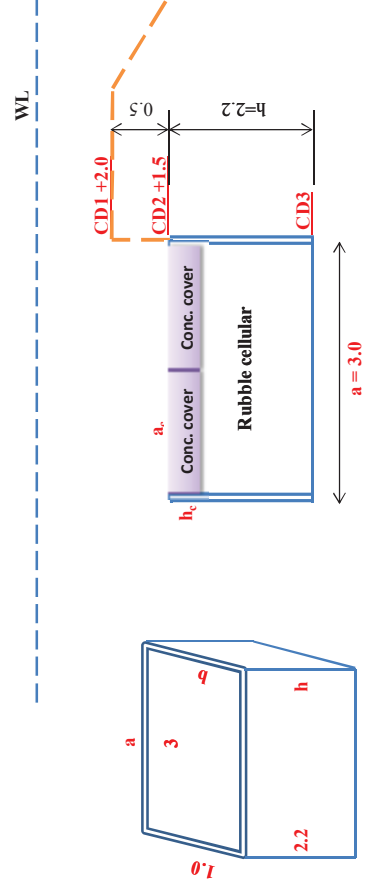
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -3.0m

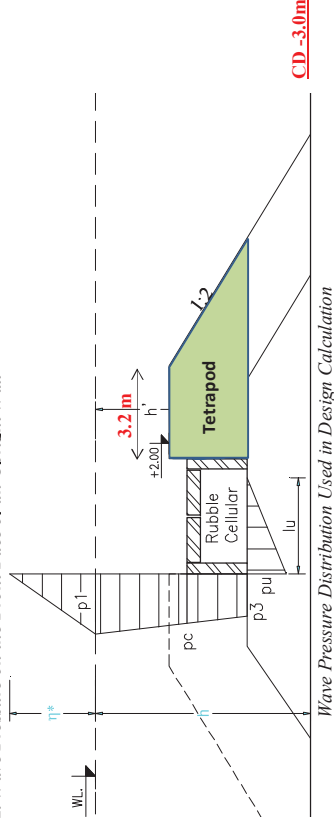
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	1.5
- Bottom Upright Wall Level (CDL):	-0.7
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	2.0
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-3.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	2.427
- H <sub>max</sub> , m:	3.343
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	72.30



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (if \quad h' \leq 0)$$

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

3. Uplift beneath Upright Wall

$$P_3 = P_u = \alpha_3 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

breaking zone

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	61.332
H <sub>D</sub> (m):	0.480
L <sub>w</sub> (m):	3.343
LWL (CDL):	72.30
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-3.0
h (m):	1.5
h' (m):	6.55
h' (m):	4.25
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
4 πh/L:	1.138
2 πh/L:	0.569
λ:	0.999

5.2. Wave pressure on the front face of an Upright wall

η* =	3.70
α <sub>1</sub> =	0.930
α <sub>3</sub> =	0.907
(1 + cos β)	1.480
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	3.183
P <sub>1</sub> (t/m <sup>2</sup> ) =	2.35
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	2.25
P <sub>3</sub> (t/m <sup>2</sup> ) =	2.14

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	4.83
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	10.95
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	11.41
M <sub>pp</sub> , t.m	8.37

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	7.85
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	10.95

Sliding	S.F. (Normal)	1.30
	S.F. (with back fill layer effect)	2.89

	Mu	0.00
	Mp	5.35
Overturning	Mtetrapod	8.37
	Mconc. (=W*t)	11.77
	S.F. (Normal)	2.20
	S.F. (with back fill layer effect)	OK

	30 years
CD1	+2 m
CD2	+1.5 m
CD3	-0.7 m
LWL	+3.55 m
a	3 m
h	2.2 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.44 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cover</sub> (1 unit)	4.36 t/m
W <sub>conc. cover</sub> (1 unit)	0.43 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	2.63 t/m
ΣW	7.85 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	3.2 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	15.93 m <sup>2</sup>
Wtetrapod	10.95 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.2
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	10.37
Pp, t/m	11.41
γp, m	0.73
Mpp, t.m	8.37

Back fill beside conc. Block

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	11.77 t.m/m

P	4.83 t/m
U	0.00 t/m
γ <sub>p</sub>	1.109 m
x <sub>u</sub>	3.000 m
Mp	5.35 t.m/m
Mu	0.00 t.m/m
M total	5.35 t.m/m

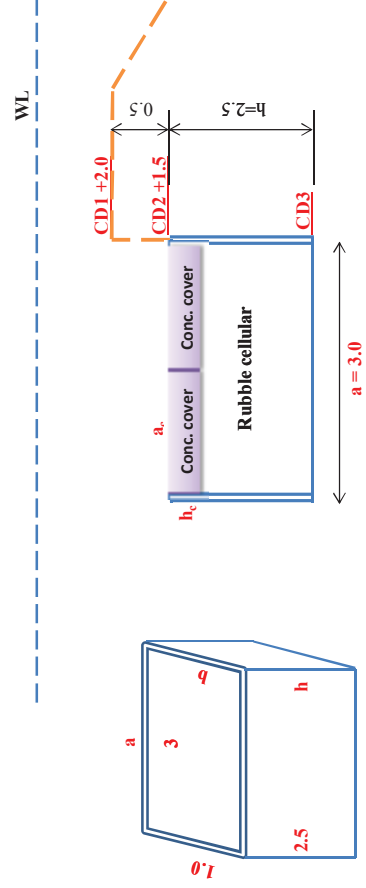
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDD)

Sand Protection Training Dike - Section for GL upto CD -4.0m

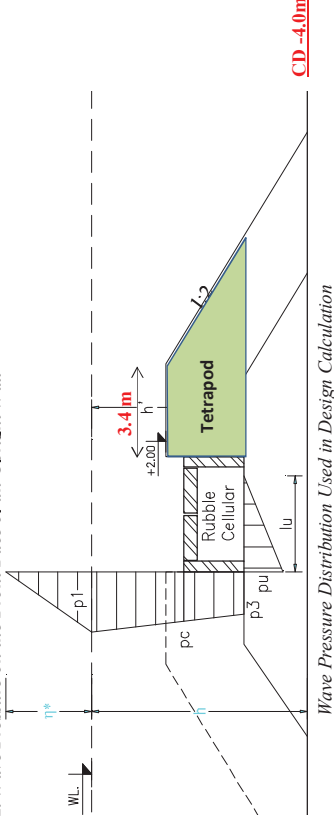
I. Wave Force under Wave Crest

I. Input data:

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	1.5
- Bottom Upright Wall Level (CDL):	-1.0
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	2.0
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-4.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	2.985
- H <sub>max</sub> , m:	4.012
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	78.98



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_4 = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (if \quad h' \leq 0)$$

3. Uplift beneath Upright Wall

$$P_5 = P_6 = \alpha_3 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

breaking zone

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.936
H <sub>D</sub> (m):	0.486
L (m):	4.012
LWL (CDL):	78.98
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-4.0
h (m):	1.5
h' (m):	7.55
h' (m):	4.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
4 πh/L:	1.201
2 πh/L:	0.601
λ:	0.997

5.2. Wave pressure on the front face of an Upright wall

η* =	4.46
α <sub>1</sub> =	0.916
α <sub>5</sub> =	0.906
(1 + cos β)	1.486
α <sub>3</sub> λ ρ <sub>0</sub> H <sub>D</sub>	3.755
P <sub>1</sub> (t/m <sup>2</sup> ) =	2.79
P <sub>6</sub> (at crest level), (t/m <sup>2</sup> ) =	2.67
P <sub>3</sub> (t/m <sup>2</sup> ) =	2.53

5.3. Uplift beneath Upright Wall

P <sub>5</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	6.50
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	13.20
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	14.73
M <sub>pp</sub> , t.m	12.28

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	8.91
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	13.20

Sliding	S.F. (Normal)	1.10
	S.F. (with back fill layer effect)	2.52

	Mu	0.00
	Mp	8.19
Overturning	Mtetrapod	12.28
	Mconc. (=W*t)	13.36
	S.F. (Normal)	1.63
	S.F. (with back fill layer effect)	OK

	30 years
CD1	+2 m
CD2	+1.5 m
CD3	-1 m
LWL	+3.55 m
a	3 m
h	2.5 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.44 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub>	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	4.95 t/m
W <sub>conc. cover</sub> (1 unit)	0.43 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	3.10 t/m
ΣW	8.91 t/m

Vertical Effect	
b <sub>back fill layer</sub>	3.4 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	19.20 m <sup>2</sup>
Witetrapod	13.20 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.5
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
γp, m	0.83
M <sub>pp</sub> , t.m	12.28

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	13.36 t.m/m

P	6.50 t/m
U	0.00 t/m
γ <sub>h</sub>	1.262 m
x <sub>u</sub>	3.000 m
Mp	8.19 t.m/m
Mu	0.00 t.m/m
M total	8.19 t.m/m



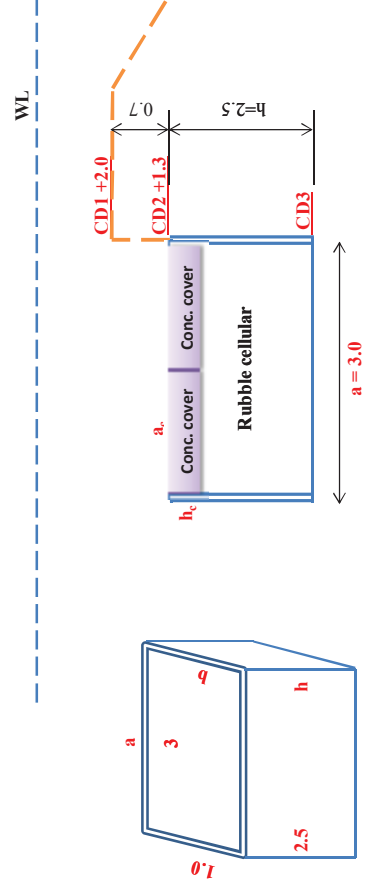
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -5.0m

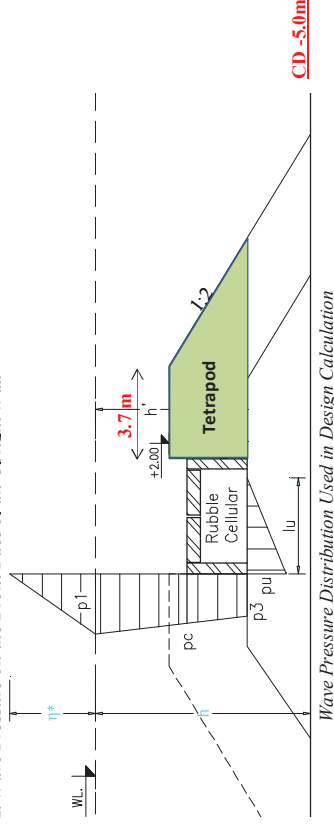
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	1.3
- Bottom Upright Wall Level (CDL):	-1.2
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	2.0
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-5.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	3.457
- H <sub>max</sub> , m:	4.595
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	84.32



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_4 = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

3. Uplift beneath Upright Wall

$P_5 = P_6 = \alpha_3 P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

H<sub>D</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.681
H <sub>D</sub> (m):	0.490
L (m):	4.595
LWL (CDL):	84.32
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-5.0
h (m):	1.3
h' (m):	8.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	4.75
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.274
λ:	0.637
	0.994

5.2. Wave pressure on the front face of an Upright wall

η* =	5.11
α <sub>1</sub> =	0.899
α <sub>3</sub> =	0.904
(1 + cos β)	1.490
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	4.210
P <sub>1</sub> (t/m <sup>2</sup> ) =	3.14
P <sub>5</sub> (at crest level), (t/m <sup>2</sup> ) =	2.99
P <sub>3</sub> (t/m <sup>2</sup> ) =	2.83

5.3. Uplift beneath Upright Wall

P <sub>5</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	7.28
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	15.18
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	14.73
M <sub>pp</sub> , t.m	12.28

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	8.91
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	15.18

S.F. (Normal)	0.98
S.F. (with back fill layer effect)	2.44

Mu	0.00
Mp	9.19
Mtetrapod	12.28
Mconc. (=W*t)	13.36
S.F. (Normal)	1.45
S.F. (with back fill layer effect)	OK

CD1	30 years
CD2	+2 m
CD3	+1.3 m
LWL	-1.2 m
a	+3.55 m
h	3 m
b	2.5 m
t	1 m
h <sub>c</sub>	0.20 m
a <sub>c</sub>	0.50 m
b <sub>c</sub>	1.25 m
S <sub>conc. cell</sub> (1 cell)	0.50 m
γ <sub>conc.</sub>	1.44 t/m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cover</sub> (1 unit)	4.95 t/m
NOS. of cover block	0.43 t/m
S <sub>rubber filling</sub> (1 cell)	2.00
γ <sub>rubber stone</sub>	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
W <sub>rubber filling</sub> (1 cell)	37%
ΣW	3.10 t/m
	8.91 t/m

Vertical Effect	
b <sub>back fill layer</sub>	3.7 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Wtetrapod	22.08 m <sup>2</sup>
Wtetrapod	15.18 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.5
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
γp, m	0.83
Mpp, t.m	12.28

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	13.36 t.m/m

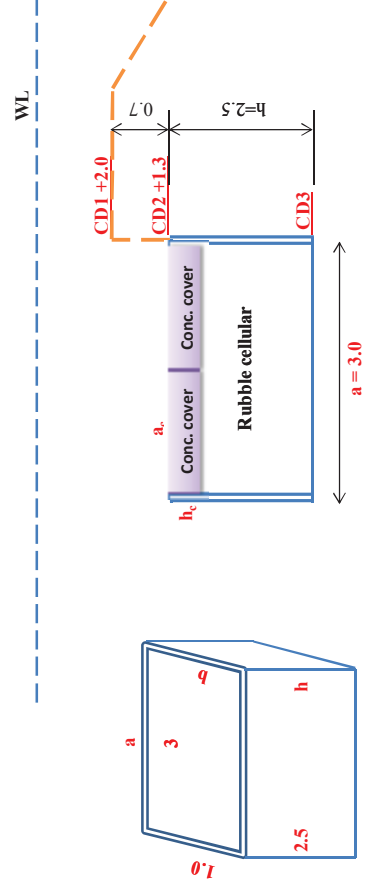
P	7.28 t/m
U	0.00 t/m
γ <sub>p</sub>	1.261 m
x <sub>u</sub>	3.000 m
Mp	9.19 t.m/m
Mu	0.00 t.m/m
M total	9.19 t.m/m

**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDD)**  
**Sand Protection Training Dike - Section for GL upto CD -6.0m**

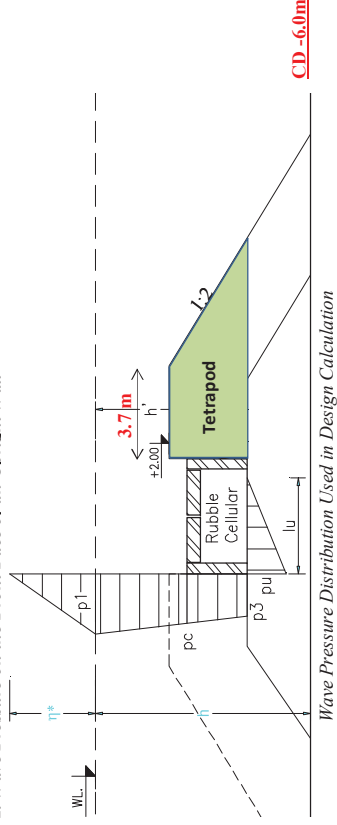
**I. Wave Force under Wave Crest**

**I. Input data:**

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	1.3
- Bottom Upright Wall Level (CDL):	-1.2
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	2.0
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-6.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.036
- H <sub>max</sub> , m:	5.296
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	88.23



**2. Wave Pressure on the Front Face of an Upright Wall**



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_w g H_D$

At toe of upright wall:  $P_3 = P_4 = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (\text{if } -h' \leq 0)$$

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

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

**3. Uplift beneath Upright Wall**

$$P_5 = P_6 = \alpha_3 P_1$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**  
*H<sub>D</sub> = H<sub>max</sub>*  
*breaking zone*

**5. Results of Calculation**

**5.1. Input data**

β (deg) =	30 years
cos β:	60.37
H <sub>D</sub> (m):	0.494
L (m):	5.296
LWL (CDL):	88.23
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-6.0
h (m):	1.3
h' (m):	9.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	4.75
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.360
λ:	0.680
	0.989

**5.2. Wave pressure on the front face of an Upright wall**

η* =	5.87
α <sub>1</sub> =	0.879
α <sub>3</sub> =	0.904
(1 + cos β)	1.494
α <sub>3</sub> λ ρ <sub>0</sub> H <sub>D</sub>	4.719
P <sub>1</sub> (t/m <sup>2</sup> ) =	3.53
P <sub>6</sub> (at crest level), (t/m <sup>2</sup> ) =	3.36
P <sub>3</sub> (t/m <sup>2</sup> ) =	3.19

**5.3. Uplift beneath Upright Wall**

P <sub>5</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

**5.4. Total wave pressure on Upright Wall**

P (t/m) =	8.19
U (t/m) =	0.00

**II. Effect of Back Fill Tetrapod Block**

**Vertical Effect**

ΣW (t) =	15.18
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**Horizontal Effect (Passive Pressure)**

P <sub>p</sub> , t/m	14.73
M <sub>pp</sub> , t.m	12.28

**III. Safety Factor**

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	8.91
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	15.18

Sliding	S.F. (Normal)	0.87
	S.F. (with back fill layer effect)	2.17
	Mu	0.00
	Mp	10.33
Overturning	Mtetrapod	12.28
	Mconc. (=W*t)	13.36
	S.F. (Normal)	1.29
	S.F. (with back fill layer effect)	OK

	30 years
CD1	+2 m
CD2	+1.3 m
CD3	-1.2 m
LWL	+3.55 m
a	3 m
h	2.5 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.44 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cover</sub> (1 unit)	4.95 t/m
NOS. of cover block	0.43 t/m
S <sub>rubber filling</sub> (1 cell)	2.00
γ <sub>rubber stone</sub>	1.56 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	3.10 t/m
ΣW	8.91 t/m

Vertical Effect	
b <sub>back fill layer</sub>	3.7 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	22.08 m <sup>2</sup>
Witetrapod	15.18 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.5
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
γp, m	0.83
M <sub>pp</sub> , t.m	12.28

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	13.36 t.m/m

P	8.19 t/m
U	0.00 t/m
γ <sub>h</sub>	1.261 m
x <sub>u</sub>	3.000 m
Mp	10.33 t.m/m
Mu	0.00 t.m/m
M total	10.33 t.m/m

**11 STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - CASE A-2 (LWL)**

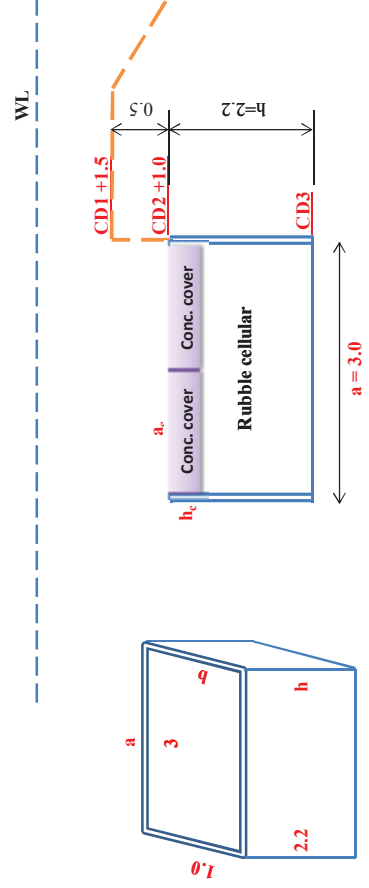
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDD)

Sand Protection Training Dike - Section for GL upto CD -3.0m

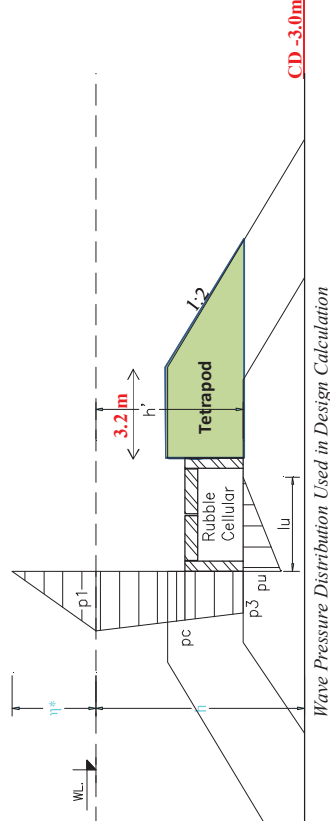
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	1.0
- Bottom Upright Wall Level (CDL):	-1.2
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	1.5
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-3.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	2.427
- H <sub>max</sub> , m:	3.343
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	72.30



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_w g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

3. Uplift beneath Upright Wall

$P_3 = P_a = \alpha_3 P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

$H_D = H_{max}$

breaking zone

5. Results of Calculation

5.1. Input data

$\beta$ (deg) =	30 years
$\cos \beta$ :	61.332
H <sub>D</sub> (m):	0.480
L <sub>w</sub> (m):	3.343
LWL (CDL):	72.30
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-3.0
h (m):	1.0
h' (m):	6.55
$\rho_0$ (t/m <sup>3</sup> ):	4.75
g (m/s <sup>2</sup> ):	9.81
4 $\pi h/L$ :	1.138
2 $\pi h/L$ :	0.569
$\lambda$ :	1.000

5.2. Wave pressure on the front face of an Upright wall

$\eta^*$ =	3.71
$\alpha_1$ =	0.930
$\alpha_3$ =	0.897
(1 + cos $\beta$ )	1.480
$\alpha_1 \lambda \rho_w H_D$	3.186
P <sub>1</sub> (t/m <sup>2</sup> ) =	2.36
P <sub>a</sub> (at crest level), (t/m <sup>2</sup> ) =	2.23
P <sub>3</sub> (t/m <sup>2</sup> ) =	2.11

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	4.77
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

$\Sigma W$ (t) =	10.95
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	11.41
M <sub>pp</sub> , t.m	8.37

III. Safety Factor

$\mu$ (friction factor between rubble cellular & stone):	0.80
W <sub>con.</sub> (total), t/m	7.85
$\mu$ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	10.95

Sliding	S.F. (Normal)	1.31
	S.F. (with back fill layer effect)	2.92

	Mu	0.00
	Mp	5.30
Overturning	Mtetrapod	8.37
	Mconc. (=W*t)	11.77
	S.F. (Normal)	2.22
	S.F. (with back fill layer effect)	OK

	30 years
CD1	+ 1.5 m
CD2	+ 1 m
CD3	-1.20 m
LWL	+ 3.55 m
a	3 m
h	2.2 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.44 m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
$\gamma_{conc. cell}$	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	4.36 t/m
W <sub>conc. cover</sub> (1 unit.)	0.43 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell.)	1.56 m <sup>2</sup>
$\gamma_{rubber stone}$	2.6 t/m <sup>3</sup>
$\gamma_{rubber stone}$	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell.)	2.63 t/m
$\Sigma W$	7.85 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	3.2 m
Slope	2
$\gamma_{Tetrapod}$	2.4 t/m <sup>3</sup>
$\gamma_{Tetrapod}$	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	15.93 m <sup>2</sup>
Witetrapod	10.95 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.2
$\phi$ (deg)	40.0
$\gamma_s$ (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + $\phi/2$ )	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	10.37
Pp, t/m	11.41
Wp, m	0.73
M <sub>pp</sub> , t.m	8.37

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	11.77 t.m/m

P	4.77 t/m
U	0.00 t/m
$\gamma_h$	1.110 m
x <sub>u</sub>	3.000 m
Mp	5.30 t.m/m
Mu	0.00 t.m/m
M total	5.30 t.m/m

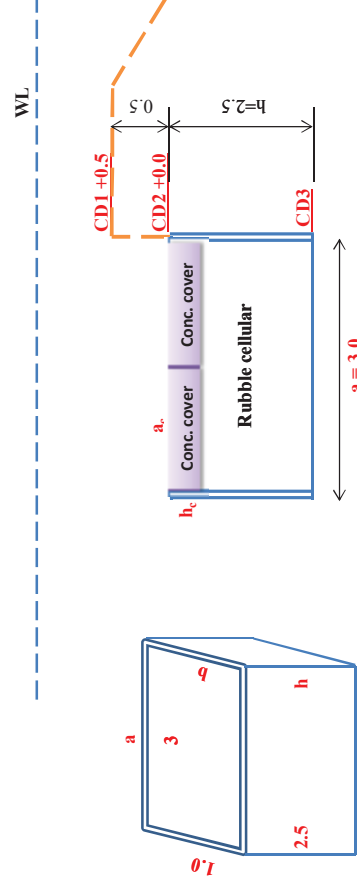
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)

Sand Protection Training Dike - Section for GL upto CD -4.0m

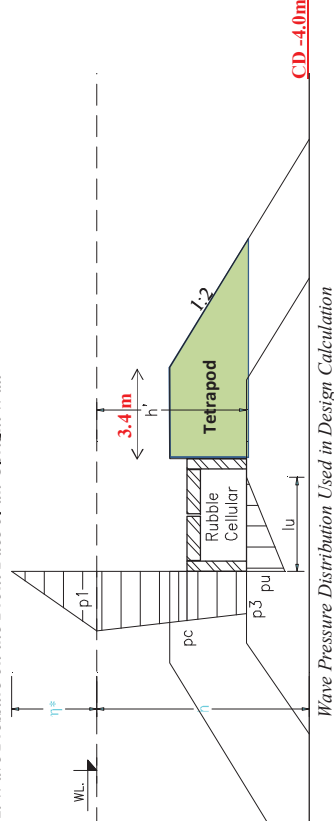
I. Wave Force under Wave Crest

I. Input data:

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	0.0
- Bottom Upright Wall Level (CDL):	-2.5
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	0.5
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-4.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	2.985
- H <sub>max</sub> , m:	4.012
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	78.98



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

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

3. Uplift beneath Upright Wall

$P_3 = P_u = \alpha_3 P_1$

$$I_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

$H_D = H_{max}$

breaking zone

5. Results of Calculation

5.1. Input data

$\beta$ (deg) =	30 years
$\cos \beta$ :	60.936
$H_D$ (m):	0.486
$L$ (m):	4.012
LWL (CDL):	78.98
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-4.0
$h$ (m):	0.0
$h'$ (m):	7.55
$\rho_0$ (t/m <sup>3</sup> ):	6.05
$g$ (m/s <sup>2</sup> ):	1.025
$4 \pi h/L$ :	9.81
$2 \pi h/L$ :	1.201
$\lambda$ :	0.601
	1.000

5.2. Wave pressure on the front face of an Upright wall

$\eta^*$ =	4.47
$\alpha_1$ =	0.916
$\alpha_3$ =	0.874
$(1 + \cos \beta)$	1.486
$\alpha_3 \lambda \rho_0 H_D$	3.765
$P_1$ (t/m <sup>2</sup> ) =	2.80
$P_u$ (at crest level), (t/m <sup>2</sup> ) =	2.59
$P_3$ (t/m <sup>2</sup> ) =	2.45

5.3. Uplift beneath Upright Wall

$P_u$ (t/m <sup>2</sup> ) =	0.00
$I_u$ (m) =	0.00

5.4. Total wave pressure on Upright Wall

$P$ (t/m) =	6.30
$U$ (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

$\Sigma W$ (t) =	13.20
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Horizontal Effect (Passive Pressure)

$P_p$ , t/m	14.73
$M_{pp}$ , t.m	12.28

III. Safety Factor

$\mu$ (friction factor between rubble cellular & stone):	0.80
$W_{conc.}$ (total), t/m	8.91
$\mu$ (friction factor between tetrapod & stone)	0.70
$W_{tetrapod}$ , t/m	13.20

Sliding	S.F. (Normal)	1.13
	S.F. (with back fill layer effect)	2.60

	$M_u$	0.00
	$M_p$	7.95
Overturning	$M_{tetrapod}$	12.28
	$M_{conc. (=W*t)}$	13.36
	S.F. (Normal)	1.68
	S.F. (with back fill layer effect)	OK

	30 years
CD1	+0.5 m
CD2	+0 m
CD3	-2.5 m
LWL	+3.55 m
a	3 m
h	2.5 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.44 m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
$\gamma_{conc. cell}$	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	4.95 t/m
W <sub>conc. cover</sub> (1 unit)	0.43 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.56 m <sup>2</sup>
$\gamma_{rubber stone}$	2.6 t/m <sup>3</sup>
$\gamma_{rubber stone}$	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	3.10 t/m
$\Sigma W$	8.91 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	3.4 m
Slope	2
$\gamma_{Tetrapod}$	2.4 t/m <sup>3</sup>
$\gamma_{Tetrapod}$	1.375 t/m <sup>3</sup>
void ratio	50%
S <sub>tetrapod</sub>	19.20 m <sup>2</sup>
W <sub>tetrapod</sub>	13.20 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.5
$\phi$ (deg)	40.0
$\gamma_s$ (t/m <sup>3</sup> )	1.03
$K_p = \tan^2(45^\circ + \phi/2)$	4.6
$P_p$ (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
$\gamma_p$ , m	0.83
M <sub>pp</sub> , t.m	12.28

Back fill beside conc. Block

Rubble Cell	$x_{sc}$	1.50 m
	M (Sc)	13.36 t.m/m

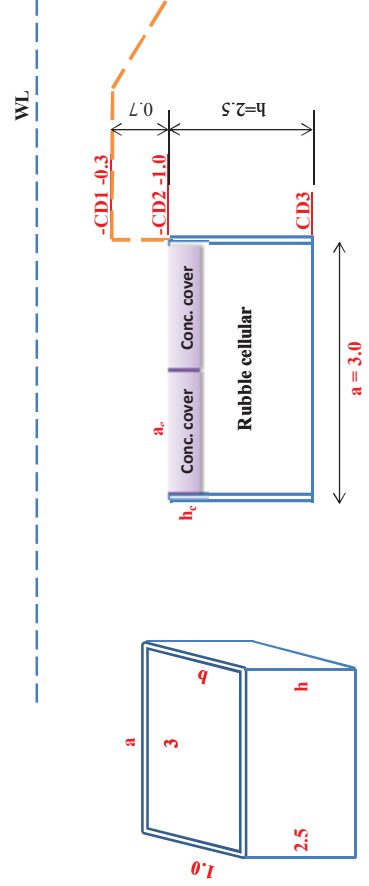
P	6.30 t/m
U	0.00 t/m
$\gamma_h$	1.262 m
$x_u$	3.000 m
$M_p$	7.95 t.m/m
$M_u$	0.00 t.m/m
M total	7.95 t.m/m

STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)  
Sand Protection Training Dike - Section for GL upto CD -5.0m

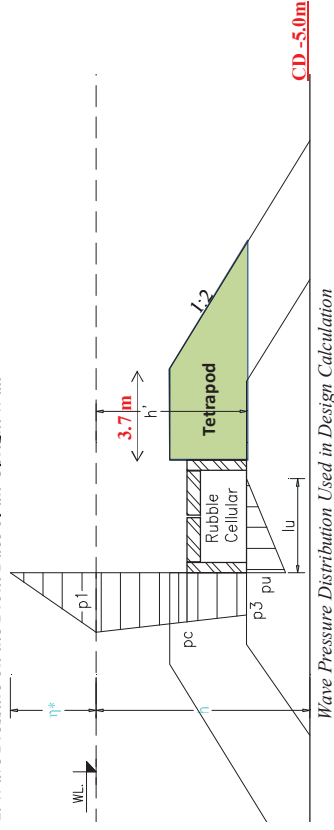
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - 1.WL (CDL):	3.55
- Crest Level (CDL):	-1.0
- Bottom Upright Wall Level (CDL):	-3.5
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	-0.3
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-5.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	3.457
- H <sub>max</sub> , m:	4.595
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	84.32



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

3. Uplift beneath Upright Wall

$P_3 = P_u = \alpha_3 P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation  
breaking zone

H<sub>D</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	60.681
H <sub>D</sub> (m):	0.490
L (m):	4.595
LWL (CDL):	84.32
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-5.0
h (m):	-1.0
h' (m):	8.55
h (m):	7.05
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
4 πh/L:	1.274
2 πh/L:	0.637
λ:	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	5.13
α <sub>1</sub> =	0.899
α <sub>3</sub> =	0.857
(1 + cos β)	1.490
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	4.233
P <sub>1</sub> (t/m <sup>2</sup> ) =	3.15
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	2.86
P <sub>3</sub> (t/m <sup>2</sup> ) =	2.70

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	6.96
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	15.18
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	14.73
M <sub>pp</sub> , t.m	12.28

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>con.</sub> (total), t/m	8.91
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	15.18

Sliding	S.F. (Normal)	1.02
	S.F. (with back fill layer effect)	2.55

	Mu	0.00
	Mp	8.78
Overturning	Mtetrapod	12.28
	Mconc. (=W*t)	13.36
	S.F. (Normal)	1.52
	S.F. (with back fill layer effect)	OK

	30 years
CD1	-0.3 m
CD2	-1 m
CD3	-3.5 m
LWL	+ 3.55 m
a	3 m
h	2.5 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.25 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell (1 cell)</sub>	1.44 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell (1 cell)</sub>	1.375 t/m <sup>3</sup>
W <sub>conc. cell (1 cell)</sub>	4.95 t/m
W <sub>conc. cover (1 unit)</sub>	0.43 t/m
NOS. of cover block	2.00
S <sub>rubble filling (1 cell)</sub>	1.56 m <sup>2</sup>
γ <sub>rubble stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubble stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubble filling (1 cell)</sub>	3.10 t/m
ΣW	8.91 t/m

Vertical Effect	
b <sub>back fill layer</sub>	3.7 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	22.08 m <sup>2</sup>
Witetrapod	15.18 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.5
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
γp, m	0.83
Mpp, t.m	12.28

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	13.36 t.m/m

P	6.96 t/m
U	0.00 t/m
γ <sub>p</sub>	1.262 m
x <sub>u</sub>	3.000 m
Mp	8.78 t.m/m
Mu	0.00 t.m/m
M total	8.78 t.m/m

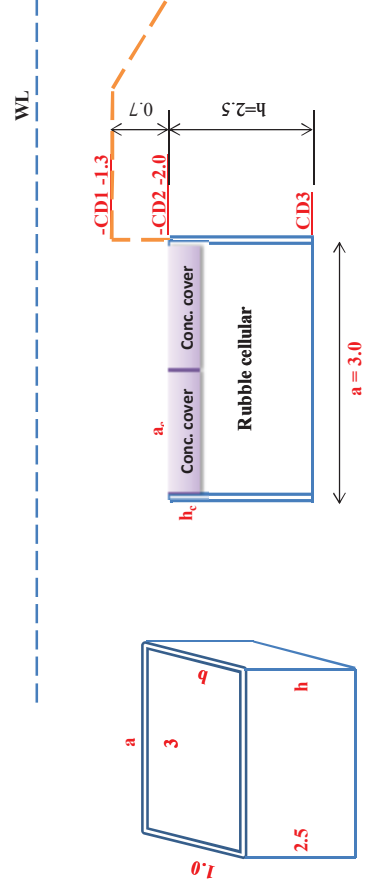
**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)**

**Sand Protection Training Dike - Section for GL upto CD -6.0m**

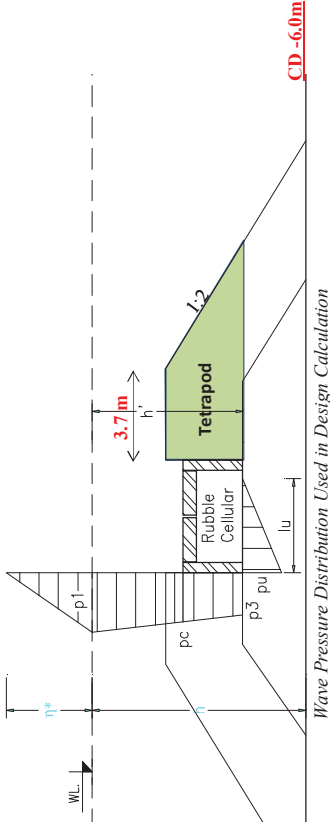
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	-2.0
- Bottom Upright Wall Level (CDL):	-4.5
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	-1.3
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-6.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.036
- H <sub>max</sub> , m:	5.296
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	88.23



**2. Wave Pressure on the Front Face of an Upright Wall**



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} (if \quad h' \leq 0)$$

**3. Uplift beneath Upright Wall**

$$P_5 = P_u = \alpha_5 P_1$$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

H<sub>D</sub> = H<sub>max</sub>  
breaking zone

**5. Results of Calculation**

**5.1. Input data**

β (deg) =	30 years
cos β:	60.37
H <sub>D</sub> (m):	0.494
L (m):	5.296
LWL (CDL):	88.23
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-6.0
h (m):	-2.0
h' (m):	9.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	8.05
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.360
λ:	0.680
	1.000

**5.2. Wave pressure on the front face of an Upright wall**

η* =	5.94
α <sub>1</sub> =	0.879
α <sub>5</sub> =	0.837
(1 + cos β)	1.494
α <sub>5</sub> λ ρ <sub>0</sub> H <sub>D</sub>	4.773
P <sub>1</sub> (t/m <sup>2</sup> ) =	3.57
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	3.16
P <sub>5</sub> (t/m <sup>2</sup> ) =	2.98

**5.3. Uplift beneath Upright Wall**

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

**5.4. Total wave pressure on Upright Wall**

P (t/m) =	7.68
U (t/m) =	0.00

**II. Effect of Back Fill Tetrapod Block**

**Vertical Effect**

ΣW (t) =	15.18
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**Horizontal Effect (Passive Pressure)**

P <sub>p</sub> , t/m	14.73
M <sub>pp</sub> , t.m	12.28

**III. Safety Factor**

μ (friction factor between rubble cellular & stone):	0.80
W <sub>conc.</sub> (total), t/m	8.91
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	15.18

Sliding	S.F. (Normal)	0.93
	S.F. (with back fill layer effect)	2.31

	Mu	0.00
	Mp	9.70
Overturning	Mtetrapod	12.28
	Mconc. (=W*t)	13.36
	S.F. (Normal)	1.38
	S.F. (with back fill layer effect)	OK

	30 years
CD1	-1.3
CD2	-2
CD3	-4.5
LWL	+ 3.55
a	3
h	2.5
b	1
t	0.20
h <sub>c</sub>	0.50
a <sub>c</sub>	1.25
b <sub>c</sub>	0.50
S <sub>conc. cell</sub> (1 cell)	1.44
γ <sub>conc.</sub>	2.4
γ <sub>conc. cell</sub> (1 cell)	1.375
W <sub>conc. cover</sub> (1 unit.)	4.95
W <sub>conc. cover</sub> (1 cell.)	0.43
NOS. of cover block	2.00
S <sub>rubble filling</sub> (1 cell.)	1.56
γ <sub>rubble stone</sub>	2.6
γ <sub>rubble stone</sub>	1.575
void ratio	37%
W <sub>rubble filling</sub> (1 cell.)	3.10
ΣW	8.91

<b>Vertical Effect</b>	
b <sub>back fill layer</sub>	3.7
Slope	2
γ <sub>Tetrapod</sub>	2.4
γ <sub>Tetrapod</sub>	1.375
void ratio	50%
Stetrapod	22.08
Wtetrapod	15.18
<b>Horizontal Effect (Passive Pressure)</b>	
H (m)	2.5
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	11.78
Pp, t/m	14.73
γp, m	0.83
M <sub>pp</sub> , t.m	12.28

Rubble Cell	x <sub>sc</sub>	1.50
	M (Sc)	13.36

	P	7.68
	U	0.00
	γ <sub>h</sub>	1.262
	x <sub>u</sub>	3.000
	Mp	9.70
	Mu	0.00
	M total	9.70

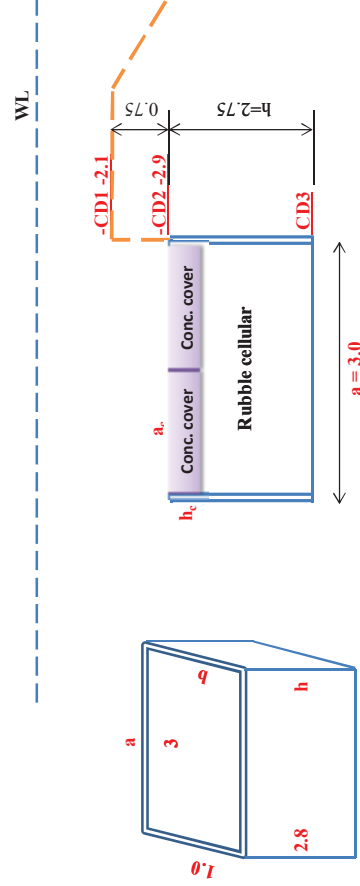
STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)

Sand Protection Training Dike - Section for GL upto CD -7.0m

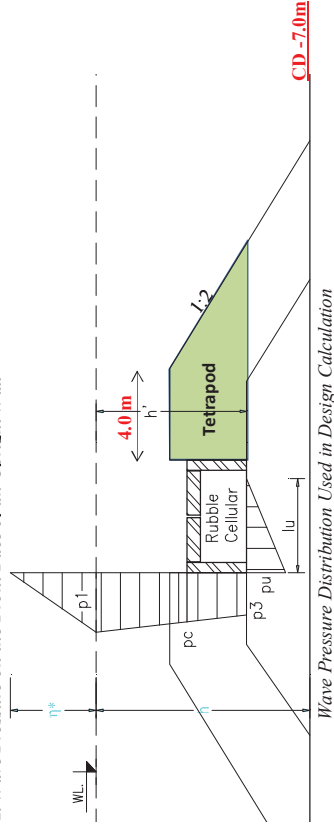
I. Wave Force under Wave Crest

I. Input data:

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	-2.9
- Bottom Upright Wall Level (CDL):	-5.6
- Bottom Width of Upright Wall (m):	3.0
- Back Fill Level (CDL):	-2.1
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-7.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.545
- H <sub>max</sub> , m:	5.922
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	95.19



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

3. Uplift beneath Upright Wall

$P_3 = P_u = \alpha_3 P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation

$H_D = H_{max}$

breaking zone

5. Results of Calculation

5.1. Input data

$\beta$ (deg) =	30 years
$\cos \beta$ :	59.65
H <sub>D</sub> (m):	0.505
L <sub>w</sub> (m):	5.922
LWL (CDL):	95.19
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-7.0
h (m):	-2.9
h' (m):	10.55
$\rho_0$ (t/m <sup>3</sup> ):	9.15
g (m/s <sup>2</sup> ):	1.025
4 $\pi h / L$ :	9.81
2 $\pi h / L$ :	1.393
$\lambda$ :	0.696
	1.000

5.2. Wave pressure on the front face of an Upright wall

$\eta^* =$	6.69
$\alpha_1 =$	0.872
$\alpha_3 =$	0.825
(1 + $\cos \beta$ )	1.505
$\alpha_1 \lambda \rho_0 H_D$	5.292
P <sub>1</sub> (t/m <sup>2</sup> ) =	3.98
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	3.50
P <sub>3</sub> (t/m <sup>2</sup> ) =	3.29

5.3. Uplift beneath Upright Wall

P <sub>u</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	9.33
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

$\Sigma W$ (t) =	18.05
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	17.82
M <sub>pp</sub> , t.m	16.34

III. Safety Factor

$\mu$ (friction factor between rubble cellular & stone):	0.80
W <sub>con.</sub> (total), t/m	9.79
$\mu$ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	18.05

S.F. (Normal)	0.84
S.F. (with back fill layer effect)	2.19

Mu	0.00
Mp	12.96
Mtetrapod	16.34
Mconc. (=W*t)	14.68
S.F. (Normal)	1.13
S.F. (with back fill layer effect)	OK

CD1	30 years
CD2	-2.1 m
CD3	-2.85 m
LWL	-5.6 m
a	+3.55 m
h	3 m
b	2.75 m
t	1 m
h <sub>c</sub>	0.20 m
a <sub>c</sub>	0.50 m
b <sub>c</sub>	1.25 m
S <sub>conc. cell</sub> (1 cell)	0.50 m
$\gamma_{conc.}$	1.44 t/m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cover</sub> (1 unit.)	5.45 t/m
NOS. of cover block	0.43 t/m
S <sub>rubber filling</sub> (1 cell.)	2.00 t/m <sup>2</sup>
$\gamma_{rubber stone}$	1.56 m <sup>2</sup>
$\gamma_{rubber stone}$	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell.)	3.48 t/m
$\Sigma W$	9.79 t/m

Cellular

Conc. Cover

Vertical Effect	
b <sub>back fill layer</sub>	4 m
Slope	2
$\gamma_{tetrapod}$	2.4 t/m <sup>3</sup>
$\gamma_{tetrapod}$	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	26.25 m <sup>2</sup>
Witetrapod	18.05 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.8
$\phi$ (deg)	40.0
$\gamma_s$ (t/m <sup>3</sup> )	1.03
Kp = $\tan^2(45^\circ + \phi/2)$	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	12.96
Pp, t/m	17.82
Wp, m	0.92
Mpp, t.m	16.34

Rubble Cell	x <sub>sc</sub>	1.50 m
	M (Sc)	14.68 t.m/m

P	9.33 t/m
U	0.00 t/m
$\gamma_h$	1.389 m
x <sub>u</sub>	3.000 m
Mp	12.96 t.m/m
Mu	0.00 t.m/m
M total	12.96 t.m/m

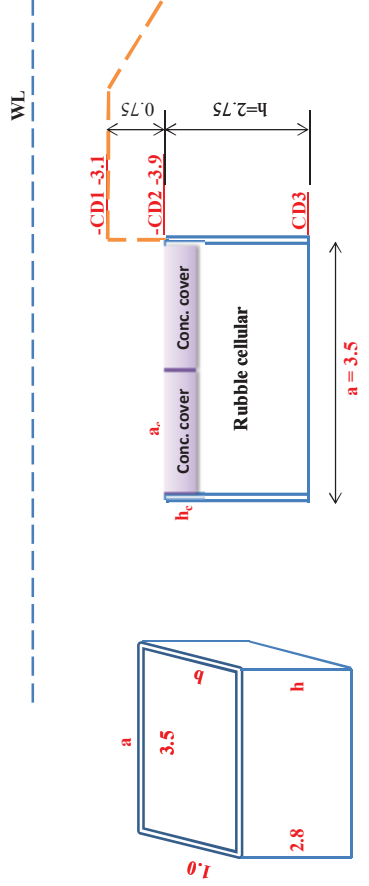


**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
Sand Protection Training Dike - Section for GL upto CD -8.0m

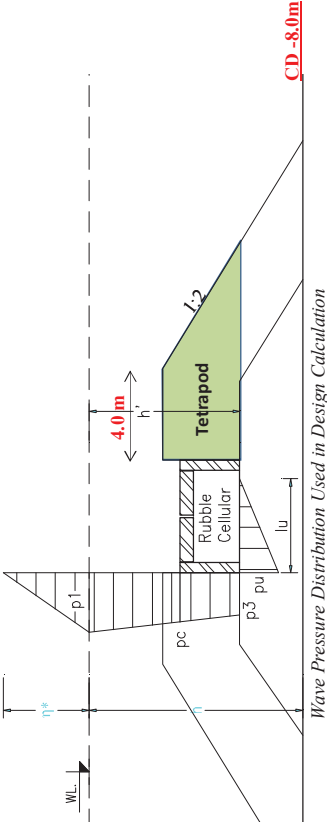
**I. Wave Force under Wave Crest**

**I. Input data:**

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	-3.9
- Bottom Upright Wall Level (CDL):	-6.6
- Bottom Width of Upright Wall (m):	3.5
- Back Fill Level (CDL):	-3.1
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-8.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.956
- H <sub>max</sub> , m:	6.47
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	99.27



**2. Wave Pressure on the Front Face of an Upright Wall**



At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0gH_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

**3. Uplift beneath Upright Wall**

$P_3 = P_a = \alpha_3P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

H<sub>D</sub> = H<sub>max</sub>

**5. Results of Calculation**

**5.1. Input data**

β (deg) =	30 years
cos β:	59.60
H <sub>D</sub> (m):	0.506
L (m):	6.47
LWL (CDL):	99.27
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-8.0
h (m):	-3.9
h' (m):	11.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	10.15
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.462
λ:	0.731
	1.000

**5.2. Wave pressure on the front face of an Upright wall**

η* =	7.31
α <sub>1</sub> =	0.856
α <sub>3</sub> =	0.808
(1+cosβ)	1.506
α <sub>1</sub> λ.ρ <sub>0</sub> .H <sub>D</sub>	5.680
P <sub>1</sub> (t/m <sup>2</sup> ) =	4.28
P <sub>a</sub> (at crest level), (t/m <sup>2</sup> ) =	3.68
P <sub>3</sub> (t/m <sup>2</sup> ) =	3.46

**5.3. Uplift beneath Upright Wall**

P <sub>a</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

**5.4. Total wave pressure on Upright Wall**

P (t/m) =	9.81
U (t/m) =	0.00

**II. Effect of Back Fill Tetrapod Block**

**Vertical Effect**

ΣW (t) =	18.05
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**Horizontal Effect (Passive Pressure)**

P <sub>p</sub> , t/m	17.82
M <sub>pp</sub> , t.m	16.34

**III. Safety Factor**

μ (friction factor between rubble cellular & stone):	0.80
W <sub>con.</sub> (total), t/m	11.39
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	18.05

S.F. (Normal)	0.93
S.F. (with back fill layer effect)	2.22

Mu	0.00
Mp	13.63
Mtetrapod	16.34
Mconc. (=W*t)	19.92
S.F. (Normal)	1.46
S.F. (with back fill layer effect)	OK

CD1	30 years
CD2	-3.1 m
CD3	-3.85 m
LWL	-6.6 m
a	+3.55 m
h	3.5 m
b	2.75 m
t	1 m
h <sub>c</sub>	0.20 m
a <sub>c</sub>	0.50 m
b <sub>c</sub>	1.50 m
S <sub>conc. cell</sub> (1 cell)	0.50 m
S <sub>conc.</sub>	1.64 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cover</sub> (1 unit.)	6.20 t/m
NOS. of cover block	0.52 t/m
S <sub>rubber filling</sub> (1 cell.)	2.00 m <sup>2</sup>
γ <sub>rubber stone</sub>	1.86 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell.)	4.15 t/m
ΣW	11.39 t/m

Vertical Effect	
b <sub>back fill layer</sub>	4 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
W <sub>tetrapod</sub>	26.25 m <sup>2</sup>
W <sub>tetrapod</sub>	18.05 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.8
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45°+φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	12.96
Pp, t/m	17.82
γp, m	0.92
M <sub>pp</sub> , t.m	16.34

Rubble Cell	x <sub>sc</sub>
	M (Sc)
	1.75 m
	19.92 t.m/m

P	9.81 t/m
U	0.00 t/m
γ <sub>h</sub>	1.389 m
x <sub>u</sub>	3.500 m
Mp	13.63 t.m/m
Mu	0.00 t.m/m
M total	13.63 t.m/m

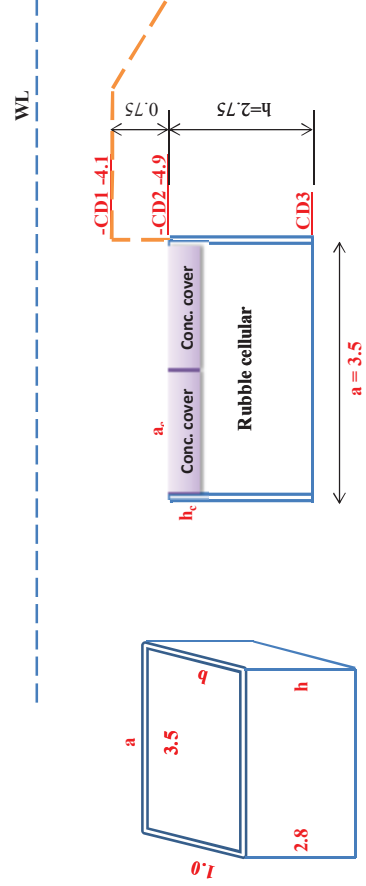
# STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDD)

## Sand Protection Training Dike - Section for GL upto CD -9.0m

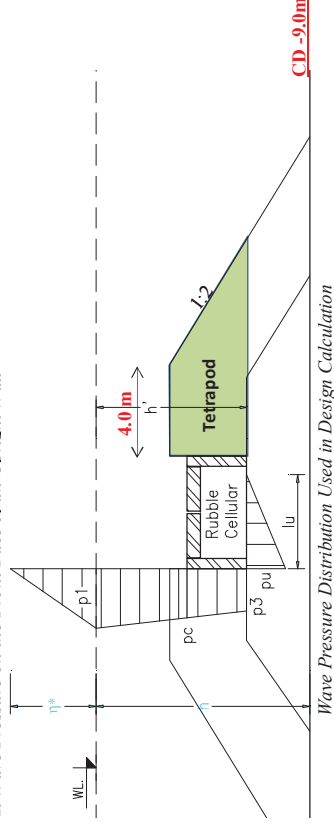
### I. Wave Force under Wave Crest

#### I. Input data:

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	-4.9
- Bottom Upright Wall Level (CDL):	-7.6
- Bottom Width of Upright Wall (m):	3.5
- Back Fill Level (CDL):	-4.1
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-9.0
- Significant Wave Height ( $H_{1/10}$ ), m:	5.022
- $H_{max}$ , m:	7.17
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	104.19



### 2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_w g H_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \right] \left( 1 - \frac{h'}{h} \right)^5$$

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

### 3. Uplift beneath Upright Wall

$P_3 = P_a = \alpha_3 P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

### 4. Wave Height and Wavelength Used in the Wave Pressure Calculation

$H_D = H_{max}$

breaking zone

### 5. Results of Calculation

#### 5.1. Input data

$\beta$ (deg) =	30 years
$\cos \beta$ :	59.43
$H_D$ (m):	0.509
$L$ (m):	7.17
LWL (CDL):	104.19
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-9.0
$h$ (m):	-4.9
$h'$ (m):	12.55
$\rho_b$ (t/m <sup>3</sup> ):	11.15
$g$ (m/s <sup>2</sup> ):	1.025
$4 \pi h / L$ :	9.81
$2 \pi h / L$ :	1.514
$\lambda$ :	0.757
	1.000

#### 5.2. Wave pressure on the front face of an Upright wall

$\eta^* =$	8.11
$\alpha_1 =$	0.845
$\alpha_3 =$	0.795
$(1 + \cos \beta)$	1.509
$\alpha_1 \lambda \rho_b H_D$	6.211
$P_1$ (t/m <sup>2</sup> ) =	4.69
$P_c$ (at crest level), (t/m <sup>2</sup> ) =	3.96
$P_3$ (t/m <sup>2</sup> ) =	3.72

### 5.3. Uplift beneath Upright Wall

$P_u$ (t/m <sup>2</sup> ) =	0.00
$l_u$ (m) =	0.00

### 5.4. Total wave pressure on Upright Wall

$P$ (t/m) =	10.57
$U$ (t/m) =	0.00

### II. Effect of Back Fill Tetrapod Block

#### Vertical Effect

$\Sigma W$ (t) =	18.05
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#### Horizontal Effect (Passive Pressure)

$P_p$ , t/m	17.82
$M_{pp}$ , t.m	16.34

### III. Safety Factor

$\mu$ (friction factor between rubble cellular & stone):	0.80
$W_{con}$ (total), t/m	11.39
$\mu$ (friction factor between tetrapod & stone)	0.70
$W_{tetrapod}$ , t/m	18.05

Sliding	S.F. (Normal)	0.86
	S.F. (with back fill layer effect)	2.06

	Mu	0.00
	Mp	14.68
Overturning	Mtetrapod	16.34
	Mconc. (=W*t)	19.92
	S.F. (Normal)	1.36
	S.F. (with back fill layer effect)	OK

	30 years
CD1	-4.1 m
CD2	-4.85 m
CD3	-7.6 m
LWL	+3.55 m
a	3.5 m
h	2.75 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.50 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.64 m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
$\gamma_{conc.}$	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	6.20 t/m
W <sub>conc. cover</sub> (1 unit)	0.52 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.86 m <sup>2</sup>
$\gamma_{rubber stone}$	2.6 t/m <sup>3</sup>
$\gamma_{rubber stone}$	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	4.15 t/m
$\Sigma W$	11.39 t/m

Vertical Effect	
b <sub>back fill layer</sub>	4 m
Slope	2
$\gamma_{tetrapod}$	2.4 t/m <sup>3</sup>
$\gamma_{tetrapod}$	1.375 t/m <sup>3</sup>
void ratio	50%
Sitetrapod	26.25 m <sup>2</sup>
Witetrapod	18.05 t/m
Horizontal Effect (Passive Pressure)	
H (m)	2.8
$\phi$ (deg)	40.0
$\gamma_s$ (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + $\phi/2$ )	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	12.96
Pp, t/m	17.82
Wp, m	0.92
Mpp, t.m	16.34

Rubble Cell	$x_{sc}$	1.75 m
	M (Sc)	19.92 t.m/m

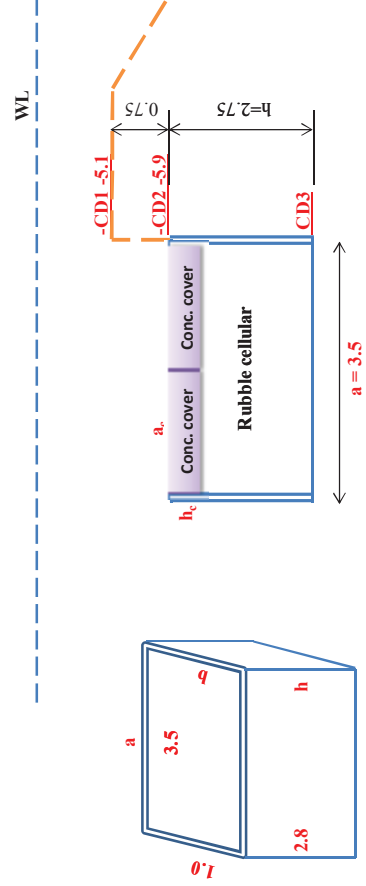
	P	10.57 t/m
	U	0.00 t/m
	$\gamma_h$	1.389 m
	$x_u$	3.500 m
	Mp	14.68 t.m/m
	Mu	0.00 t.m/m
	M total	14.68 t.m/m

STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)  
Sand Protection Training Dike - Section for GL upto CD -10.0m

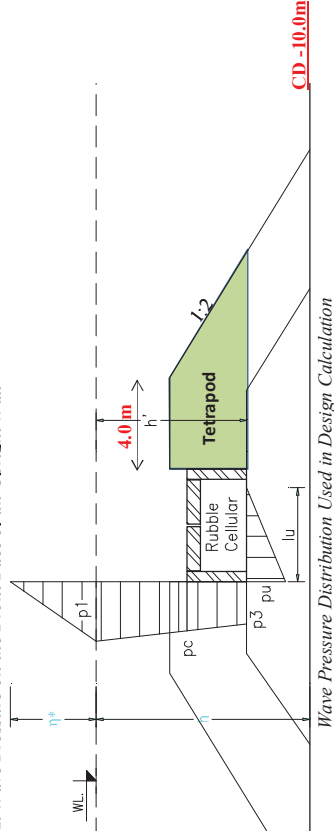
I. Wave Force under Wave Crest

1. Input data:

- Wave Return Period:	30 years
- Tide Level - I.WL (CDL):	3.55
- Crest Level (CDL):	-5.9
- Bottom Upright Wall Level (CDL):	-8.6
- Bottom Width of Upright Wall (m):	3.5
- Back Fill Level (CDL):	-5.1
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-10.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	5.079
- H <sub>max</sub> , m:	7.818
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	108.50



2. Wave Pressure on the Front Face of an Upright Wall



At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0gH_D$

At toe of upright wall:  $P_3 = P_a = \alpha_3P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

3. Uplift beneath Upright Wall

$P_3 = P_a = \alpha_3P_1$

$$l_u = \min \left\{ B, 0.2 \frac{(\eta^* + h')}{|h'|} \right\}$$

4. Wave Height and Wavelength Used in the Wave Pressure Calculation  
breaking zone

H<sub>D</sub> = H<sub>max</sub>

5. Results of Calculation

5.1. Input data

β (deg) =	30 years
cos β:	59.72
H <sub>D</sub> (m):	0.504
L (m):	7.818
LWL (CDL):	108.50
Sea bed Level (CDL):	3.55
Crest Level (CDL):	-10.0
h (m):	-5.9
h' (m):	13.55
ρ <sub>0</sub> (t/m <sup>3</sup> ):	12.15
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.569
λ:	0.785
	1.000

5.2. Wave pressure on the front face of an Upright wall

η* =	8.82
α <sub>1</sub> =	0.833
α <sub>3</sub> =	0.781
(1 + cos β)	1.504
α <sub>1</sub> λρ <sub>0</sub> H <sub>D</sub>	6.677
P <sub>1</sub> (t/m <sup>2</sup> ) =	5.02
P <sub>3</sub> (at crest level), (t/m <sup>2</sup> ) =	4.17
P <sub>3</sub> (t/m <sup>2</sup> ) =	3.92

5.3. Uplift beneath Upright Wall

P <sub>3</sub> (t/m <sup>2</sup> ) =	0.00
l <sub>u</sub> (m) =	0.00

5.4. Total wave pressure on Upright Wall

P (t/m) =	11.12
U (t/m) =	0.00

II. Effect of Back Fill Tetrapod Block

Vertical Effect

ΣW (t) =	18.05
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Horizontal Effect (Passive Pressure)

P <sub>p</sub> , t/m	17.82
M <sub>pp</sub> , t.m	16.34

III. Safety Factor

μ (friction factor between rubble cellular & stone):	0.80
W <sub>con.</sub> (total), t/m	11.39
μ (friction factor between tetrapod & stone)	0.70
W <sub>tetrapod</sub> , t/m	18.05

S.F. (Normal)	0.82
S.F. (with back fill layer effect)	1.95

Mu	0.00
Mp	15.45
Mtetrapod	16.34
Mconc. (=W*t)	19.92
S.F. (Normal)	1.29
S.F. (with back fill layer effect)	OK

	30 years
CD1	-5.1 m
CD2	-5.85 m
CD3	-8.6 m
LWL	+3.55 m
a	3.5 m
h	2.75 m
b	1 m
t	0.20 m
h <sub>c</sub>	0.50 m
a <sub>c</sub>	1.50 m
b <sub>c</sub>	0.50 m
S <sub>conc. cell</sub> (1 cell)	1.64 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc. cell</sub> (1 cell)	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell)	6.20 t/m
W <sub>conc. cover</sub> (1 unit)	0.52 t/m
NOS. of cover block	2.00
S <sub>rubber filling</sub> (1 cell)	1.86 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell)	4.15 t/m
ΣW	11.39 t/m

	Vertical Effect
b <sub>back fill layer</sub>	4 m
Slope	2
γ <sub>Tetrapod</sub>	2.4 t/m <sup>3</sup>
γ <sub>Tetrapod</sub>	1.375 t/m <sup>3</sup>
void ratio	50%
W <sub>tetrapod</sub>	26.25 m <sup>2</sup>
W <sub>tetrapod</sub>	18.05 t/m
	Horizontal Effect (Passive Pressure)
H (m)	2.8
φ (deg)	40.0
γ <sub>s</sub> (t/m <sup>3</sup> )	1.03
Kp = tan <sup>2</sup> (45° + φ/2)	4.6
P <sub>p</sub> (wet), t/m <sup>2</sup>	12.96
Pp, t/m	17.82
γp, m	0.92
M <sub>pp</sub> , t.m	16.34

Rubble Cell	x <sub>sc</sub>	1.75 m
	M (Sc)	19.92 t.m/m

P	11.12 t/m
U	0.00 t/m
γ <sub>h</sub>	1.389 m
x <sub>u</sub>	3.500 m
Mp	15.45 t.m/m
Mu	0.00 t.m/m
M total	15.45 t.m/m

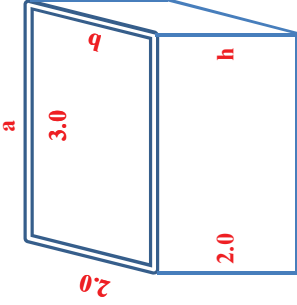
**12 STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - CASE B-1 (LWL)**

**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD -1.0m**

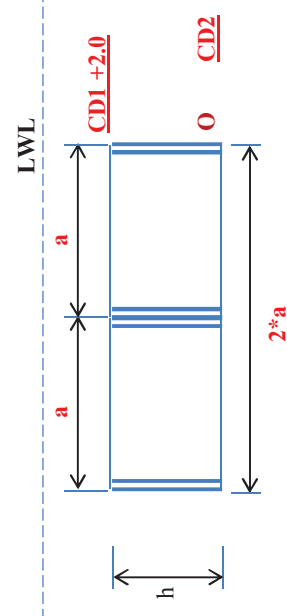
**I. Wave Force under Wave Crest**

**1. Input data:**

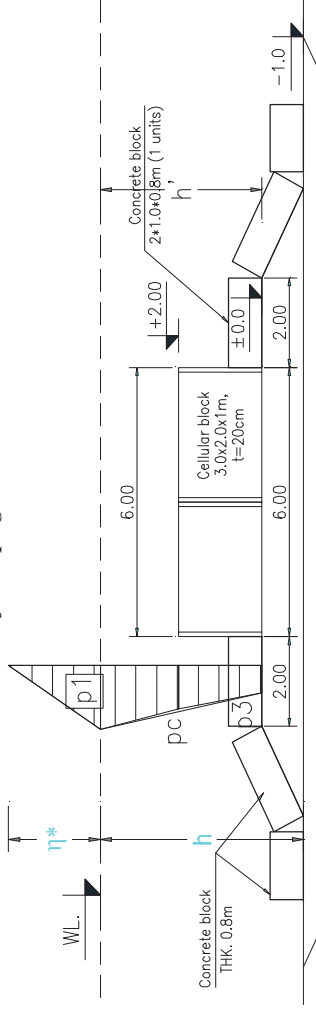
- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	0.0
- Bottom Width of Upright Wall (m):	6.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-1.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	1.48
- H <sub>max</sub> , m:	3.727
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	76.21



*Bellow sketch for reference only*



**2. Wave Pressure on the Front Face of an Upright Wall**



**Wave Pressure Distribution Used in Design Calculation**

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (\text{if } -h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{\max}$   
*breaking zone*

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	57.02
$H_D$ (m):	0.544
$L$ (m):	3.727
$L$ (m):	76.21
HWL (CDL):	0.43
Sea bed Level (CDL):	-1.0
Crest Level (CDL):	2.0
$h$ (m):	1.4
$h'$ (m)	0.43
$\rho_0$ (t/m <sup>3</sup> ):	1.025
$g$ (m/s <sup>2</sup> ):	9.81
$4\pi h/L$ :	0.236
$2\pi h/L$ :	0.118
$\lambda$	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	4.32
$\alpha_1 =$	1.091
$\alpha_3 =$	0.998
$(1 + \cos \beta)$	1.544
$\alpha_1 \lambda \rho_0 H_D$	4.167
$P_1$ (t/m <sup>2</sup> ) =	3.22
$P_c$ (at crest level), (t/m <sup>2</sup> ) =	3.24
$P_3$ (t/m <sup>2</sup> ) =	3.21

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	6.45
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
$W_{\text{con. cell. (total), t/m}}$	13.32

Sliding	<b>S.F. at CD2 Elev.</b>	1.65
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Overturnin	$M_{p \text{ at O}}$	6.46
g	$M_{\text{conc. cell at O}} (=W^*t)$	59.92
	<b>S.F. at O</b>	9.28

CD1	30 years
CD2	+2 m
LWL	0 m
a	+ 0.43 m
h	3 m
b	2 m
t	2.00 m
	0.20 m
$S_{\text{conc. cell (1 cell.)}}$	1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$	1.375 t/m <sup>3</sup>
$W_{\text{conc. cell (1 cell.)}}$	5.06 t/m
$S_{\text{rubber filling (1 cell.)}}$	4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$	2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
$W_{\text{rubber filling (1 cell.)}}$	8.26 t/m
Nos. of cellular block	2.00 unit
$x_o$	4.50 m
$M_o$	59.92 tm/m

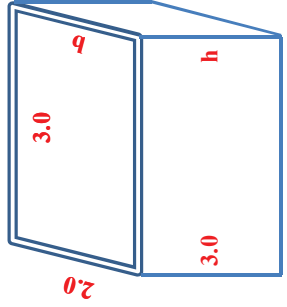
Wave	$P_{\text{up to bottom Elev.}}$	6.45 t/m
	$\gamma_{p \text{ upto bottom Elev.}}$	1.000 m
	$M_{p \text{ at O}}$	6.46 t.m/m

**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD -2.0m**

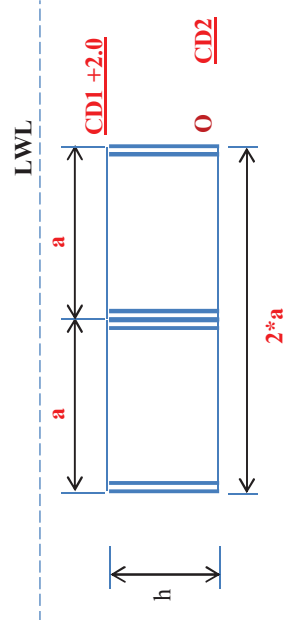
**I. Wave Force under Wave Crest**

**1. Input data:**

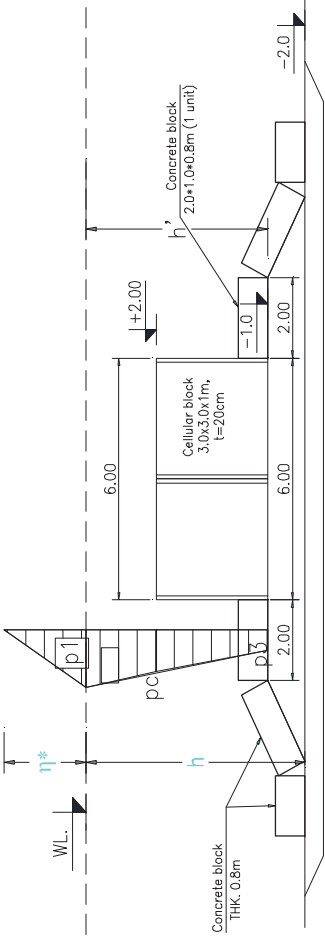
- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-1.0
- Bottom Width of Upright Wall (m):	6.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-2.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	1.952
- H <sub>max</sub> , m:	2.87
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	67.12



*Bellow sketch for reference only*



**2. Wave Pressure on the Front Face of an Upright Wall**



**Wave Pressure Distribution Used in Design Calculation**

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \quad (\text{if } -h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
*breaking zone*

**5. Results of Calculation**

**5.1. Input data**

β (deg) =	30 years
cos β:	65.468
H <sub>D</sub> (m):	0.415
L (m):	2.87
HWL (CDL):	67.12
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-2.0
h (m):	2.0
h' (m):	2.4
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.43
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	0.455
λ:	0.227
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

η* =	3.05
α <sub>1</sub> =	1.067
α <sub>3</sub> =	0.985
(1+cosβ)	1.415
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	3.139
p <sub>1</sub> (t/m <sup>2</sup> ) =	2.22
p <sub>c</sub> (at crest level), (t/m <sup>2</sup> ) =	2.26
p <sub>3</sub> (t/m <sup>2</sup> ) =	2.19

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	6.67
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**II. Safety Factor**

μ (friction factor between conc. cell. stone & stone):	0.80
W <sub>con. cell.</sub> (total), t/m	19.97

Sliding	S.F. at CD2 Elev.	2.4
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Overturnin	Mp <sub>atO</sub>	10.03
g	M <sub>conc. cell.atO</sub> (=W*t)	89.88
	S.F. at O	8.96

CD1	30 years
CD2	+2 m
LWL	-1 m
a	+ 0.43 m
h	3.0 m
b	3.0 m
t	2.0 m
S <sub>conc. cell</sub> (1 cell.)	0.2 m
γ <sub>conc.</sub>	1.84 t/m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)	1.375 t/m <sup>3</sup>
S <sub>rubber filling</sub> (1 cell.)	7.59 t/m
γ <sub>rubber stone</sub>	4.16 t/m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
W <sub>rubber filling</sub> (1 cell.)	37%
Nos. of cellular block	12.38 t/m
x <sub>o</sub>	2.00 unit
M <sub>o</sub>	4.50 m
	89.88 tm/m

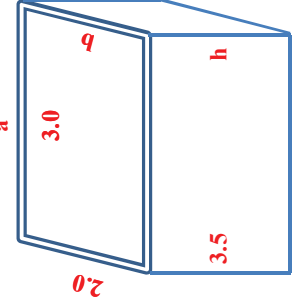
Wave	P <sub>up to bottom Elev.</sub>	6.67 t/m
	γ <sub>p upto bottom Elev.</sub>	1.504 m
	Mp <sub>atO</sub>	10.03 t.m/m

**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD-2.0m - From Sta. 1+750-2+950**

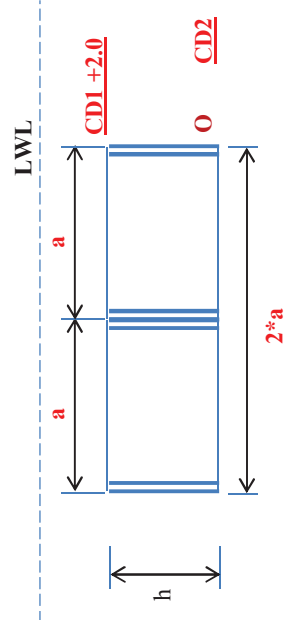
**I. Wave Force under Wave Crest**

**1. Input data:**

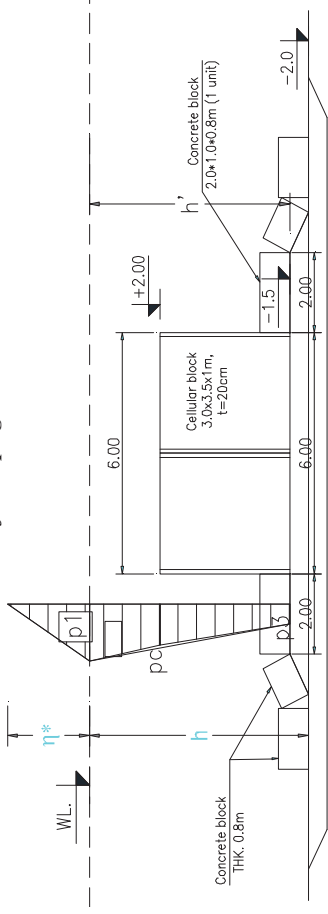
- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-1.5
- Bottom Width of Upright Wall (m):	6.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-2.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	1.952
- H <sub>max</sub> , m:	2.87
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	67.12



Bellow sketch for reference only



**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

H<sub>D</sub> = H<sub>max</sub>  
 breaking zone

**5. Results of Calculation**

**5.1. Input data**

β (deg) =	30 years
cos β:	65.468
H <sub>D</sub> (m):	0.415
L (m):	2.87
HWL (CDL):	67.12
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-2.0
h (m):	2.0
h' (m):	2.4
ρ <sub>0</sub> (t/m <sup>3</sup> ):	1.93
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	0.455
λ:	0.227
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

η* =	3.05
α <sub>1</sub> =	1.067
α <sub>3</sub> =	0.980
(1+cosβ)	1.415
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	3.139
P <sub>1</sub> (t/m <sup>2</sup> ) =	2.22
P <sub>c</sub> (at crest level), (t/m <sup>2</sup> ) =	2.26
P <sub>3</sub> (t/m <sup>2</sup> ) =	2.18

**5.3. Total wave pressure on Upright Wall**

P <sub>up to bottom Elev.</sub> (t/m) =	7.76
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**II. Safety Factor**

μ (friction factor between conc. cell. stone & stone):	0.80
W <sub>con. cell.</sub> (total), t/m	23.30

Sliding	S.F. at CD2 Elev.	2.4
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Overturnin	Mp <sub>atO</sub>	13.62
g	M <sub>conc. cell.atO</sub> (=W*t)	104.86
	S.F. at O	7.70

CD1	30 years
CD2	+2 m
LWL	-1.5 m
a	+ 0.43 m
h	3 m
b	3.5 m
t	2.00 m
S <sub>conc. cell</sub> (1 cell.)	0.20 m
γ <sub>conc.</sub>	1.84 t/m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)	1.375 t/m <sup>3</sup>
S <sub>rubber filling</sub> (1 cell.)	8.86 t/m
γ <sub>rubber stone</sub>	4.16 t/m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
W <sub>rubber filling</sub> (1 cell.)	37%
Nos. of cellular block	14.45 t/m
X <sub>O</sub>	2.00 unit
M <sub>O</sub>	4.50 m
	104.86 t/m/m

Wave	P <sub>up to bottom Elev.</sub>	7.76 t/m
	Y <sub>p upto bottom Elev.</sub>	1.756 m
	Mp <sub>atO</sub>	13.62 t.m/m

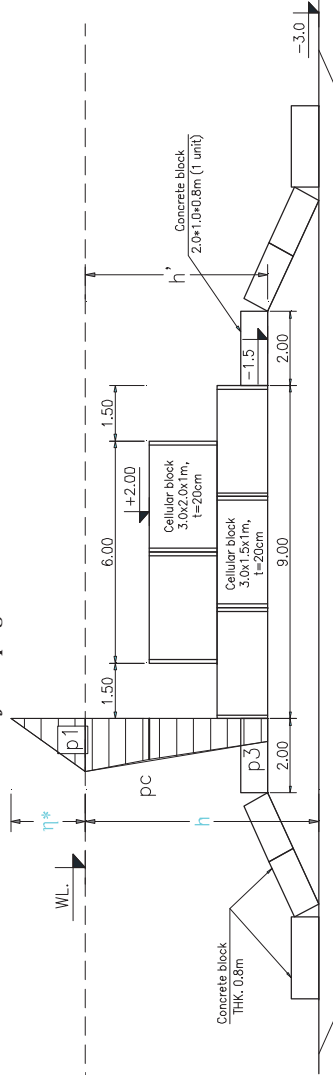
**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD -3.0m**

**I. Wave Force under Wave Crest**

**I. Input data:**

- Wave Return Period:	<b>30 years</b>
- Tide Level - LWL (CDL):	<b>0.43</b>
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-1.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	<b>-3.0</b>
- Significant Wave Height ( $H_{1/3}$ ), m:	2.427
- $H_{max}$ , m:	3.343
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	72.30

**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_w g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  *breaking zone*

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	61.332
$H_D$ (m):	0.480
L (m):	3.343
HWL (CDL):	72.30
Sea bed Level (CDL):	<b>-2.5</b>
Crest Level (CDL):	2.0
h (m):	2.93
h' (m):	1.93
$\rho_0$ (t/m <sup>3</sup> ):	1.025
g (m/s <sup>2</sup> ):	9.81
$4\pi h/L$ :	0.509
$2\pi h/L$ :	0.255
$\lambda$ :	1.000 <i>Upright wall</i>

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	3.71
$\alpha_1 =$	1.059
$\alpha_3 =$	0.979
$(1 + \cos \beta)$	1.480
$\alpha_1 \lambda \rho_w H_D$	3.628
$p_1$ (t/m <sup>2</sup> ) =	2.68
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	2.73
$p_{O1}$ (at CD2 level), (t/m <sup>2</sup> ) =	2.67
$p_3$ (t/m <sup>2</sup> ) =	2.63

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to CD2 Elev.}}$ (t/m) =	5.30
$P_{up \text{ to bottom Elev.}}$ (t/m) =	9.38

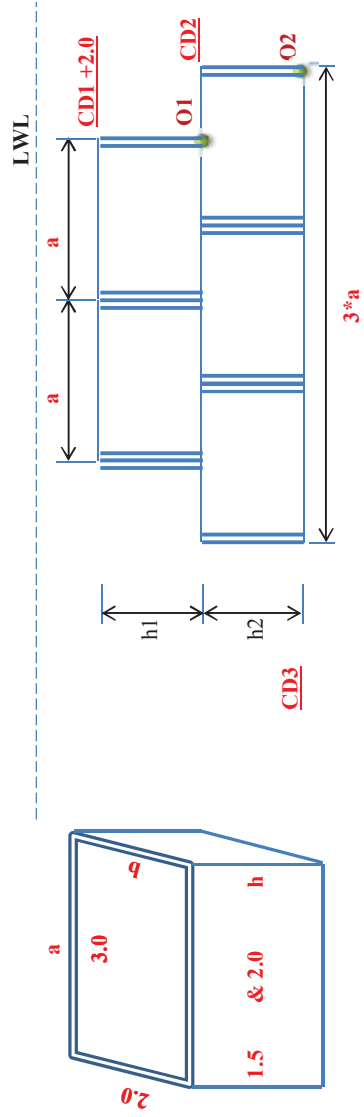
**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W1 con. cell. (total), t/m	13.32
W2 con. cell. (total), t/m	14.98

Sliding	<b>S.F. at CD2 Elev. 2.01</b>
	<b>S.F. at CD3 Elev. 2.41</b>

Overturning	$M_{p \text{ at O1}}$	5.31
	$M_{\text{conc. cell. at O1}} (=W^*t)$	59.92
	<b>S.F. at O1</b>	<b>11.29</b>
	$M_{p \text{ at O2}}$	16.47
	$M_{\text{conc. cell. at O2}} (=W^*t)$	168.77
	<b>S.F. at O2</b>	<b>10.25</b>

*Bellow sketch for reference only*



<b>CD1</b>	30 years
<b>CD2</b>	+2 m
<b>CD3</b>	+0 m
<b>LWL</b>	-1.50 m
<b>a</b>	+0.43 m
<b>h1</b>	3 m
<b>h2</b>	2 m
<b>b</b>	1.5 m
<b>t</b>	2.00 m
$S_{\text{conc. cell}}$ (1 cell.)	0.20 m
$S_{\text{conc. cell}}$ (1 cell.)	1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4 t/m <sup>3</sup>
$\gamma'_{\text{conc.}}$	1.375 t/m <sup>3</sup>
W1 conc. cell (1 cell.)	5.06 t/m
W2 conc. cell (1 cell.)	3.80 t/m
$S_{\text{rubber filling}}$ (1 cell.)	4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$	2.6 t/m <sup>3</sup>
$\gamma'_{\text{rubber stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
W1 rubber filling (1 cell.)	8.26 t/m
W2 rubber filling (1 cell.)	6.19 t/m
Nos. of cellular block	2.00 unit
$X_{O1}$	4.50 m
$M_{O1}$	59.92 tm/m
Nos. of cellular block	3.00 unit
$X_{O2}$	6.00 m
$M_{O2}$	168.77 tm/m

Wave	$P_{up \text{ to CD2 Elev.}}$	5.30 t/m
	$\gamma_p$ upto CD2 Elev.	1.00
	$M_p$ at O1	5.31 t.m/m
	$P_{up \text{ to bottom Elev.}}$	9.38 t/m
	$\gamma_p$ upto bottom Elev.	1.756 m
	$M_p$ at O2	16.47 t.m/m

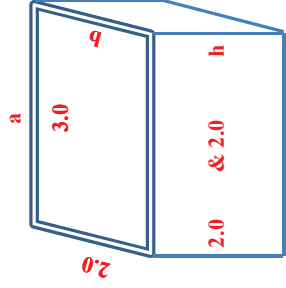


**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD -3.0m**

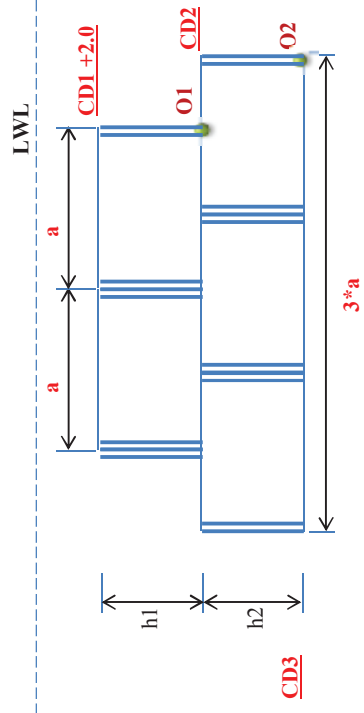
**I. Wave Force under Wave Crest**

**I. Input data:**

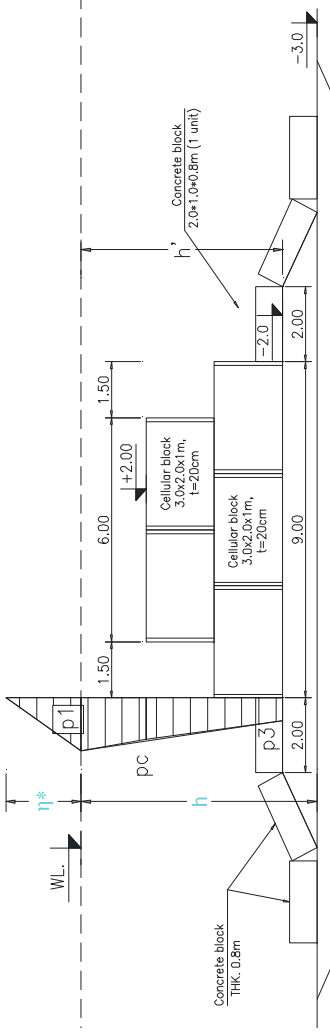
- Wave Return Period:	<b>30 years</b>
- Tide Level - LWL (CDL):	<b>0.43</b>
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-2.0
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	<b>-3.0</b>
- Significant Wave Height ( $H_{1/3}$ ), m:	2.427
- $H_{max}$ , m:	3.343
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	72.30



*Below sketch for reference only*



**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0gH_D$

At toe of upright wall:  $P_3 = P_u = \alpha_5 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_5 = 1 + \frac{h'}{\eta^*} \text{ (if } h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	61.332
$H_D$ (m):	0.480
L (m):	3.343
HWL (CDL):	72.30
Sea bed Level (CDL):	0.43
Crest Level (CDL):	<b>-3.0</b>
h (m):	2.0
h' (m):	3.43
$\rho_0$ (t/m <sup>3</sup> ):	2.43
g (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	0.596
$\lambda$ :	0.298
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	3.71
$\alpha_1 =$	1.045
$\alpha_3 =$	0.970
$(1 + \cos \beta)$	1.480
$\alpha_1 \lambda \rho_0 H_D$	3.580
$p_1$ (t/m <sup>2</sup> ) =	2.65
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	2.70
$p_{O1}$ (at CD2 level), (t/m <sup>2</sup> ) =	2.63
$p_3$ (t/m <sup>2</sup> ) =	2.57

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to CD2 Elev.}}$ (t/m) =	5.20
$P_{up \text{ to bottom Elev.}}$ (t/m) =	10.54

**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W1 con. cell. (total), t/m	13.32
W2 con. cell. (total), t/m	19.97

<b>S.F. at CD2 Elev.</b>	<b>2.05</b>
<b>S.F. at CD3 Elev.</b>	<b>2.53</b>

$M_{p \text{ at O1}}$	5.21
$M_{conc. \text{ cell. at O1}} (=W^*t)$	59.92
<b>S.F. at O1</b>	<b>11.51</b>
$M_{p \text{ at O2}}$	21.18
$M_{conc. \text{ cell. at O2}} (=W^*t)$	199.73
<b>S.F. at O2</b>	<b>9.43</b>

<b>CD1</b>	30 years
<b>CD2</b>	+2 m
<b>CD3</b>	+0 m
<b>LWL</b>	+2 m
<b>a</b>	+0.43 m
<b>h1</b>	3 m
<b>h2</b>	2 m
<b>b</b>	2.00 m
<b>t</b>	0.20 m
$S_{conc. \text{ cell.}}$ (1 cell.)	1.84 m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
$\gamma'_{conc.}$	1.375 t/m <sup>3</sup>
$W1_{conc. \text{ cell.}}$ (1 cell.)	5.06 t/m
$W2_{conc. \text{ cell.}}$ (1 cell.)	5.06 t/m
$S_{rubber \text{ filling}}$ (1 cell.)	4.16 m <sup>2</sup>
$\gamma_{rubber \text{ stone}}$	2.6 t/m <sup>3</sup>
$\gamma'_{rubber \text{ stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
$W1_{rubber \text{ filling}}$ (1 cell.)	8.26 t/m
$W2_{rubber \text{ filling}}$ (1 cell.)	8.26 t/m
Nos. of cellular block	2.00 unit
$X_{O1}$	4.50 m
$M_{O1}$	59.92 tm/m
Nos. of cellular block	3.00 unit
$X_{O2}$	6.00 m
$M_{O2}$	199.73 tm/m

<b>Upper Layer</b>	
$P_{up \text{ to CD2 Elev.}}$	5.20 t/m
$\gamma'_{p \text{ upto CD2 Elev.}}$	1.00
<b>Lower Layer</b>	
$M_{p \text{ at O1}}$	5.21 t.m/m
$P_{up \text{ to bottom Elev.}}$	10.54 t/m
$\gamma'_{p \text{ upto bottom Elev.}}$	2.010 m
$M_{p \text{ at O2}}$	21.18 t.m/m

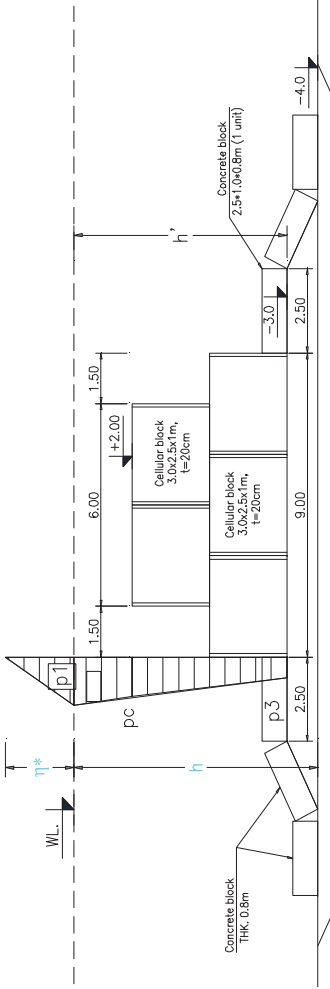
**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OCDI)**  
**Sand Protection Training Dike - Section for GL upto CD -3.0m**

**I. Wave Force under Wave Crest**

**I. Input data:**

- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-3.0
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-4.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	2.985
- H <sub>max</sub> , m:	4.012
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	78.98

**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0gH_D$

At toe of upright wall:  $P_3 = P_u = \alpha_5P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_5 = 1 + \frac{h'}{\eta^*} \text{ (if } h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	60.936
$H_D$ (m):	0.486
$L$ (m):	4.012
HWL (CDL):	78.98
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-4.0
$h$ (m):	2.0
$h'$ (m):	4.43
$\rho_0$ (t/m <sup>3</sup> ):	3.43
$g$ (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	0.705
$\lambda$ :	0.352
$\lambda$ :	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	4.47
$\alpha_1 =$	1.025
$\alpha_3 =$	0.954
$(1 + \cos \beta)$	1.486
$\alpha_1 \lambda \rho_0 H_D$	4.214
$p_1$ (t/m <sup>2</sup> ) =	3.13
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	3.20
$p_{O1}$ (at CD2 level), (t/m <sup>2</sup> ) =	3.09
$p_3$ (t/m <sup>2</sup> ) =	2.99

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to CD2 Elev.}}$ (t/m) =	7.60
$P_{up \text{ to bottom Elev.}}$ (t/m) =	15.46

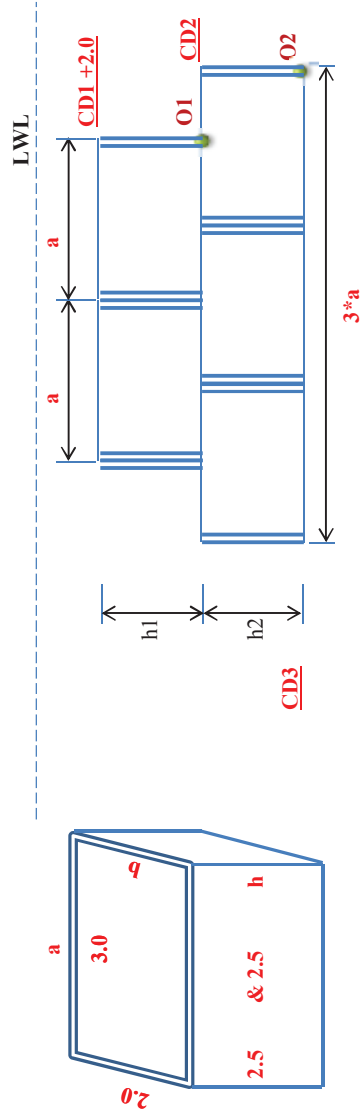
**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W1 con. cell. (total), t/m	16.64
W2 con. cell. (total), t/m	24.97

Sliding	<b>S.F. at CD2 Elev. 1.75</b>
	<b>S.F. at CD3 Elev. 2.15</b>

Overturning	$M_{p \text{ at O1}}$	9.52
	$M_{conc. \text{ cell. at O1}} (=W*u)$	74.90
	<b>S.F. at O1</b>	<b>7.87</b>
	$M_{p \text{ at O2}}$	38.95
	$M_{conc. \text{ cell. at O2}} (=W*u)$	249.67
	<b>S.F. at O2 6.41</b>	

*Below sketch for reference only*



CD1	+2 m
CD2	-0.50 m
CD3	-3.00 m
LWL	+ 0.43 m
a	3 m
h1	2.5 m
h2	2.5 m
t	2.00 m
t	0.20 m
S <sub>conc. cell</sub> (1 cell.)	1.84 m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
$\gamma'_{conc.}$	1.375 t/m <sup>3</sup>
W1 <sub>conc. cell</sub> (1 cell.)	6.33 t/m
W2 <sub>conc. cell</sub> (1 cell.)	6.33 t/m
S <sub>rubber filling</sub> (1 cell.)	4.16 m <sup>2</sup>
$\gamma_{rubber \text{ stone}}$	2.6 t/m <sup>3</sup>
$\gamma'_{rubber \text{ stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
W1 <sub>rubber filling</sub> (1 cell.)	10.32 t/m
W2 <sub>rubber filling</sub> (1 cell.)	10.32 t/m
Nos. of cellular block	2.00 unit
X <sub>O1</sub>	4.50 m
M <sub>O1</sub>	74.90 tm/m
Nos. of cellular block	3.00 unit
X <sub>O2</sub>	6.00 m
M <sub>O2</sub>	249.67 tm/m

Wave	$P_{up \text{ to CD2 Elev.}}$	7.60 t/m
	$\gamma'_{p \text{ upto CD2 Elev.}}$	1.25
	$M_{p \text{ at O1}}$	9.52 t.m/m
	$P_{up \text{ to bottom Elev.}}$	15.46 t/m
	$\gamma'_{p \text{ upto bottom Elev.}}$	2.519 m
	$M_{p \text{ at O2}}$	38.95 t.m/m

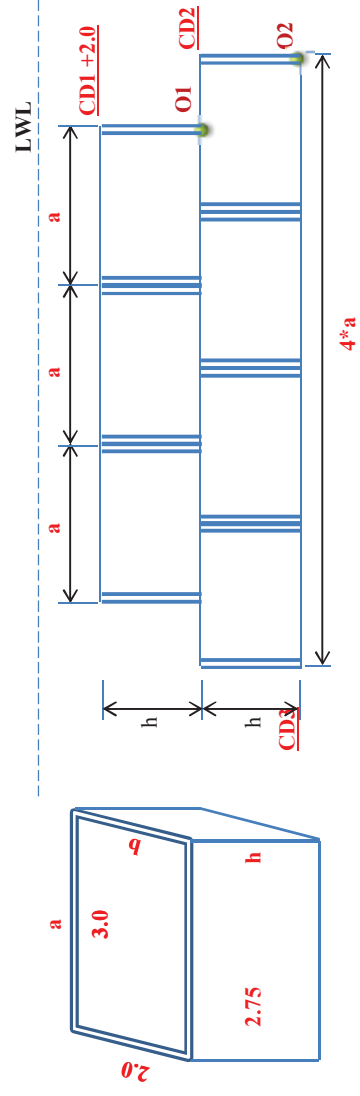
**STABILITY CALCULATION DUE TO WAVE FORCE (Clause 5.5.2 - OC DI)**  
**Sand Protection Training Dike - Section for GL upto CD -5.0m**

**I. Wave Force under Wave Crest**

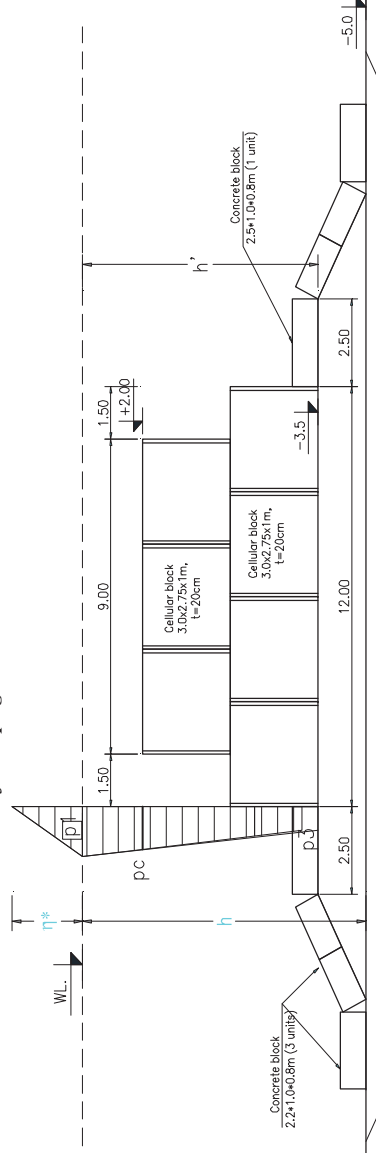
**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	<b>0.43</b>
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-3.5
- Bottom Width of Upright Wall (m):	12.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	<b>-5.0</b>
- Significant Wave Height ( $H_{1/3}$ ), m:	3.457
- $H_{max}$ , m:	4.595
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	84.32

Bellow sketch for reference only



**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0gH_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3P_1$

$$\lambda = \exp\left[-10\left(\frac{h}{L}\right)^{1.5}\left(1 - \frac{h'}{h}\right)^5\right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$

*breaking zone*

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30 years
$\cos \beta$ :	60.681
$H_D$ (m):	0.490
L (m):	4.595
HWL (CDL):	84.32
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-5.0
h (m):	2.0
h' (m):	5.43
$\rho_0$ (t/m <sup>3</sup> ):	3.93
g (m/s <sup>2</sup> ):	1.025
4 $\pi h/L$ :	9.81
2 $\pi h/L$ :	0.809
$\lambda$ :	0.405
	1.000

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	5.13
$\alpha_1 =$	1.004
$\alpha_3 =$	0.945
$(1 + \cos \beta)$	1.490
$\alpha_1 \lambda \rho_0 H_D$	4.728
$P_1$ (t/m <sup>2</sup> ) =	3.52
$P_2$ (at crest level), (t/m <sup>2</sup> ) =	3.60
$P_{O1}$ (at CD2 level), (t/m <sup>2</sup> ) =	3.46
$P_3$ (t/m <sup>2</sup> ) =	3.33

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to CD2 Elev.}}$ (t/m) =	9.33
$P_{up \text{ to bottom Elev.}}$ (t/m) =	19.05

**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
Wcon. cell. (total), t/m	64.08

Sliding	<b>S.F. at CD2 Elev.</b>	2.35
	<b>S.F. at CD3 Elev.</b>	2.69

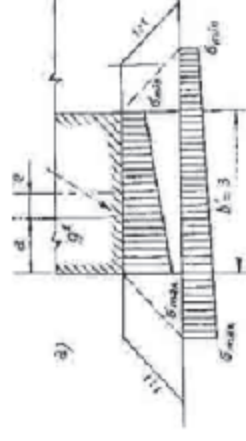
Overturning	$M_{p \text{ at O1}}$	12.87
	$M_{conc. \text{ cell. at O1}} (=W^*t)$	123.58
	<b>S.F. at O1</b>	<b>9.60</b>
	$M_{p \text{ at O2}}$	52.87
	$M_{conc. \text{ cell. at O2}} (=W^*t)$	384.49
	<b>S.F. at O2</b>	<b>7.27</b>

CD1	30 years
CD2	+ 2 m
CD3	-0.8 m
LWL	-3.5 m
a	+ 0.43 m
h	3 m
b	2.75 m
t	2.00 m
S <sub>conc. cell</sub> (1 cell.)	0.20 m
$\gamma_{conc.}$	1.84 m <sup>2</sup>
$\gamma'_{conc.}$	2.4 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)	1.375 t/m <sup>3</sup>
S <sub>rubber filling</sub> (1 cell.)	6.96 t/m
$\gamma_{rubber \text{ stone}}$	4.16 m <sup>2</sup>
$\gamma'_{rubber \text{ stone}}$	2.6 t/m <sup>3</sup>
void ratio	1.575 t/m <sup>3</sup>
W <sub>rubber filling</sub> (1 cell.)	37%
Nos. of cellular block	11.35 t/m
$X_{O1}$	3.00 unit
$M_{O1}$	4.50 m
Nos. of cellular block	123.58 tm/m
$X_{O2}$	4.00 unit
$M_{O2}$	6.00 m
	384.49 tm/m

Wave	$P_{up \text{ to CD2 Elev.}}$	9.33 t/m
	$\gamma_{p \text{ upto CD2 Elev.}}$	1.38
	$M_{p \text{ at O1}}$	12.87 t.m/m
	$P_{up \text{ to bottom Elev.}}$	19.05 t/m
	$\gamma_{p \text{ upto bottom Elev.}}$	2.776 m
	$M_{p \text{ at O2}}$	52.87 t.m/m

**Bearing capacity of soft soil under foundation**

a = 5.17 m  
 b = 12 m  
 b/3 = 4 m  
 Checking condition  $a \geq b/3$ :  
 c = 0.83 m



— ( — ) rubberstone = 6.0 kg/cm<sup>2</sup>  
 $\sigma_{max} = 7.54 \text{ t/m}^2 = 0.75 \text{ kg/cm}^2$  **OK**  
 $\sigma_{min} = 3.14 \text{ t/m}^2 = 0.31 \text{ kg/cm}^2$  **OK**  
 — soft soil = 2.0 kg/cm<sup>2</sup>  
 $b' = 12 \text{ m}$   
 $hn = 1.5 \text{ m}$   
 $\gamma_{c-k} = 1.575 \text{ t/m}^3$   
 $\sigma'_{max} = 8.40 \text{ t/m}^2 = 0.84 \text{ kg/cm}^2$  **OK**  
 $\sigma'_{min} = 4.87 \text{ t/m}^2 = 0.49 \text{ kg/cm}^2$  **OK**

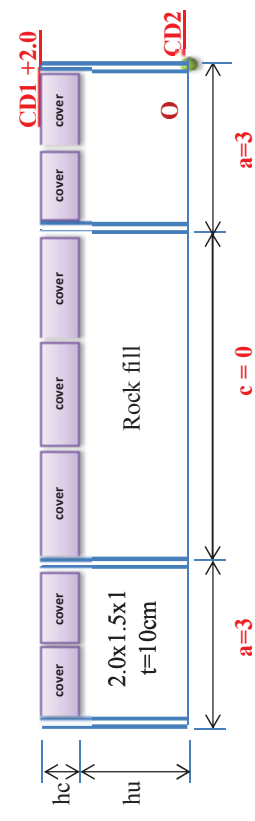
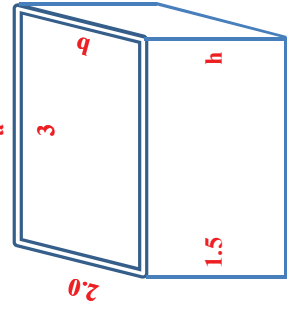
**13 STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - CASE B-2 (LWL)**

**D. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD 0.0m**  
(Clause 5.5.2 - OCD1)

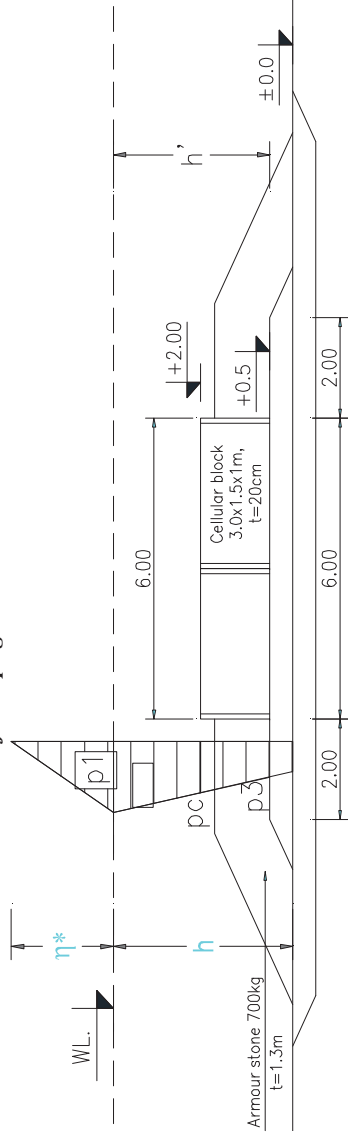
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	0.5
- Bottom Width of Upright Wall (m):	6.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	0.0
- Significant Wave Height ( $H_{1/3}$ ), m:	0.93
- $H_{max}$ , m:	3.14
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	70.13



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_v = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{h} \eta^* \quad (\text{if } h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$

breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	32.35
$H_D$ (m):	0.845
L (m):	3.14
LWL (CDL):	70.13
Sea bed Level (CDL):	0.43
Crest Level (CDL):	0.0
h (m):	2.0
h' (m):	0.43
$\rho_0$ (t/m <sup>3</sup> ):	-0.07
g (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	0.077
$\lambda$ :	0.039
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	4.34
$\alpha_1 =$	1.099
$\alpha_3 =$	0.984
$(1 + \cos \beta)$	1.845
$\alpha_1 \lambda \rho_0 H_D$	3.537
$P_1$ (t/m <sup>2</sup> ) =	3.26
$P_v$ (at crest level), (t/m <sup>2</sup> ) =	2.08
$P_3$ (t/m <sup>2</sup> ) =	3.21

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	3.97
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W <sub>con. cell.</sub> (total), t/m	7.92
W <sub>conc. cover</sub> (total), t/m	3.58
W <sub>core rubble at bottom</sub> , t/m	0.00

**Sliding**

<b>S.F. at bottom Elev.</b>	<b>2.32</b>
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**Overturnin g**

$M_{p \text{ at O}}$	2.77
$M_{\text{conc. cell at O2}} (=W*t)$	55.09
<b>S.F. at O</b>	<b>19.91</b>

<b>CDI</b>	30 years
<b>CD2</b>	+2 m
<b>LWL</b>	+0.5 m
<b>a</b>	+0.43 m
<b>h</b>	3 m
<b>hu</b>	1.5 m
<b>hu</b>	1.00 m
<b>b</b>	2.00 m
<b>t</b>	0.20 m
<b>c</b>	0.00 m
<b>hc</b>	0.50 m
$2e_{\text{side}}$	2.6 m
$2e_{\text{center}}$	0.00 m
$S_{\text{conc. cell}}$ (1 cell.)	1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4 t/m <sup>3</sup>
$\gamma'_{\text{conc.}}$	1.375 t/m <sup>3</sup>
$W_{\text{conc. cell}}$ (1 cell.)	3.80 t/cell.
$S_{\text{rubber filling}}$ (1 cell.)	4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$	2.6 t/m <sup>3</sup>
$\gamma'_{\text{rubber stone}}$	1.575 t/m <sup>3</sup>
<b>void ratio</b>	37%
$W_{\text{rubber filling}}$ (1 cell.)	4.13 t/cell.
$W_{\text{conc. cover side}}$	1.79 t/m
$W_{\text{conc. cover center}}$	0.00 t/m
$W_{\text{core rubble at bottom}}$	0.00 t/m
<b>Nos. of cellular block</b>	<b>2.00</b> unit
$x_{O1}$	3.00 m
$M_{O1}$	55.09 tm/m

Upper Layer

<b>Wave</b>	
$P_{\text{up to bottom Elev.}}$	3.97 t/m
$y_{p \text{ upto bottom Elev.}}$	0.697 m
$M_{p \text{ at O2}}$	2.77 t.m/m

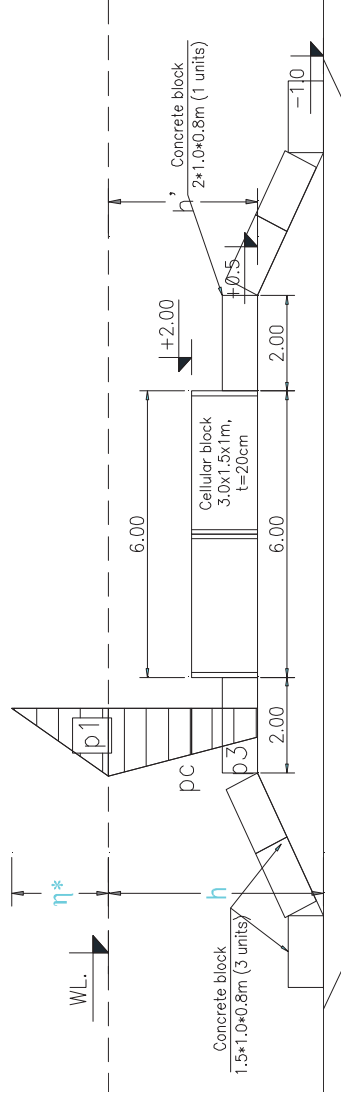
**E. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -1.0m**  
(Clause 5.5.2 - OC DI)

**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	2.0
- Bottom Upright Wall (CDL):	0.5
- Bottom Width of Upright Wall (m):	6.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-1.0
- Significant Wave Height ( $H_{1/3}$ ), m:	1.48
- $H_{max}$ , m:	3.727
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	76.21

**2. Wave Pressure on the Front Face of an Upright Wall**



**Wave Pressure Distribution Used in Design Calculation**

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	57.02
$H_D$ (m):	0.544
L (m):	3.727
LWL (CDL):	76.21
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-1.0
h (m):	2.0
h' (m):	1.43
$\rho_0$ (t/m <sup>3</sup> ):	-0.07
g (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	0.236
$\lambda$ :	0.118
$\lambda$ :	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^*$ =	4.32
$\alpha_1$ =	1.091
$\alpha_3$ =	0.984
$(1 + \cos \beta)$ :	1.544
$\alpha_1 \lambda \rho_0 H_D$ :	4.167
$P_1$ (t/m <sup>2</sup> ) =	3.22
$P_C$ (at crest level), (t/m <sup>2</sup> ) =	2.05
$P_3$ (t/m <sup>2</sup> ) =	3.17

**5.3. Total wave pressure on Upright Wall**

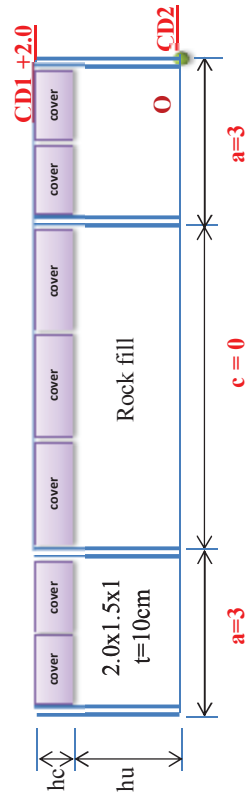
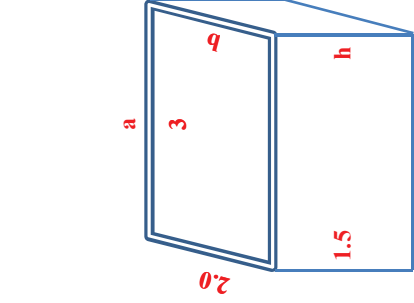
$P_{up \text{ to bottom Elev. (t/m)}}$ =	3.91
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W <sub>conc. cell. (total)</sub> , t/m	7.92
W <sub>conc. cover (total)</sub> , t/m	3.58
W <sub>conc. rubble at bottom</sub> , t/m	0.00

**Sliding**

<b>S.F. at bottom Elev.</b>	<b>2.35</b>
$M_{p \text{ at O}}$	2.72
$M_{\text{conc. cell. at O2}} (=W^*t)$	55.09
<b>S.F. at O</b>	<b>20.23</b>



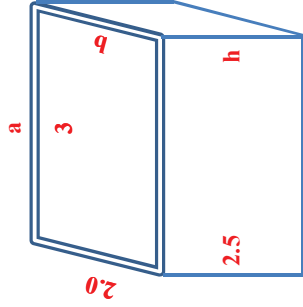
CDI	30 years	+ 2 m
CD2		+ 0.5 m
LWL		+ 0.43 m
a		3 m
h		1.5 m
hu		1.00 m
b		2.00 m
t		0.20 m
c		0.00 m
hc		0.50 m
a <sub>c</sub> side		2.6 m
a <sub>c</sub> center		0.00 m
S <sub>conc. cell (1 cell.)</sub>		1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$		2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$		1.375 t/m <sup>3</sup>
W <sub>conc. cell (1 cell.)</sub>		3.80 t/cell.
S <sub>rubber filling (1 cell.)</sub>		4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$		2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$		1.575 t/m <sup>3</sup>
void ratio		37%
W <sub>rubber filling (1 cell.)</sub>		4.13 t/cell.
W <sub>conc. cover side</sub>		1.79 t/m
W <sub>conc. cover center</sub>		0.00 t/m
W <sub>conc. rubble at bottom</sub>		0.00 t/m
Nos. of cellular block		2.00 unit
X <sub>O1</sub>		3.00 m
M <sub>O1</sub>		55.09 tm/m
<b>Upper Layer</b>		
<b>Wave</b>		
P <sub>up to bottom Elev.</sub>		3.91 t/m
Y <sub>p upto bottom Elev.</sub>		0.696 m
M <sub>p at O2</sub>		2.72 t.m/m

**F. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -2.0m**  
(Clause 5.5.2 - OCDI)

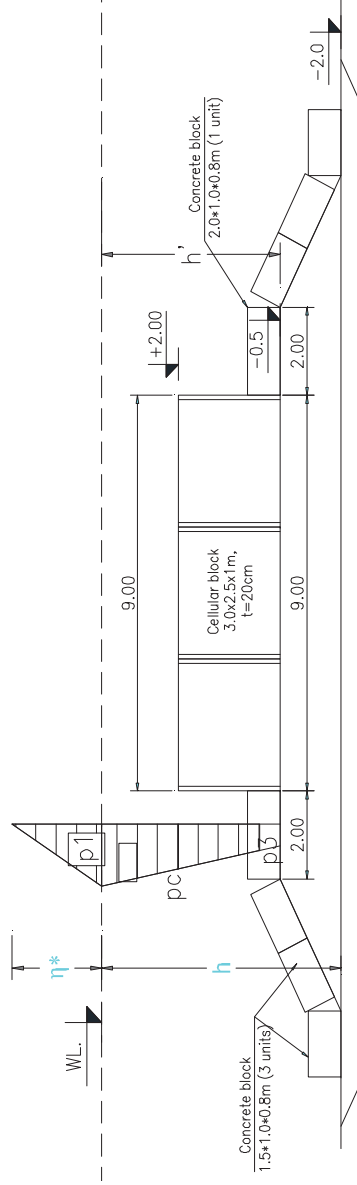
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	2.0
- Bottom Upright Wall Level (CDL):	-0.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-2.0
- Significant Wave Height ( $H_{1/3}$ ), m:	1.952
- $H_{max}$ , m:	2.87
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	67.12



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	65.468
$H_D$ (m):	0.415
L (m):	2.87
LWL (CDL):	67.12
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-2.0
h (m):	2.0
h' (m):	2.43
$\rho_0$ (t/m <sup>3</sup> ):	0.93
g (m/s <sup>2</sup> ):	1.025
4 $\pi h / L$ :	9.81
2 $\pi h / L$ :	4.455
$\lambda$ :	0.227
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	3.05
$\alpha_1 =$	1.067
$\alpha_3 =$	0.990
$(1 + \cos \beta)$	1.415
$\alpha_1 \lambda \rho_0 H_D$	3.139
$p_1$ (t/m <sup>2</sup> ) =	2.22
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	2.26
$p_3$ (t/m <sup>2</sup> ) =	2.20

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	5.57
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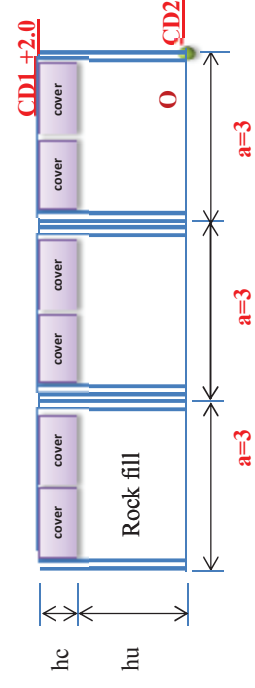
**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W <sub>con. cell.</sub> (total), t/m	21.87
W <sub>conc. cover</sub> (total), t/m	5.36

Sliding	<b>3.91</b>
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Overturnin g	6.99
$M_{\text{conc. cell at O2}} (=W*t)$	220.97
<b>S.F. at O</b>	<b>31.60</b>

Sketch of detail of super structure  
LWL



CD1	30 years
CD2	+2 m
WL	-0.5 m
a	+0.43 m
h	3 m
hu	2.5 m
hc	2.00 m
b	2.00 m
t	0.20 m
hc	0.50 m
a inside	2.6 m
S <sub>conc. cell</sub> (1 cell.)	1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)	6.33 t/cell.
S <sub>rubber filling</sub> (1 cell.)	4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$	2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell.)	8.26 t/cell.
W <sub>conc. cover side</sub>	1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	X <sub>O1</sub>	4.50 m
	M <sub>O1</sub>	220.97 tm/m

Wave	P <sub>up to bottom Elev.</sub>	5.57 t/m
	$\gamma_p$ upto bottom Elev.	1.255 m
	MP <sub>at O2</sub>	6.99 t.m/m

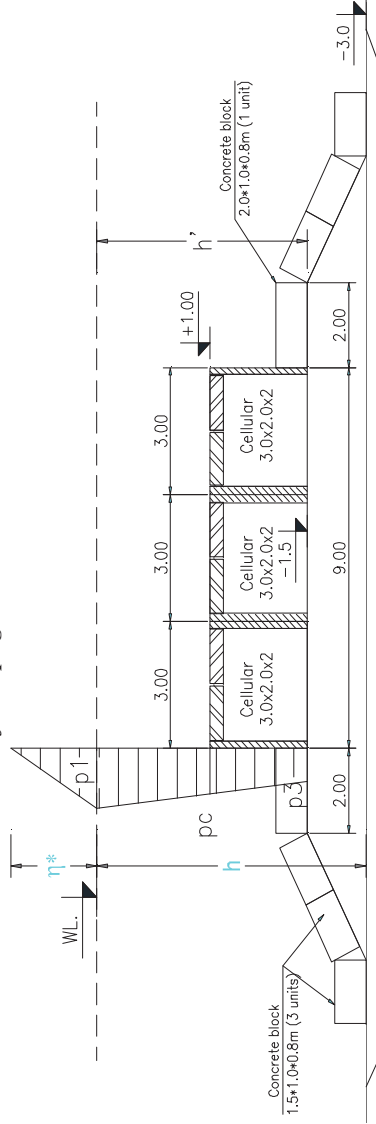
**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	1.0
- Bottom Upright Wall Level (CDL):	-1.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-3.0
- Significant Wave Height ( $H_{1/3}$ ), m:	2.427
- $H_{max}$ , m:	3.343
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	72.30

**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	61.332
$H_D$ (m):	0.480
L (m):	3.343
LWL (CDL):	72.30
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-3.0
h (m):	1.0
h' (m):	3.43
$\rho_0$ (t/m <sup>3</sup> ):	1.93
g (m/s <sup>2</sup> ):	1.025
4 $\pi h / L$ :	9.81
2 $\pi h / L$ :	0.596
$\lambda$ :	0.298
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^*$ =	3.71
$\alpha_1$ =	1.045
$\alpha_3$ =	0.976
(1 + $\cos \beta$ )	1.480
$\alpha_1 \lambda \rho_0 H_D$	3.580
$p_1$ (t/m <sup>2</sup> ) =	2.65
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	2.67
$p_3$ (t/m <sup>2</sup> ) =	2.58

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to bottom Elev.}}$ (t/m) =	6.57
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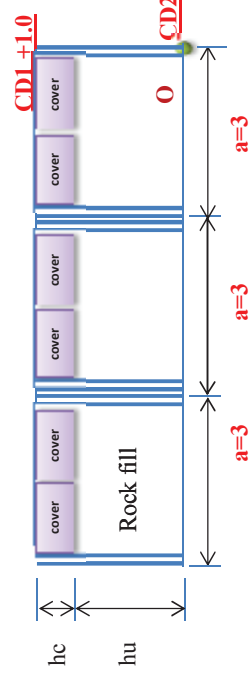
**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
Wconc. cell. (total), t/m	21.87
Wconc. cover (total), t/m	5.36

Sliding	S.F. at bottom Elev.	3.32
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Overturnin g	$M_{p \text{ at } O}$	8.25
	$M_{conc. \text{ cell. at } O2} (=W*t)$	220.97
	S.F. at O	26.78

Sketch of detail of super structure  
LWL



CD1	30 years	+1	m
CD2		-1.5	m
LWL		+0.43	m
a		3	m
h		2.5	m
hu		2.00	m
b		2.00	m
t		0.20	m
hc		0.50	m
a inside		2.6	m
S <sub>conc. cell (1 cell.)</sub>		1.84	m <sup>2</sup>
$\gamma_{conc.}$		2.4	t/m <sup>3</sup>
$\gamma_{conc.}$		1.375	t/m <sup>3</sup>
W <sub>conc. cell (1 cell.)</sub>		6.33	t/cell.
S <sub>rubber filling (1 cell.)</sub>		4.16	m <sup>2</sup>
$\gamma_{rubber \text{ stone}}$		2.6	t/m <sup>3</sup>
$\gamma_{rubber \text{ stone}}$		1.575	t/m <sup>3</sup>
void ratio		37%	
W <sub>rubber filling (1 cell.)</sub>		8.26	t/cell.
W <sub>conc. cover side</sub>		1.79	t/m

Upper Layer	Nos. of cellular block	3.00	unit
	X <sub>O1</sub>	4.50	m
	M <sub>O1</sub>	220.97	tm/m

Wave	P <sub>up to bottom Elev.</sub>	6.57	t/m
	$\gamma_p$ upto bottom Elev.	1.257	m
	MP <sub>at O2</sub>	8.25	t.m/m



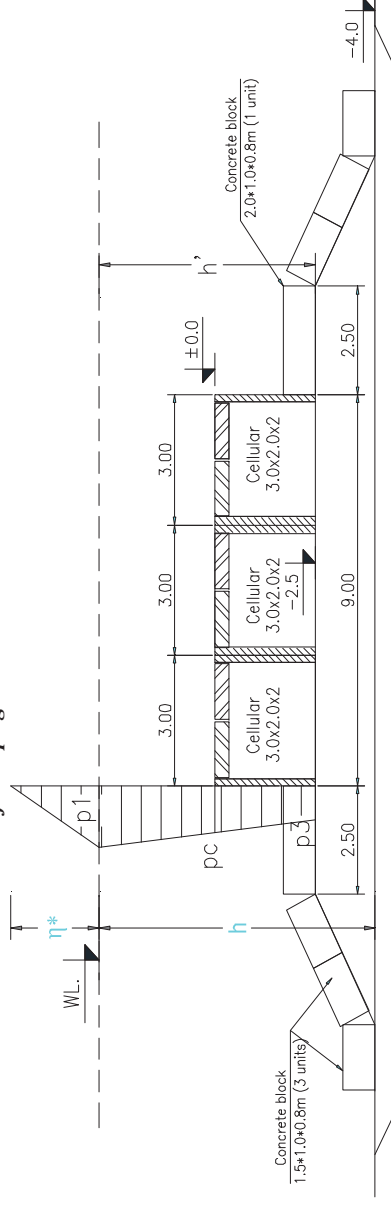
**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	0.0
- Bottom Upright Wall Level (CDL):	-2.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-4.0
- Significant Wave Height ( $H_{1/3}$ ), m:	2.985
- $H_{max}$ , m:	4.012
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	78.98

**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$  breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	60.936
$H_D$ (m):	0.486
L (m):	4.012
LWL (CDL):	78.98
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-4.0
h (m):	0.0
h' (m):	4.43
$\rho_0$ (t/m <sup>3</sup> ):	2.93
g (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	4.705
$\lambda$ :	0.352
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^*$ =	4.47
$\alpha_1$ =	1.025
$\alpha_3$ =	0.961
$(1 + \cos \beta)$	1.486
$\alpha_1 \lambda \rho_0 H_D$	4.214
$p_1$ (t/m <sup>2</sup> ) =	3.13
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	3.11
$p_3$ (t/m <sup>2</sup> ) =	3.01

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to bottom Elev.}}$ (t/m) =	7.65
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**II. Safety Factor**

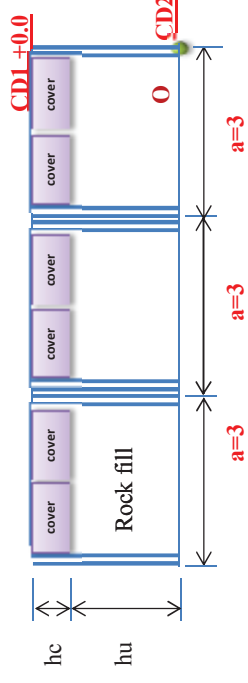
$\mu$ (friction factor between conc. cell. stone & stone):	0.80
Wcon. cell. (total), t/m	21.87
Wconc. cover (total), t/m	5.36

Sliding	<b>S.F. at bottom Elev.</b>	2.85
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Overturnin g	$M_{p \text{ at O}}$	9.62
	$M_{\text{conc. cell. at O2}} (=W*t)$	220.97
	<b>S.F. at O</b>	22.97

*Sketch of detail of super structure*

LWL



CD1	30 years	+0	m
CD2		-2.5	m
LWL		+0.43	m
a		3	m
h		2.5	m
hu		2.00	m
b		2.00	m
t		0.20	m
hc		0.50	m
a inside		2.6	m
S <sub>conc. cell</sub> (1 cell.)		1.84	m <sup>2</sup>
$\gamma_{\text{conc.}}$		2.4	t/m <sup>3</sup>
$\gamma_{\text{conc.}}$		1.375	t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)		6.33	t/cell.
S <sub>rubber filling</sub> (1 cell.)		4.16	m <sup>2</sup>
$\gamma_{\text{rubber stone}}$		2.6	t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$		1.575	t/m <sup>3</sup>
void ratio		37%	
W <sub>rubber filling</sub> (1 cell.)		8.26	t/cell.
W <sub>conc. cover side</sub>		1.79	t/m

Upper Layer	Nos. of cellular block	3.00	unit
	X <sub>O1</sub>	4.50	m
	M <sub>O1</sub>	220.97	tm/m

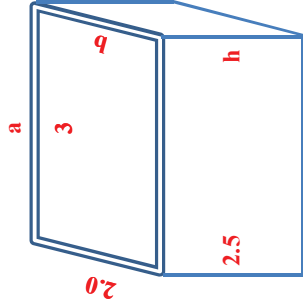
Wave	P <sub>up to bottom Elev.</sub>	7.65	t/m
	$\gamma_p$ upto bottom Elev.	1.257	m
	MP <sub>at O2</sub>	9.62	t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

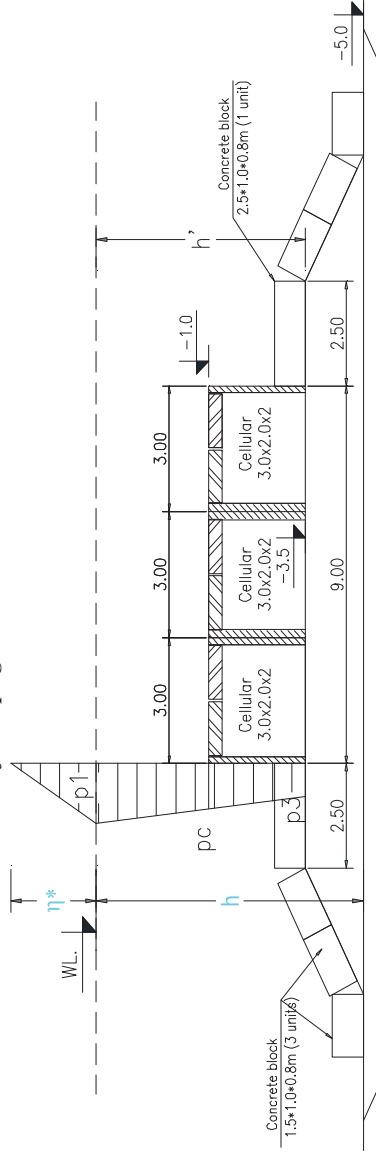
**I. Wave Force under Wave Crest**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	-1.0
- Bottom Upright Wall Level (CDL):	-3.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-5.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	3.457
- H <sub>max</sub> , m:	4.595
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	84.32



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

H<sub>D</sub> = H<sub>max</sub>

**5. Results of Calculation**

**5.1. Input data**

β (deg) =	30
cos β:	60.681
H <sub>D</sub> (m):	0.490
L (m):	4.595
LWL (CDL):	84.32
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-5.0
h (m):	-1.0
h' (m):	5.43
ρ <sub>0</sub> (t/m <sup>3</sup> ):	3.93
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	0.809
λ:	0.405
	Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

η* =	5.13
α <sub>1</sub> =	1.004
α <sub>3</sub> =	0.945
(1+cosβ)	1.490
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	4.728
p <sub>1</sub> (t/m <sup>2</sup> ) =	3.52
p <sub>c</sub> (at crest level), (t/m <sup>2</sup> ) =	3.45
p <sub>3</sub> (t/m <sup>2</sup> ) =	3.33

**5.3. Total wave pressure on Upright Wall**

<b>P</b> <sub>up to bottom Elev. (t/m) =</sub>	8.47
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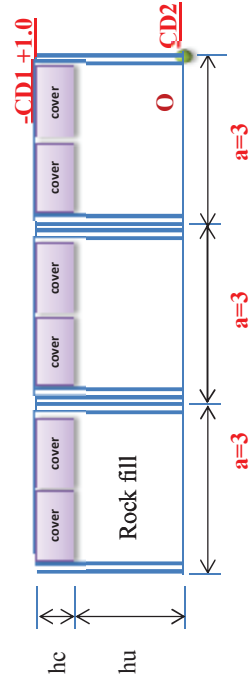
**II. Safety Factor**

μ (friction factor between conc. cell. stone & stone):	0.80
W <sub>con. cell. (total), t/m</sub>	21.87
W <sub>conc. cover (total), t/m</sub>	5.36

<b>S.F. at bottom Elev.</b>	2.57
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M <sub>p at O</sub>	10.65
M <sub>conc. cell. at O2 (=W*t)</sub>	220.97
<b>S.F. at O</b>	20.74

Sketch of detail of super structure  
LWL



CD1	30 years	-1 m
CD2		-3.5 m
LWL		+ 0.43 m
a		3 m
h		2.5 m
hu		2.00 m
b		2.00 m
t		0.20 m
hc		0.50 m
a <sub>inside</sub>		2.6 m
S <sub>conc. cell (1 cell.)</sub>		1.84 m <sup>2</sup>
γ <sub>conc.</sub>		2.4 t/m <sup>3</sup>
γ <sub>conc.</sub>		1.375 t/m <sup>3</sup>
W <sub>conc. cell (1 cell.)</sub>		6.33 t/cell.
S <sub>rubber filling (1 cell.)</sub>		4.16 m <sup>2</sup>
γ <sub>rubber stone</sub>		2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>		1.575 t/m <sup>3</sup>
void ratio		37%
W <sub>rubber filling (1 cell.)</sub>		8.26 t/cell.
W <sub>conc. cover side</sub>		1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	X <sub>O1</sub>	4.50 m
	M <sub>O1</sub>	220.97 tm/m

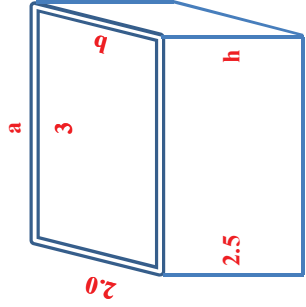
Wave	P <sub>up to bottom Elev.</sub>	8.47 t/m
	γ <sub>p upto bottom Elev.</sub>	1.258 m
	M <sub>p at O2</sub>	10.65 t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OC DI)

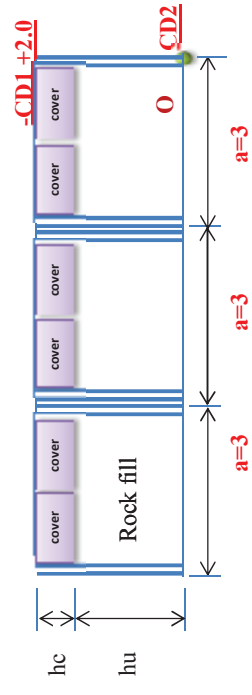
**I. Wave Force under Wave Crest**

**1. Input data:**

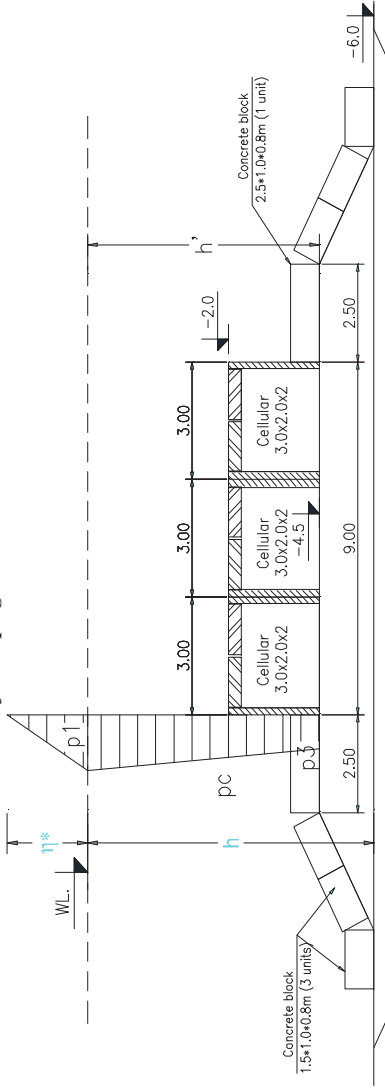
- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	-2.0
- Bottom Upright Wall Level (CDL):	-4.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-6.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.036
- H <sub>max</sub> , m:	5.296
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	88.23



Sketch of detail of super structure  
LWL



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0gH_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

H<sub>D</sub> = H<sub>max</sub>

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	60.374
H <sub>D</sub> (m):	0.494
L (m):	5.296
LWL (CDL):	88.23
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-6.0
h (m):	-2.0
h' (m):	6.43
$\rho_0$ (t/m <sup>3</sup> ):	4.93
g (m/s <sup>2</sup> ):	1.025
4 $\pi h/L$ :	9.81
2 $\pi h/L$ :	4.916
$\lambda$ :	0.458
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^*$ =	5.94
$\alpha_1$ =	0.981
$\alpha_3$ =	0.926
(1+cos $\beta$ )	1.494
$\alpha_1 \lambda \rho_0 H_D$	5.325
$p_1$ (t/m <sup>2</sup> ) =	3.98
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	3.83
$p_3$ (t/m <sup>2</sup> ) =	3.68

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	9.40
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W <sub>con. cell.</sub> (total), t/m	21.87
W <sub>conc. cover</sub> (total), t/m	5.36

Sliding	S.F. at bottom Elev.	2.32
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Overturnin g	M <sub>p at O</sub>	11.82
	M <sub>conc. cell. at O2 (=W*t)</sub>	220.97
	S.F. at O	18.69

CD1	30 years
CD2	-2 m
LWL	-4.5 m
a	+0.43 m
h	3 m
hu	2.5 m
hu	2.00 m
b	2.00 m
t	0.20 m
hc	0.50 m
a inside	2.6 m
S <sub>conc. cell</sub> (1 cell.)	1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)	6.33 t/cell.
S <sub>rubber filling</sub> (1 cell.)	4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$	2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell.)	8.26 t/cell.
W <sub>conc. cover side</sub>	1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	X <sub>O1</sub>	4.50 m
	M <sub>O1</sub>	220.97 tm/m

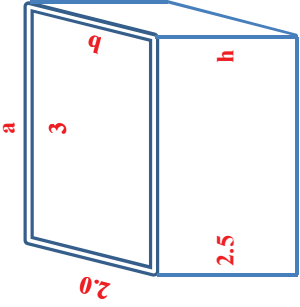
Wave	P <sub>up to bottom Elev.</sub>	9.40 t/m
	$\gamma_p$ upto bottom Elev.	1.258 m
	M <sub>p at O2</sub>	11.82 t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

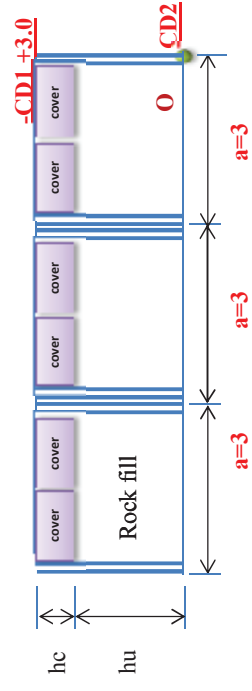
**I. Wave Force under Wave Crest**

**1. Input data:**

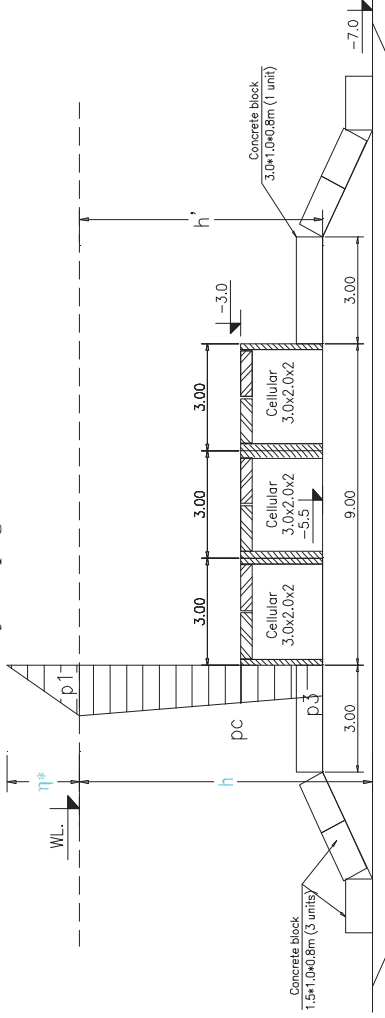
- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	-3.0
- Bottom Upright Wall Level (CDL):	-5.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-7.0
- Significant Wave Height ( $H_{1/3}$ ), m:	4.545
- $H_{max}$ , m:	5.922
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	95.19



*Sketch of detail of super structure*  
LWL



**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	59.653
$H_D$ (m):	0.505
L (m):	5.922
LWL (CDL):	95.19
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-7.0
h (m):	-3.0
h' (m):	7.43
$\rho_0$ (t/m <sup>3</sup> ):	5.93
g (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	0.981
$\lambda$ :	0.490
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	6.69
$\alpha_1 =$	0.966
$\alpha_3 =$	0.913
$(1 + \cos \beta)$	1.505
$\alpha_1 \lambda \rho_0 H_D$	5.866
$p_1$ (t/m <sup>2</sup> ) =	4.41
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	4.19
$p_3$ (t/m <sup>2</sup> ) =	4.03

**5.3. Total wave pressure on Upright Wall**

$P_{up}$ to bottom Elev. (t/m) =	10.28
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**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
W <sub>con. cell.</sub> (total), t/m	21.87
W <sub>conc. cover</sub> (total), t/m	5.36

Sliding **S.F. at bottom Elev. 2.12**

M <sub>p at O</sub>	12.93
M <sub>conc. cell. at O2 (=W*t)</sub>	220.97
<b>S.F. at O</b>	<b>17.09</b>

CD1	30 years
CD2	-3 m
LWL	-5.5 m
a	+0.43 m
h	3 m
hu	2.5 m
hc	2.00 m
b	2.00 m
t	0.20 m
hc	0.50 m
a inside	2.6 m
S <sub>conc. cell</sub> (1 cell.)	1.84 m <sup>2</sup>
$\gamma_{conc.}$	2.4 t/m <sup>3</sup>
$\gamma_{conc.}$	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)	6.33 t/cell.
S <sub>rubber filling</sub> (1 cell.)	4.16 m <sup>2</sup>
$\gamma_{rubber stone}$	2.6 t/m <sup>3</sup>
$\gamma_{rubber stone}$	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell.)	8.26 t/cell.
W <sub>conc. cover side</sub>	1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	X <sub>O1</sub>	4.50 m
	M <sub>O1</sub>	220.97 tm/m

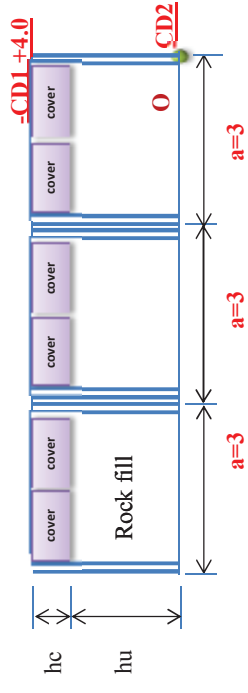
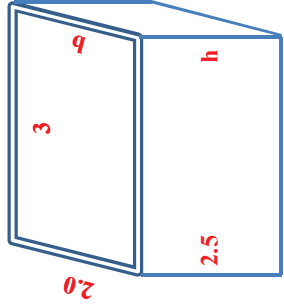
Wave	P <sub>up</sub> to bottom Elev.	10.28 t/m
	$\gamma_p$ upto bottom Elev.	1.258 m
	M <sub>p at O2</sub>	12.93 t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

**I. Wave Force under Wave Crest**

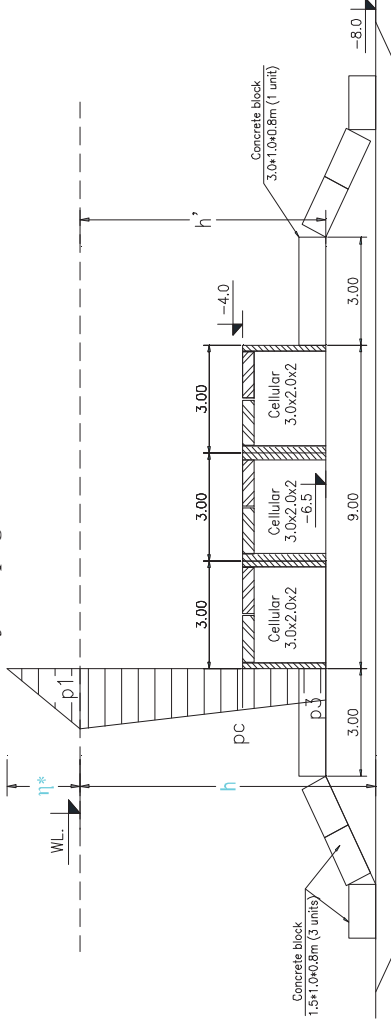
**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	-4.0
- Bottom Upright Wall Level (CDL):	-6.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-8.0
- Significant Wave Height (H <sub>1/3</sub> ), m:	4.956
- H <sub>max</sub> , m:	6.47
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	99.27



Sketch of detail of super structure  
LWL

**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta)\lambda\alpha_1\rho_0H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0 \text{)}$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

H<sub>D</sub> = H<sub>max</sub>

**5. Results of Calculation**

**5.1. Input data**

β (deg) =	30
cos β:	59.6
H <sub>D</sub> (m):	0.506
L (m):	6.47
LWL (CDL):	99.27
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-8.0
h (m):	-4.0
h' (m):	8.43
ρ <sub>0</sub> (t/m <sup>3</sup> ):	6.93
g (m/s <sup>2</sup> ):	1.025
4 πh/L:	9.81
2 πh/L:	1.067
λ:	0.534
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

η* =	7.31
α <sub>1</sub> =	0.947
α <sub>3</sub> =	0.895
(1+cosβ)	1.506
α <sub>1</sub> λ ρ <sub>0</sub> H <sub>D</sub>	6.278
p <sub>1</sub> (t/m <sup>2</sup> ) =	4.73
p <sub>c</sub> (at crest level), (t/m <sup>2</sup> ) =	4.41
p <sub>3</sub> (t/m <sup>2</sup> ) =	4.23

**5.3. Total wave pressure on Upright Wall**

P <sub>up to bottom Elev.</sub> (t/m) =	10.81
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**II. Safety Factor**

μ (friction factor between conc. cell. stone & stone):	0.80
W <sub>con. cell.</sub> (total), t/m	21.87
W <sub>conc. cover</sub> (total), t/m	5.36

<b>S.F. at bottom Elev.</b>	<b>2.02</b>
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M <sub>p at O</sub>	13.60
M <sub>conc. cell. at O2 (=W*t)</sub>	220.97
<b>S.F. at O</b>	<b>16.25</b>

CD1	30 years
CD2	-4 m
LWL	-6.5 m
a	+ 0.43 m
h	3 m
hu	2.5 m
hu	2.00 m
b	2.00 m
t	0.20 m
hc	0.50 m
a inside	2.6 m
S <sub>conc. cell</sub> (1 cell.)	1.84 m <sup>2</sup>
γ <sub>conc.</sub>	2.4 t/m <sup>3</sup>
γ <sub>conc.</sub>	1.375 t/m <sup>3</sup>
W <sub>conc. cell</sub> (1 cell.)	6.33 t/cell.
S <sub>rubber filling</sub> (1 cell.)	4.16 m <sup>2</sup>
γ <sub>rubber stone</sub>	2.6 t/m <sup>3</sup>
γ <sub>rubber stone</sub>	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling</sub> (1 cell.)	8.26 t/cell.
W <sub>conc. cover side</sub>	1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	X <sub>O1</sub>	4.50 m
	M <sub>O1</sub>	220.97 tm/m

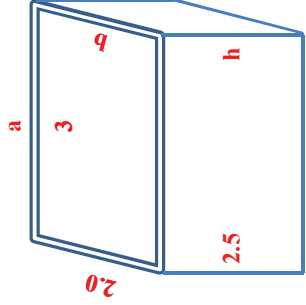
Wave	P <sub>up to bottom Elev.</sub>	10.81 t/m
	γ <sub>p upto bottom Elev.</sub>	1.259 m
	M <sub>p at O2</sub>	13.60 t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

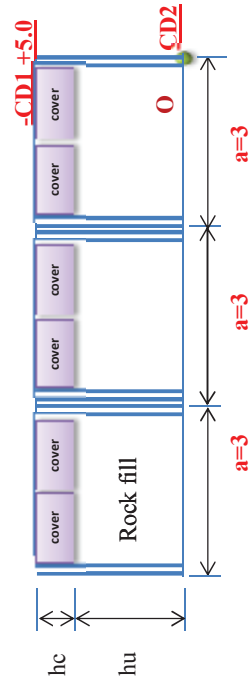
**I. Wave Force under Wave Crest**

**1. Input data:**

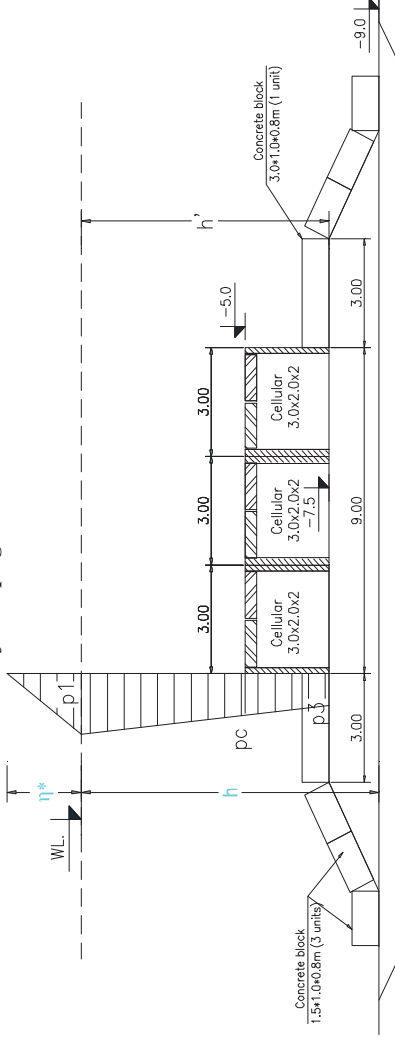
- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	-5.0
- Bottom Upright Wall Level (CDL):	-7.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-9.0
- Significant Wave Height ( $H_{1/3}$ ), m:	5.022
- $H_{max}$ , m:	7.17
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	104.19



Sketch of detail of super structure  
LWL



**2. Wave Pressure on the Front Face of an Upright Wall**



Wave Pressure Distribution Used in Design Calculation

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 g H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{\eta^*} \text{ (if } -h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	59.43
$H_D$ (m):	0.509
L (m):	7.17
LWL (CDL):	104.19
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-9.0
h (m):	-5.0
h' (m):	9.43
$\rho_0$ (t/m <sup>3</sup> ):	7.93
g (m/s <sup>2</sup> ):	1.025
4 $\pi h/L$ :	9.81
2 $\pi h/L$ :	1.137
$\lambda$ :	0.569
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	8.11
$\alpha_1 =$	0.931
$\alpha_3 =$	0.880
(1+ $\cos \beta$ )	1.509
$\alpha_1 \lambda \rho_0 H_D$	6.839
$p_1$ (t/m <sup>2</sup> ) =	5.16
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	4.74
$p_3$ (t/m <sup>2</sup> ) =	4.54

**5.3. Total wave pressure on Upright Wall**

$P_{\text{up to bottom Elev.}}$ (t/m) =	11.59
---	-------

**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
Wconc. cell. (total), t/m	21.87
Wconc. cover (total), t/m	5.36

Sliding **S.F. at bottom Elev. 1.9**

$M_{p \text{ at } O}$	14.59
$M_{\text{conc. cell. at } O2} (=W*t)$	220.97
<b>S.F. at O</b>	<b>15.14</b>

CD1	30 years
CD2	-5 m
LWL	-7.5 m
a	+0.43 m
h	3 m
hu	2.5 m
hu	2.00 m
b	2.00 m
t	0.20 m
hc	0.50 m
a inside	2.6 m
S <sub>conc. cell (1 cell.)</sub>	1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$	2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$	1.375 t/m <sup>3</sup>
W <sub>conc. cell (1 cell.)</sub>	6.33 t/cell.
S <sub>rubber filling (1 cell.)</sub>	4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$	2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$	1.575 t/m <sup>3</sup>
void ratio	37%
W <sub>rubber filling (1 cell.)</sub>	8.26 t/cell.
W <sub>conc. cover side</sub>	1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
	$X_{O1}$	4.50 m
	$M_{O1}$	220.97 tm/m

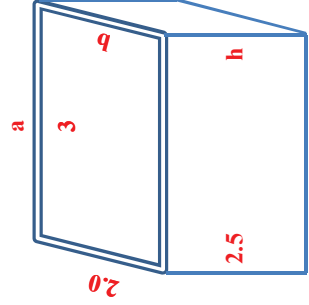
Wave	$P_{\text{up to bottom Elev.}}$	11.59 t/m
	$\gamma_{p \text{ upto bottom Elev.}}$	1.259 m
	$M_{p \text{ at } O2}$	14.59 t.m/m

**G. STABILITY CALCULATION DUE TO WAVE FORCE OF SAND PROTECTION TRAINING DIKE - Section for GL upto CD -3.0m**  
(Clause 5.5.2 - OCDI)

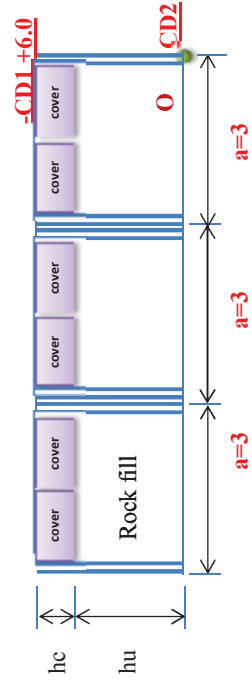
**I. Wave Force under Wave Crest**

**1. Input data:**

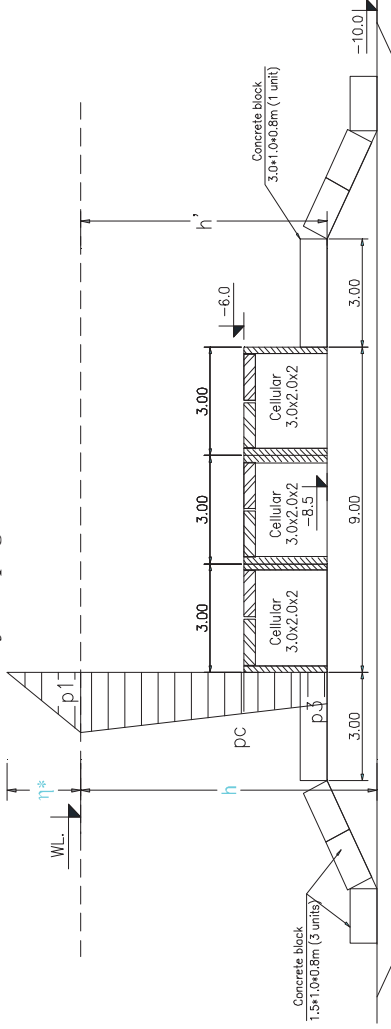
- Wave Return Period:	30 years
- Tide Level - LWL (CDL):	0.43
- Crest Level (CDL):	-6.0
- Bottom Upright Wall Level (CDL):	-8.5
- Bottom Width of Upright Wall (m):	9.0
- Back Fill Level (CDL):	
- Crest Level of Armor stone (CDL):	
- Seabed slope:	
- Seabed Surface Level (CDL):	-10.0
- Significant Wave Height ( $H_{1/3}$ ), m:	5.079
- $H_{max}$ , m:	7.818
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	108.50



*Sketch of detail of super structure*  
LWL



**2. Wave Pressure on the Front Face of an Upright Wall**



*Wave Pressure Distribution Used in Design Calculation*

At still water level:  $P_1 = 0.5(1 + \cos \beta) \lambda \alpha_1 \rho_0 H_D$

At toe of upright wall:  $P_3 = P_u = \alpha_3 P_1$

$$\lambda = \exp \left[ -10 \left( \frac{h}{L} \right)^{1.5} \left( 1 - \frac{h'}{h} \right)^5 \right]$$

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

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

$$\alpha_3 = 1 + \frac{h'}{h} \eta^* \quad (\text{if } -h' \leq 0)$$

**4. Wave Height and Wavelength Used in the Wave Pressure Calculation**

$H_D = H_{max}$   
breaking zone

**5. Results of Calculation**

**5.1. Input data**

$\beta$ (deg) =	30
$\cos \beta$ :	59.72
$H_D$ (m):	0.504
L (m):	7.818
LWL (CDL):	108.50
Sea bed Level (CDL):	0.43
Crest Level (CDL):	-10.0
h (m):	-6.0
h' (m):	10.43
$\rho_0$ (t/m <sup>3</sup> ):	8.93
g (m/s <sup>2</sup> ):	1.025
$4\pi h/L$ :	9.81
$2\pi h/L$ :	4.208
$\lambda$ :	0.604
	1.000 Upright wall

**5.2. Wave pressure on the front face of an Upright wall**

$\eta^* =$	8.82
$\alpha_1 =$	0.914
$\alpha_3 =$	0.864
$(1 + \cos \beta)$	1.504
$\alpha_1 \lambda \rho_0 H_D$	7.325
$p_1$ (t/m <sup>2</sup> ) =	5.51
$p_c$ (at crest level), (t/m <sup>2</sup> ) =	4.97
$p_3$ (t/m <sup>2</sup> ) =	4.76

**5.3. Total wave pressure on Upright Wall**

$P_{up \text{ to bottom Elev.}}$ (t/m) =	12.17
--	-------

**II. Safety Factor**

$\mu$ (friction factor between conc. cell. stone & stone):	0.80
Wconc. cell. (total), t/m	21.87
Wconc. cover (total), t/m	5.36

Sliding	S.F. at bottom Elev.	1.79
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Overturmin g	$M_{p \text{ at O}}$	15.32
	$M_{\text{conc. cell. at O2}} (=W*t)$	220.97
	S.F. at O	14.42

CD1	30 years	-6 m
CD2		-8.5 m
LWL		+0.43 m
a		3 m
h		2.5 m
hu		2.00 m
b		2.00 m
t		0.20 m
hc		0.50 m
a inside		2.6 m
S <sub>conc. cell (1 cell.)</sub>		1.84 m <sup>2</sup>
$\gamma_{\text{conc.}}$		2.4 t/m <sup>3</sup>
$\gamma_{\text{conc.}}$		1.375 t/m <sup>3</sup>
W <sub>conc. cell (1 cell.)</sub>		6.33 t/cell.
S <sub>rubber filling (1 cell.)</sub>		4.16 m <sup>2</sup>
$\gamma_{\text{rubber stone}}$		2.6 t/m <sup>3</sup>
$\gamma_{\text{rubber stone}}$		1.575 t/m <sup>3</sup>
void ratio		37%
W <sub>rubber filling (1 cell.)</sub>		8.26 t/cell.
W <sub>conc. cover side</sub>		1.79 t/m

Upper Layer	Nos. of cellular block	3.00 unit
$X_{O1}$		4.50 m
$M_{O1}$		220.97 tm/m

Wave	$P_{up \text{ to bottom Elev.}}$	12.17 t/m
	$\gamma_p$ upto bottom Elev.	1.259 m
	$M_{p \text{ at O2}}$	15.32 t.m/m

**14. Summary of Cellular caisson dimension & Safety factor**

**SUMMARY OF CELLULAR DIMENSION of CASE B-1&2**

No.	Alternative	Seabed Elev. (m)	Cellular dimension				Remark
			width (m)	length (m)	height (m)	thickness (m)	
1	CASE B-1	0.0	3.00	2.00	1.50	0.20	Single layer cellular
2		-1.0	3.00	2.00	2.00	0.20	Single layer cellular
3		-2.0	3.00	2.00	3.00	0.20	Single layer cellular
4		-2.0	3.00	2.00	3.50	0.20	Single layer cellular
5		-3.0	3.00	2.00	1.50	0.20	Double layer cellular
6		-4.0	3.00	2.00	2.50	0.20	Double layer cellular
7	CASE B-2	-5.0	3.00	2.00	2.75	0.20	Double layer cellular
7		0.0	3.00	2.00	1.50	0.20	Single layer cellular
8		-1.0	3.00	2.00	1.50	0.20	Single layer cellular
9		-2.0	3.00	2.00	2.50	0.20	Single layer cellular
10		-3.0	3.00	2.00	2.50	0.20	Single layer cellular
11		-4.0	3.00	2.00	2.50	0.20	Single layer cellular
12	-5.0	3.00	2.00	2.50	0.20	Single layer cellular	

**SUMMARY OF CELLULAR DIMENSION of CASE A-1&2**

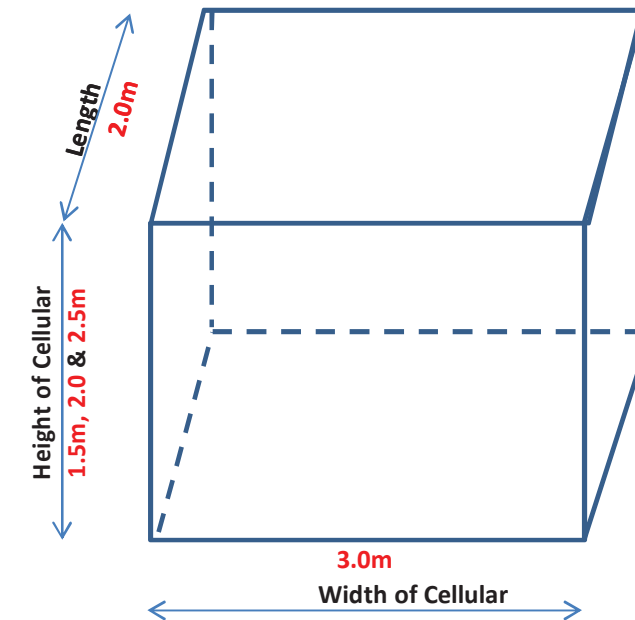
No.	Alternative	Seabed Elev. (m)	Cellular dimension			
			width (m)	length (m)	height (m)	thickness (m)
1	CASE A-1	0.0	2.50	1.00	2.00	0.20
2		-1.0	2.50	1.00	2.00	0.20
3		-2.0	2.50	1.00	2.20	0.20
4		-3.0	3.00	1.00	2.20	0.20
5		-4.0	3.00	1.00	2.50	0.20
6		-5.0	3.00	1.00	2.50	0.20
7	CASE A-2	0.0	2.50	1.00	2.00	0.20
8		-1.0	2.50	1.00	2.00	0.20
9		-2.0	2.50	1.00	2.20	0.20
10		-3.0	3.00	1.00	2.20	0.20
11		-4.0	3.00	1.00	2.50	0.20
12		-5.0	3.00	1.00	2.50	0.20

**SUMMARY OF CELLULAR STABILITY COEFFICIENT of CASE B-1&2**

No.	Alternative	Seabed Elev. (m)	SAFETY FACTORS	
			SLIDING	OVERTURNING
1	CASE B-1	0.0	1.95	9.70
2		-1.0	1.81	10.10
3		-2.0	1.57	5.00
4		-3.0	1.38	6.98
5		-4.0	1.30	4.71
6		-5.0	1.83	5.60
7	CASE B-2	0.0	2.25	17.89
8		-1.0	2.08	16.53
9		-2.0	2.56	20.62
10		-3.0	2.29	18.49
11		-4.0	1.84	14.86
12		-5.0	2.00	16.15

**SUMMARY OF CELLULAR TYPES**

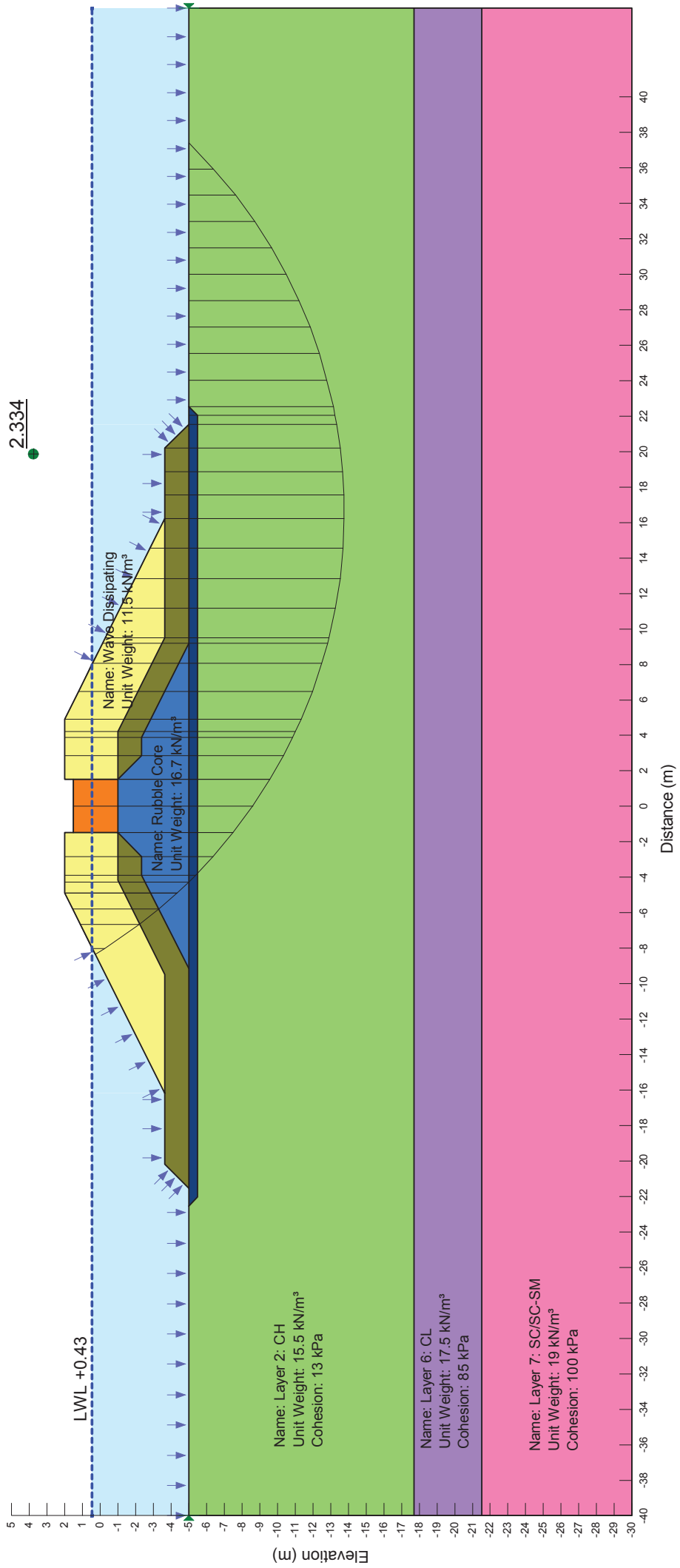
Alternative	Cellular dimension				Remark
	width (m)	length (m)	height (m)	thickness (m)	
CASE B-1	3.0	2.0	2.00	0.2	Cellular Type 1
	3.0	2.0	2.50	0.2	Cellular Type 2
	3.0	2.0	2.75	0.2	Cellular Type 3
	3.0	2.0	3.00	0.2	Cellular Type 4
	3.0	2.0	3.50	0.2	Cellular Type 5
CASE B-2	3.0	2.0	1.5	0.2	Cellular Type 1
	3.0	2.0	2.5	0.2	Cellular Type 3



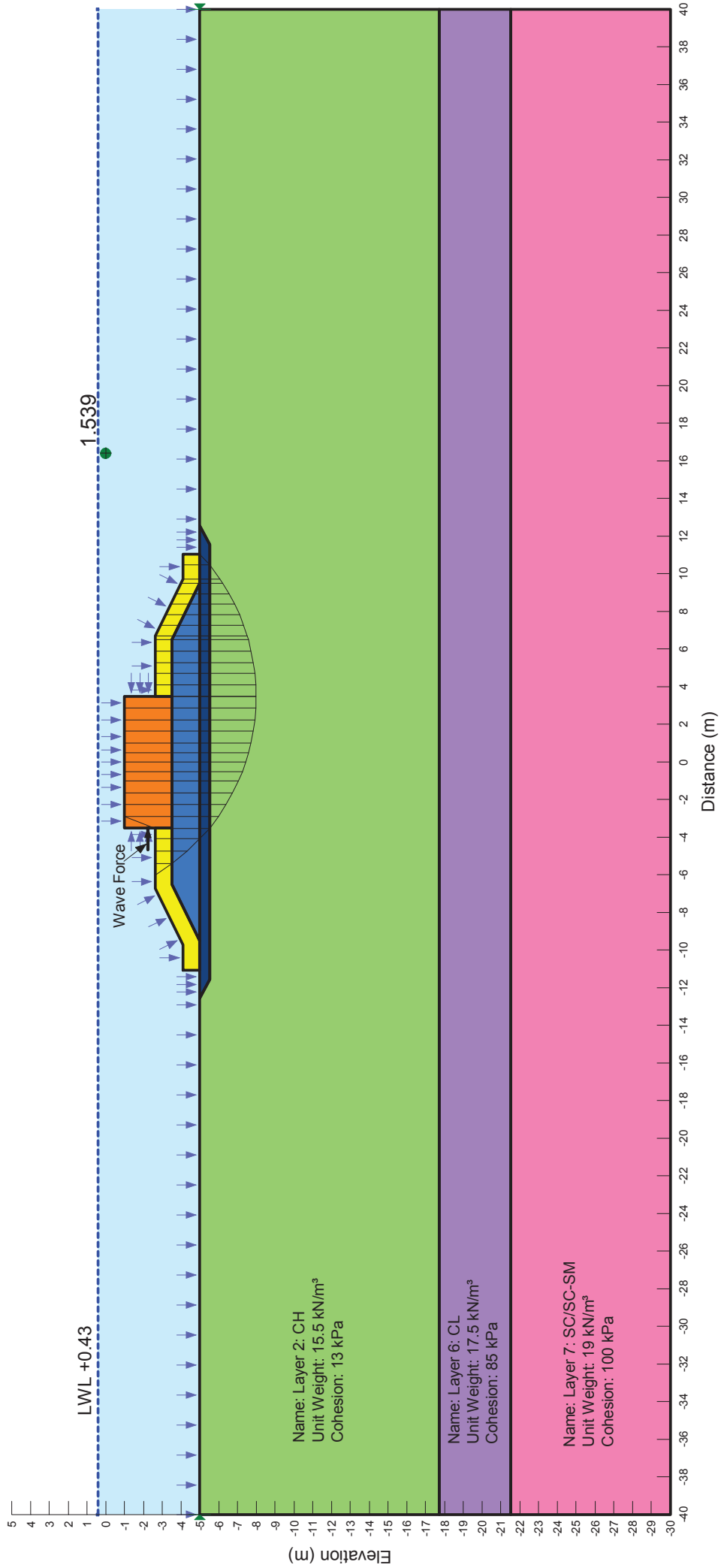


**15 SLOPE STABILITY ANALYSIS OF SAND PROTECTION TRAINING DIKE - G.L. -5.0m**

**SLOPE STABILITY OF SAND PROTECTION DIKE  
CASE A: RUBBLE MOUND CELLULAR COVERS BY TETRAPOD BLOCKS  
TYPICAL CROSS SECTION GL.-5.0 (BOREHOLE - D26)**

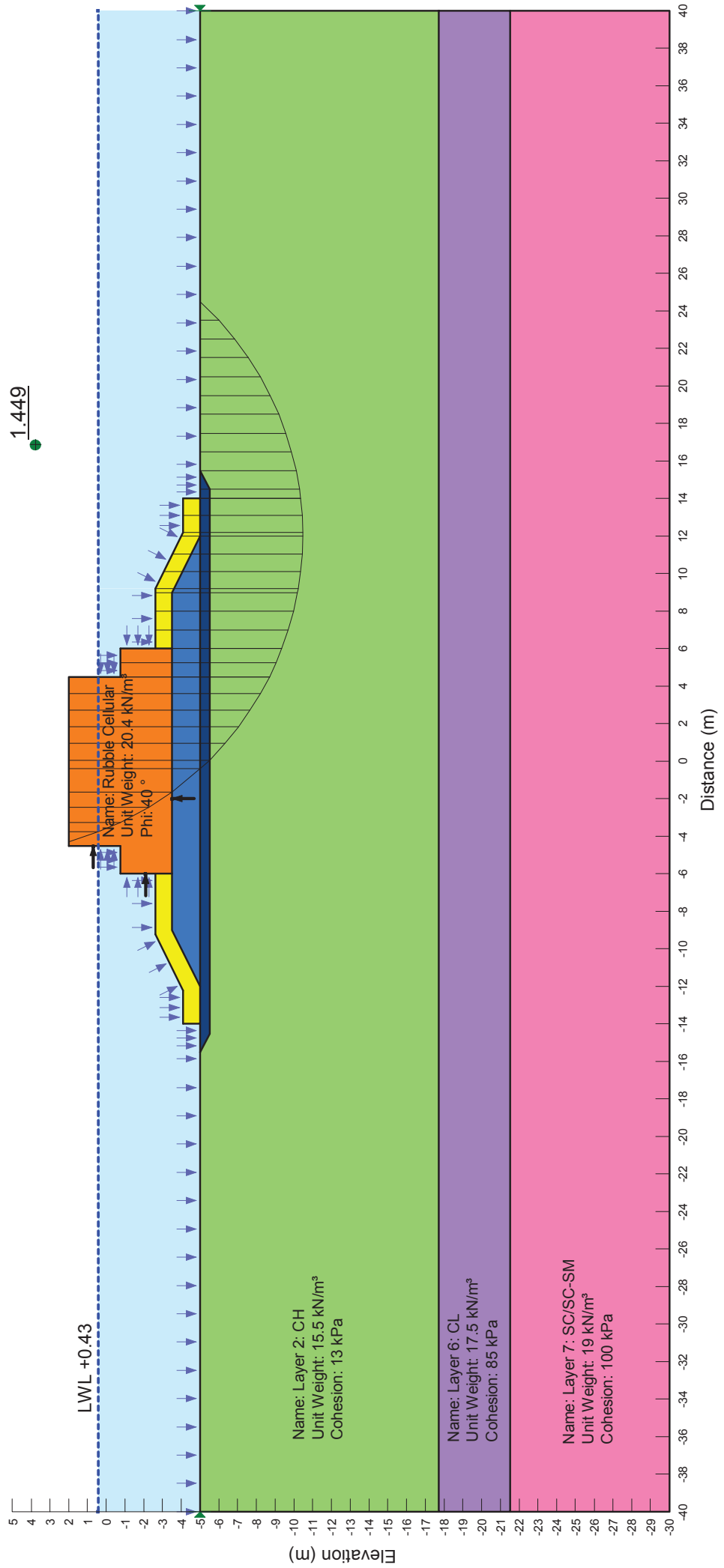


**SLOPE STABILITY OF SAND PROTECTION DIKE  
 ALTERNATIVE: RUBBLE MOUND CONCRETE CELLULAR  
 TYPICAL CROSS SECTION GL.-5.0 (BOREHOLE - D26)**



Phụ lục 19-15-2

# SLOPE STABILITY OF SAND PROTECTION DIKE CASE B: RUBBLE MOUND CONCRETE CELLULAR TYPICAL CROSS SECTION (BOREHOLE: D26)



## 16. EXTERNAL FORCE CALCULATION OF CELLULAR CAISSON

### I. CELLULAR TYPE 1:

#### Cellular Dimension:

Width, B:	3 m
Length, L:	2 m
Height, H:	2 m
Wall thickness, t:	0.2 m

#### Parameter of material filling:

$\gamma_s$	2.6 t/m <sup>3</sup>
$\gamma'_s$	1.575 t/m <sup>3</sup>
void ratio:	37%
$\gamma'_{sub}$	0.99 t/m <sup>3</sup>
Internal angle:	40 °

#### Earth Pressure on Cellular wall:

Live load on top of cellular:	0.5 t/m <sup>2</sup>
H	2 m
Ka	0.217
$p_a$ at top	0.11 t/m
$p_a$ at bottom	0.54 t/m

#### Wave Pressure on Cellular wall:

$p_{at}$ top	2.98 t/m	<i>See in section D</i>
$p_{at}$ bottom	2.90 t/m	<i>See in section D</i>

#### Combination of horizontal pressure:

$p_{at}$ top	2.87 t/m
$p_{at}$ bottom	2.36 t/m
$p_{average}$	2.62 t/m

#### Internal force of Cellular wall:

Moment of faceline direction	
$M^- =$	-0.87 tm/m
$M^+ =$	0.44 tm/m

### II. CELLULAR TYPE 2:

#### Cellular Dimension:

Width, B:	3 m
Length, L:	2 m
Height, H:	2.5 m
Wall thickness, t:	0.2 m

**Parameter of material filling:**

$\gamma_s$	2.6 t/m <sup>3</sup>
$\gamma'_s$	1.575 t/m <sup>3</sup>
void ratio:	0.37
$\gamma'_{sub}$	0.99 t/m <sup>3</sup>
Internal angle:	40 °

**Earth Pressure on Cellular wall:**

Live load on top of cellular:	0.5 t/m <sup>2</sup>
H	2.5 m
Ka	0.217
p <sub>a</sub> at top	0.11 t/m
p <sub>a</sub> at bottom	0.65 t/m

**Wave Pressure on Cellular wall:**

p <sub>at</sub> top	4.33 t/m	<i>See in section G</i>
p <sub>at</sub> bottom	4.17 t/m	<i>See in section G</i>

**Combination of horizontal pressure:**

p <sub>at</sub> top	4.22 t/m
p <sub>at</sub> bottom	3.52 t/m
p <sub>average</sub>	3.87 t/m

**Internal force of Cellular wall:**

Moment of faceline direction	
$M^- = p_{average} \cdot L^2/12$	-1.29 tm/m
$M^+ = p_{average} \cdot L^2/24$	0.65 tm/m

**III. CELLULAR TYPE 3:****Cellular Dimension:**

Width, B:	3 m
Length, L:	2 m
Height, H:	3 m
Wall thickness, t:	0.2 m

**Parameter of material filling:**

$\gamma_s$	2.6 t/m <sup>3</sup>
$\gamma'_s$	1.575 t/m <sup>3</sup>
void ratio:	0.37
$\gamma'_{sub}$	0.99 t/m <sup>3</sup>
Internal angle:	40 °

**Earth Pressure on Cellular wall:**

Live load on top of cellular:	0.5 t/m <sup>2</sup>
-------------------------------	----------------------

H	3 m
Ka	0.217
P <sub>a</sub> at top	0.11 t/m
P <sub>a</sub> at bottom	0.76 t/m

**Wave Pressure on Cellular wall:**

P <sub>at</sub> top	3.47 t/m	<i>See in section G</i>
P <sub>at</sub> bottom	3.32 t/m	<i>See in section G</i>

**Combination of horizontal pressure:**

P <sub>at</sub> top	3.36 t/m
P <sub>at</sub> bottom	2.57 t/m
P <sub>average</sub>	2.96 t/m

**Internal force of Cellular wall:**

Moment of faceline direction	
$M^- = p_{average} \cdot L^2 / 12$	-0.99 tm/m
$M^+ = p_{average} \cdot L^2 / 24$	0.49 tm/m

**IV. CELLULAR TYPE 4:**

**Cellular Dimension:**

Width, B:	3 m
Length, L:	2 m
Height, H:	2.75 m
Wall thickness, t:	0.2 m

**Parameter of material filling:**

$\gamma_s$	2.6 t/m <sup>3</sup>
$\gamma'_s$	1.575 t/m <sup>3</sup>
void ratio:	0.37
$\gamma'_{sub}$	0.99 t/m <sup>3</sup>
Internal angle:	40 °

**Earth Pressure on Cellular wall:**

Live load on top of cellular:	0.5 t/m <sup>2</sup>
H	2.75 m
Ka	0.217
P <sub>a</sub> at top	0.11 t/m
P <sub>a</sub> at bottom	0.70 t/m

**Wave Pressure on Cellular wall:**

P <sub>at</sub> top	4.65 t/m	<i>See in section G</i>
P <sub>at</sub> bottom	4.46 t/m	<i>See in section G</i>

**Combination of horizontal pressure:**

$p_{at\ top}$	4.54 t/m
$p_{at\ bottom}$	3.75 t/m
$p_{average}$	4.15 t/m

**Internal force of Cellular wall:**

Moment of faceline direction

$$M^- = p_{average} \cdot L^2 / 12 \quad -1.38 \text{ tm/m}$$

$$M^+ = p_{average} \cdot L^2 / 24 \quad 0.69 \text{ tm/m}$$

**V. CELLULAR TYPE 5:**

**Cellular Dimension:**

Width, B:	3 m
Length, L:	2 m
Height, H:	3.5 m
Wall thickness, t:	0.2 m

**Parameter of material filling:**

$\gamma_s$	2.6 t/m <sup>3</sup>
$\gamma'_s$	1.575 t/m <sup>3</sup>
void ratio:	0.37
$\gamma'_{sub}$	0.99 t/m <sup>3</sup>
Internal angle:	40 °

**Earth Pressure on Cellular wall:**

Live load on top of cellular:	0.5 t/m <sup>2</sup>
H	3.5 m
Ka	0.217
$p_a$ at top	0.11 t/m
$p_a$ at bottom	0.86 t/m

**Wave Pressure on Cellular wall:**

$p_{at\ top}$	3.47 t/m	<i>See in section G</i>
$p_{at\ bottom}$	3.30 t/m	<i>See in section G</i>

**Combination of horizontal pressure:**

$p_{at\ top}$	3.36 t/m
$p_{at\ bottom}$	2.43 t/m
$p_{average}$	2.90 t/m

**Internal force of Cellular wall:**

Moment of faceline direction

$$M^- = p_{average} \cdot L^2 / 12 \quad -0.97 \text{ tm/m}$$

$$M^+ = p_{average} \cdot L^2 / 24 \quad 0.48 \text{ tm/m}$$



## 17. REINFORCEMENT CALCULATION OF CELLULAR CAISSON

(According to TCVN 4116-85)

### GENERAL PARAMETER

Construction Grade:	<b>III</b>
Concrete Grade:	<b>M400</b>
Re-bar Grade:	<b>AII</b>
<b>STRENGTH STATE:</b>	
$R_a =$	2700.00 kG/cm <sup>2</sup>
$E_a =$	2.10E+06 kG/cm <sup>2</sup>
$R_n =$	175.00 kG/cm <sup>2</sup>
$R_k =$	12.00 kG/cm <sup>2</sup>
<b>CRACKING WIDTH STATE:</b>	
$R_a =$	2700.00 kG/cm <sup>2</sup>
$E_a =$	2.10E+06 kG/cm <sup>2</sup>
$R_n =$	225.00 kG/cm <sup>2</sup>
$R_k =$	18.00 kG/cm <sup>2</sup>
$[a_t] =$	0.080 mm

### KIND OF STRUCTURE: CELLULAR WALL

Position checking	Support	Span
Width of calculated wall strip, b (cm)	100.00	100.00
Thickness of wall, h (cm)	20.00	20.00
Height of wall, h <sub>0</sub> (cm)	14.30	14.30
Bending Moment (t.m)	<b>1.38</b>	<b>0.69</b>
Bar diameter, φ (mm)	<b>14</b>	<b>14</b>
Area of reinforcement, f <sub>a</sub> (cm <sup>2</sup> )	1.54	1.54
Maximum of bar spacing, s (cm)	<b>20</b>	<b>20</b>
No. of bar, n (bar)	5	5
Total area of reinforcement, F <sub>a</sub> (cm <sup>2</sup> )	7.70	7.70
Minimum cover to tension reinforcement, a (cm)	5.00	5.00
Distance to center of bar, a <sub>0</sub> (cm)	5.70	5.70
Percentage of re-bar, m (%)	0.54%	0.54%
Checking of re-bar percentage	<b>OK</b>	<b>OK</b>
Height of compression concrete area, ξ (cm)	1.19	1.19
z = h <sub>0</sub> - ξ/2 (cm)	13.71	13.71
Reinforcement stress, σ <sub>a</sub> (kG/cm <sup>2</sup> )	1311.10	655.551
Checking of reinforcement stress	<b>OK</b>	<b>OK</b>
Cracking width, a <sub>t</sub> (mm)	0.074	0.037
Cracking width allowance, [a <sub>t</sub> ] (mm)	0.08	0.08
Checking of cracking width	<b>OK</b>	<b>OK</b>

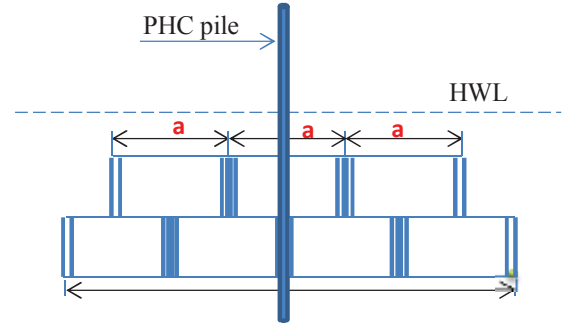
Phụ lục 19-18

Tính toán cọc PHC làm móng cột đèn báo hiệu

**CALCULATION OF WAVE FORCE ON SINGLE PILE (According to EM 1110-2-1100)**  
**Sand Protection Training Dike - Section for GL upto CD -5.0m**

**1. Input data:**

- Wave Return Period:	30 years
- Tide Level - HWL (CDL):	3.55
- Crest Level (CDL):	2.0
- Crest Level of Beacon's pile (CDL):	5.0
- Seabed Surface Level (CDL):	-5.0
- Significant Wave Height ( $H_{1/3}$ ), m:	4.96
- $H_{max}$ , m:	6.615
- Wave Period (T), sec:	12.8
- Wave Length (L), m:	100.31



**2. Wave Pressure on the single pile**

According to Morison et al. (1950)'s equation:

**Total wave force:  $F = F_i + F_D$**

Where in:

$F_i$  - the force due of inertia

$$F_i = C_M \rho g \frac{\pi D^2}{4} H K_i \quad (VI - 5 - 285)$$

$F_D$  - the force due to drag

$$F_D = C_D \frac{1}{2} \rho g D H^2 K_D \quad (VI - 5 - 286)$$

k - wave number (rad/m),  $k = \frac{2\pi}{L}$

$\omega$  - wave frequency (rad/s),  $\omega = \frac{2\pi}{T} = (gk \cdot \tanh kd)^{\frac{1}{2}}$

$$\eta = \frac{H}{2} \cos \frac{2\pi t}{T} = \frac{H}{2} \cos \omega t$$

$\rho$  - density of seawater (normally 1.025 t/m<sup>3</sup>)

D - diameter of pipe pile

$C_D$  - drag coefficient, = 1

$C_M$  - inertia coefficient, = 2

**Similarly, Total moment:  $M = M_i + M_D$**

$$M_i = C_M \rho g \frac{\pi D^2}{4} H K_i d S_i = F_i d S_i \quad (VI - 5 - 287)$$

$$M_D = C_D \frac{1}{2} \rho g D H^2 K_D d S_D = F_D d S_D \quad (VI - 5 - 288)$$

$$K_i = \frac{1}{2} \tanh \left( \frac{2\pi d}{L} \right) \sin \left( -\frac{2\pi t}{T} \right) = \frac{1}{2} \tanh(kd) \sin(-\omega t) \quad (VI - 5 - 294)$$

$$K_D = \frac{1}{8} \left( 1 + \frac{4\pi d/L}{\sinh[4\pi d/L]} \right) \cos \left( \frac{2\pi t}{T} \right) \left| \cos \left( \frac{2\pi t}{T} \right) \right| = \frac{1}{8} \left( 1 + \frac{2kd}{\sinh 2kd} \right) \cos(\omega t) |\cos(\omega t)| \quad (VI - 5 - 295)$$

$$S_i = 1 + \frac{1 - \cosh[2\pi d/L]}{(2\pi d/L) \sinh[2\pi d/L]} = 1 + \frac{1 - \cosh kd}{kd \sinh kd} \quad (VI - 5 - 296)$$

$$S_D = \frac{1}{2} + \frac{1}{2n} \left( \frac{1}{2} + \frac{1 - \cosh[4\pi d/L]}{(4\pi d/L) \sinh[4\pi d/L]} \right) = \frac{1}{2} + \frac{1}{2n} \left( \frac{1}{2} + \frac{1 - \cosh 2kd}{2kd \sinh 2kd} \right) \quad (VI - 5 - 297)$$

$$n = \frac{1}{2} \left( 1 + \frac{4\pi d/L}{\sinh[4\pi d/L]} \right) = \frac{1}{2} \left( 1 + \frac{2kd}{\sinh 2kd} \right) \quad (VI - 5 - 298)$$

Vary  $t = (0, T/4, T) \Rightarrow \omega t = (0, \pi/2, 2\pi) = (0, 1, 1.571, 2, 3, 4, 5, 6, 7)$

d (m)	k (rad/m)	$\omega$ (rad/s)	Ts (s)	$C_D$	$C_M$	$\rho$ (kN/m <sup>3</sup> )	D (m)
1.6	0.063	0.244	8.60	1	2	10.25	0.7

$K_i$	$K_D$	$S_i$	$S_D$	n	$\eta$
$0.048\sin(-\omega t)$	$0.249\cos(\omega t)/\cos(\omega t)/$	0.50	0.50	0.997	$2.48\cos(\omega t)$

$\omega t$	$K_i$	$K_D$	$F_i$ (t), kN	$F_D$ (t), kN	<b>F (t), kN</b>	$M_i$ (t), kNm	$M_D$ (t), kNm	<b>M (t), kNm</b>
0	0.00	0.249	0.0	215.8	<b>215.8</b>	0.0	167.5	<b>167.5</b>
1	-0.04	0.073	-15.6	63.0	<b>47.4</b>	-12.1	48.9	<b>36.8</b>
1.571	-0.05	0.000	-18.6	0.0	<b>-18.6</b>	-14.4	0.0	<b>-14.4</b>
2	-0.04	-0.043	-16.9	-37.4	<b>-54.3</b>	-13.1	-29.0	<b>-42.1</b>
3	-0.01	-0.244	-2.6	-211.5	<b>-214.1</b>	-2.0	-164.2	<b>-166.2</b>
4	0.04	-0.106	14.1	-92.2	<b>-78.1</b>	10.9	-71.6	<b>-60.7</b>
5	0.05	0.020	17.8	17.4	<b>35.2</b>	13.8	13.5	<b>27.3</b>
6	0.01	0.230	5.2	198.9	<b>204.1</b>	4.0	154.4	<b>158.4</b>
7	-0.03	0.142	-12.2	122.6	<b>110.4</b>	-9.5	95.2	<b>85.7</b>

Summarize of maximum wave force on pipe pile

F (t), kN	M (t), kNm	Safety factor	F (kN)	M (kNm)
215.8	167.5	<b>2.5</b>	<b>539.4</b>	<b>418.7</b>

Selecting PHC Pile (JIS A5337)

Dia (mm)	Thk (mm)	Type	Bending moment (kNm)	
			Cracking	Breaking
700	110	C	441.5	882.9

### 3. Bearing capacity checking

Location: **B1** Borehole: **D6**  
 Self weight of beacon marker  
 Dead weight of light beacon : 3.4 kN  
 Copping conection: 1.0 kN

PHC pile weight

Dimensions		Unit weight (kg/m)	Elevation of pile		Length (m)	Weight per pile (kN)
Dia (mm)	Thk (mm)		Top (m CD)	Bottom (m CD)		
700	110	509.7	5	-15	20	100.01

Negative friction on steel pile

	Elevation		Layer Thk m	Soil Condition		f 2N of Cu	As m <sup>2</sup>	F per pile kN
	Top m CD	Bottom m CD		SPT-N	Cu kPa			
Layer 1b	-2.4	-3.3	0.9	4.5		9.0	1.98	17.8
Layer 2	-3.3	-11.2	7.9	0.8	12.5	12.5	17.37	217.9
Layer 3b	-11.2	-15.0	3.8	7.8		15.6	8.36	130.4
Total								<b>366.1</b>

Total downward load:  $F_{down} = 470.5$  kN

Bearing capacity

$$R_u = 300 \times N \times A_p \quad R_u = 300N A_p + 2\bar{N} A_s$$

	Elevation		Layer Thk m	Ni	Ap m <sup>2</sup>	As m <sup>3</sup>	Rs per pile kN	Rtoe per pile kN
	Top m CD	Bottom m CD						
Rubble layer	2.0	-2.4	4.4	10	0.38	9.68	193.5	
Layer 1b	-2.4	-3.3	0.9	4.5	0.38	1.98	17.8	
Layer 2	-3.3	-11.2	7.9	0.8	0.38	17.37	27.8	
Layer 3b	-11.2	-15.6	3.8	7.8	0.38	8.36	130.4	900.5
Total, Ru:							<b>1270.0</b>	

$$R_a = R_u / 2.5 = 508.0 \text{ kN} > F_{down} \quad \text{OK}$$

**Bearing capacity checking**

Location: **B2** Borehole: **D11**

Self weight of beacon marker

Dead weight of light beacon : 3.4 kN

Copping conection: 1.0 kN

**PHC pile weight**

Dimensions		Unit weight (kg/m)	Elevation of pile		Length (m)	Weight per pile (kN)
Dia (mm)	Thk (mm)		Top (m CD)	Bottom (m CD)		
700	110	509.7	5	-15	20	100.01

**Negative friction on steel pile**

	Elevation		Layer Thk m	Soil Condition		f 2N of Cu	As m2	F per pile kN
	Top m CD	Bottom m CD		SPT-N	Cu kPa			
Layer 1a	-1.94	-6.74	4.8	4.8		9.6	10.56	101.3
Layer 2	-6.74	-12.64	5.9	0.8	12.5	12.5	12.97	162.8
Layer 3b	-12.64	-18.7	2.36	7.8		15.6	5.19	81.0
<b>Total</b>								<b>345.1</b>

Total downward load:  $F_{down} = 449.5$  kN

Bearing capacity

$R_u = 300 \times N \times A_p$        $R_u = 300N A_p + 2\bar{N} A_s$

	Elevation		Layer Thk m	Ni	Ap m2	As m3	Rs per pile kN	Rtoe per pile kN
	Top m CD	Bottom m CD						
Rubble layer	2.0	-1.94	3.94	10	0.38	8.66	173.3	
Layer 1a	-1.94	-6.74	4.8	4.8	0.38	10.56	101.3	
Layer 2	-6.74	-12.64	5.9	0.8	0.38	12.97	20.8	
Layer 3b	-12.64	-18.74	2.36	7.8	0.38	5.19	81.0	900.5
<b>Total, Ru:</b>							<b>1276.9</b>	

$R_a = R_u / 2.5 = 510.8$  kN >  $F_{down}$       **OK**

**Bearing capacity checking**

Location: **B3** Borehole: **D16**

Self weight of beacon marker

Dead weight of light beacon : 3.4 kN

Copping conection: 1.0 kN

PHC pile weight

Dimensions		Unit weight (kg/m)	Elevation of pile		Length (m)	Weight per pile (kN)
Dia (mm)	Thk (mm)		Top (m CD)	Bottom (m CD)		
700	110	509.7	5	-15	20	100.01

Negative friction on steel pile

	Elevation		Layer Thk m	Soil Condition		f 2N of Cu	As m2	F per pile kN
	Top m CD	Bottom m CD		SPT-N	Cu kPa			
Layer 1a	-1.77	-3.57	1.8	4.8		9.6	3.96	38.0
Layer 2	-3.57	-13.57	10	0.8	12.5	12.5	21.99	275.9
Layer 3b	-13.57	-20.4	1.43	7.8		15.6	3.14	49.1
<b>Total</b>								<b>362.9</b>

Total downward load:  $F_{down} = 467.4$  kN

Bearing capacity

$R_u = 300 \times N \times A_p$        $R_u = 300N A_p + 2\bar{N} A_s$

	Elevation		Layer Thk m	Ni	Ap m2	As m3	Rs per pile kN	Rtoe per pile kN
	Top m CD	Bottom m CD						
Rubble layer	2.0	-1.77	3.77	10	0.38	8.29	165.8	
Layer 1a	-1.77	-3.57	1.8	4.8	0.38	3.96	38.0	
Layer 2	-3.57	-13.57	10	0.8	0.38	21.99	35.2	
Layer 3b	-13.57	-20.37	1.43	7.8	0.38	3.14	49.1	900.5
<b>Total, Ru:</b>							<b>1188.6</b>	

$R_a = R_u / 2.5 = 475.4$  kN >  $F_{down}$     **OK**

**Bearing capacity checking**

Location: **B4** Borehole: **D21**

Self weight of beacon marker

Dead weight of light beacon : 3.4 kN

Copping conection: 1.0 kN

**PHC pile weight**

Dimensions		Unit weight (kg/m)	Elevation of pile		Length (m)	Weight per pile (kN)
Dia (mm)	Thk (mm)		Top (m CD)	Bottom (m CD)		
700	110	509.7	5	-15	20	100.01

**Negative friction on steel pile**

	Elevation		Layer Thk m	Soil Condition		f 2N of Cu	As m2	F per pile kN
	Top m CD	Bottom m CD		SPT-N	Cu kPa			
Layer 1b	-2.07	-3.67	1.6	4.5		9.0	3.52	31.7
Layer 2	-3.67	-13.77	10.1	0.8	12.5	12.5	22.21	278.6
Layer 4	-13.77	-19.0	1.23	10.7		21.4	2.70	57.9
<b>Total</b>								<b>368.2</b>

Total downward load:  $F_{down} = 472.6$  kN

Bearing capacity

$R_u = 300 \times N \times A_p$        $R_u = 300N A_p + 2\bar{N} A_s$

	Elevation		Layer Thk m	Ni	Ap m2	As m3	Rs per pile kN	Rtoe per pile kN
	Top m CD	Bottom m CD						
Rubble layer	2.0	-2.07	4.07	10	0.38	8.95	179.0	
Layer 1b	-2.07	-3.67	1.6	4.5	0.38	3.52	31.7	
Layer 2	-3.67	-13.77	10.1	0.8	0.38	22.21	35.5	
Layer 4	-13.77	-18.97	1.23	10.7	0.38	2.70	57.9	1235.4
<b>Total, Ru:</b>							<b>1539.5</b>	

$R_a = R_u / 2.5 = 615.8$  kN >  $F_{down}$       **OK**



**Bearing capacity checking**Location: **B5** Borehole: **D26**

Self weight of beacon marker

Dead weight of light beacon : 3.4 kN

Copping conection: 1.0 kN

## PHC pile weight

Dimensions		Unit weight (kg/m)	Elevation of pile		Length (m)	Weight per pile (kN)
Dia (mm)	Thk (mm)		Top (m CD)	Bottom (m CD)		
700	110	509.7	5	-20	25	125.01

## Negative friction on steel pile

	Elevation		Layer Thk m	Soil Condition		f 2N of Cu	As m <sup>2</sup>	F per pile kN
	Top m CD	Bottom m CD		SPT-N	Cu kPa			
Layer 2	-4.82	-17.72	12.9	0.8	12.5	12.5	28.37	355.9
Layer 6	-17.72	-21.25	2.28	13.2	40.1	40.1	5.01	201.1
Total								<b>556.9</b>

Total downward load:  $F_{down} = 686.4$  kN

Bearing capacity

 $R_u = 300 \times N \times A_p$        $R_u = 300NA_p + 2\bar{N}A_s$ 

	Elevation		Layer Thk m	Ni	Ap m <sup>2</sup>	As m <sup>3</sup>	Rs per pile kN	Rtoe per pile kN	
	Top m CD	Bottom m CD							
Rubble layer	2.0	-4.82	6.8	10	0.38	15.00	300.0		
Layer 2	-4.82	-17.72	12.9	0.8	0.38	28.37	45.4		
Layer 6	-17.72	-21.25	2.28	13.2	0.38	5.01	132.4	1524.0	
Total, Ru:								<b>2001.7</b>	

 $R_a = R_u/2.5 = 800.7$  kN >  $F_{down}$  **OK**