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Geotechnical Aspect for Protection Bunds

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GEOTECHNICAL ASPECT FOR PROTECTION BUNDS

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ABSTRACT

The design team of Water & Environment division of Mott MacDonald, Ahmadabad was involved in design of Rubble mound protection bund for a Sea water intake and outfall project. The project was to provide make up sea water for 2x250 MW Lignite based Thermal Power Pant (TPP) at Bhavnagar for Bhavnagar Energy Company Limited.

The makeup water required for TPP was drawn through Sea water Intake (SWI) area constructed in the intertidal zone of Gulf of Khambhat. The SWI area consists of a Pumping Station for pumping sea water from storage pond.

The storage pond was surrounded by all around protection bund. Based on design requirement & available material near site, combination of Rubble mound protection bund and earthen embankment was considered for design having Armour stone layers as Break waters. The sizes of Armour stone was designed based on the wave modeling studies for return period of 25 years. The design life considered for bund was 25 years.

Based on the wave modeling studies, significant wave height was determined and that was considered as design wave height for determining sizes of Armour stones. The main body of bund comprises of the core, built by quarry run & two under layers of armour stones.

Apart from determining armour stone sizes, checking the overall bund formation for its stability was also an important criterion for design. For analyzing slope stability of Rubble mound and earthen bund, SLOPE/W software was used. Based on the analysis results from software, toe protection & scour protection at seaward face of the bund was proposed.

This paper provides Geotechnical aspect of the bund that includes Bearing capacity check, Settlement check, and Slope Stability analysis for Seismic condition, Non seismic condition & hydrodynamic wave forces. Liquefaction potential of the soil was also considered in the study.

INTRODUCTION

This paper describes design engineering involved for a breakwater structure, which is combination of rubble mound protection bund and earthen embankment, for a sea water intake system of 2x250 MW thermal power plant (TPP) at Bhavnagar, India. The bund structure secures the intake storage pond and sea water intake (SWI) pumping station. The SWI pumping station is designed to supply 5800 m3/hr make up water to the power plant from the storage pond having approximate area of 120m x 120m having 6.0m depth from sea bed level. The cooling water required for the power plant would be drawn through a seawater intake constructed in the intertidal zone of Gulf of Khambhat. Figure 1 shows storage pond along with 10m wide feeder channel which facilitates sea water into the storage pond. The storage pond is surrounded by protection bund on both sides. The bund facing the sea is southern protection bund and rear side is the northern protection bund. The front end of the bunds near feeder channel is having roundheads to facilitate easy flow near channel mouth. The design of protection bunds and armour layer as breakwater is based on significant waves that would prevail at the intake. The significant waves are arrived based on wave modeling studies to transform offshore waves from Sea to the location of structure.

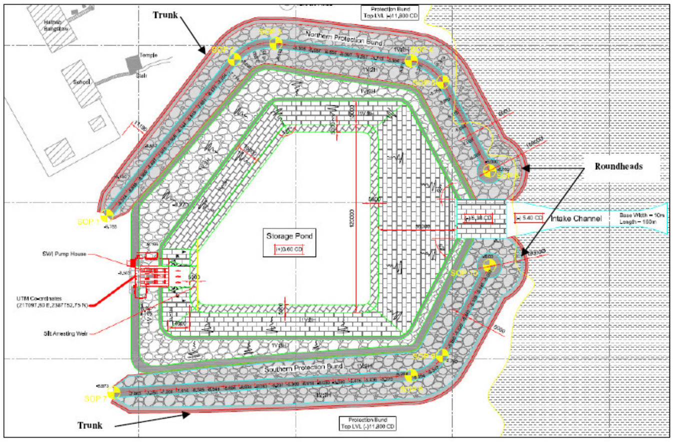

Figure.1. Plan showing Sea water intake system i.e. protection bund all around storage pond with feeder channel

WAVE MODELLING STUDIES:

To arrive at the significant waves at the SWI, wave modeling studies is undertaken. The extreme waves with 1 in 50 years return period that would prevail in the offshore Arabian Sea is transformed to the intake site using SWAN wave model (Simulating Wave Nearshore). The protection bund is designed against extreme waves that would prevail during the life time of the SWI. The protection bund prevents direct ingress of sediments into the pond with an opening to receive water from Gulf of Khambhat through a feeder channel.
SWAN model carries out propagation of offshore waves to inshore with wave distribution in time and space considering effects of refraction and shoaling, friction, wave breaking and wave-wave interactions. The model is more suited for transformation of wave energy spectra in relatively large coastal areas. Inparticular, areas where the features of the seabed, such as offshore banks, result in depth-induced wave breaking and wave-wave interactions.

<u>Application of SWAN model:</u>

SWAN wave model has been set-up to transform wave conditions from offshore Arabian Sea to the Seawater Intake located in Gulf of Khambhat as shown in Figure 2. The wave model domain covers entire Gulf of Khambhat and upto 200m contour offshore in the Arabian Sea. The model requires bathymetry (bottom), coastline (boundary), offshore waves, wind and other environmental data for carrying out the wave simulation. These simulations were carried out in three nested sub models of increasingly high resolution (see Figure 2). The outer grid covers entire gulf of Khambhat up to Cambay and up to 200m contour on the offshore in the Arabian sea with extents 268 km x 160 km in x and y directions respectively with grid resolution of 1 km. There are two inner grids nested in the model domain with higher resolution. The intermediate grid covers much of the features inside Gulf of Khambhat including Piram Island on the immediate northeast of intake and nearby reefs in the gulf. The inter grid has an extent of 30 km x 43 km and grid resolution of 200m. The inner grid covers the intake area & its immediate surroundings with resolution of 20m having an extent of 3.6 km x 3 km.

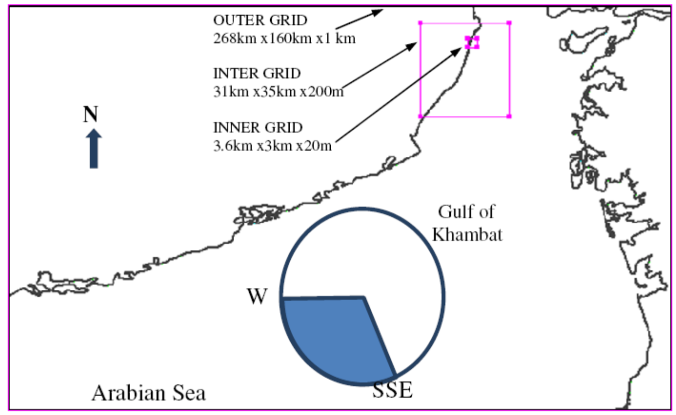

Figure.2. Extent of model area, grid and bathymetry considered for SWAN modeling

<u>Input data and model run.</u>

The model input data includes offshore extreme wave characters, wind speeds and directions and water levels for simulation of the wave transformation on the model. The location of seawater intake where the wave climate is to be derived is on the northern bank of the gulf and it is well inside. The gulf of Khambhat itself has orientation of Southwest towards Northeast. The location of the intake suggests waves travel from the western sectors to southern sectors i.e. west, south-west-west, southwest, south-south-west and south could reach the intake site as shown in Figure 2. Here the coastline of gulf of Khambhat would protect site from waves of North-western and South-eastern sectors. Piram Island would protect the site from Northeast sectors. As such the waves during southwest monsoon are the predominant for the gulf of Khambhat; the waves during the Northeast monsoon would have negligible effect for the intake site. Therefore, waves during Southwest monsoon viz. W (270º), SWW (247.5º), SW (225º), SSW (202.5º) and southern viz. (180º), SE (157.5º) direction sectors were considered for wave transformation studies. Considering the offshore extreme waves from the above direction sectors the wave transformation studies were carried out.

The offshore extreme sea waves and other climatic conditions used for different model runs based on the input data and other literature (see references) are presented in Table 1.

Table 1. Offshore sea waves and other climatic condition

Wave direction considered	Deep sea wave climate condition			
	Hs m	Tp sec	Wind m/s	Water level
157.5, 180, 202.5, 225, 247.5, 270	5.8	10.5	14	4.05

<u>Model run and output.</u>

The model run for a given offshore extreme wave characters, produces near shore wave characters and these wave characters are location specific across the model domain extracted at 20 m spacing around the Seawater Intake site. The wave characters for the protection bunds of the SWI were extracted at nine specific locations, six for southern protection bund viz. S1, S2, S3, S4, S5, S6 and six locations for the northern protection bund viz N1, N2, N3 as shown in Figure 3. These locations are at about 50m apart and 50m off the proposed protection bunds. These waves measured from the model (results) are located at about 50 m away from the proposed protection bunds so as to avoid the diffraction and reflection phenomena that would alter the approaching waves at very near to the bunds. The model was setup in such a way that the protection bunds position is considered 50m off the actual location in order that waves will have wall effects included into. These waves are considered as representative waves for the design of protection bunds.

The result of model run provided with the visualization that as the offshore waves approach the SWI site, the southern protection bund receives higher waves for much of its length and northern bund will receive only part of it exposed to the approaching waves.

The study considered the extreme waves from a range of incidence angles (direction) that would affect the site under consideration the most. The significant wave heights (Hs) and direction at each grid point in the model domain obtained as model results are plotted to colour code and presented in Figure 4. These plots represent the distribution of significant wave heights for a given offshore conditions considered for the model run.

The waves during northeast monsoon would prevail during the months of December to March in the form of wind waves from Northeast. But the winds are not as strong as winds during Southwest monsoon. Also, the fetch lengths available inside the gulf of Khambhat are limited and therefore the larger waves cannot be expected during Northeast monsoon. Hence, the northeast monsoon waves were not considered in the model study.

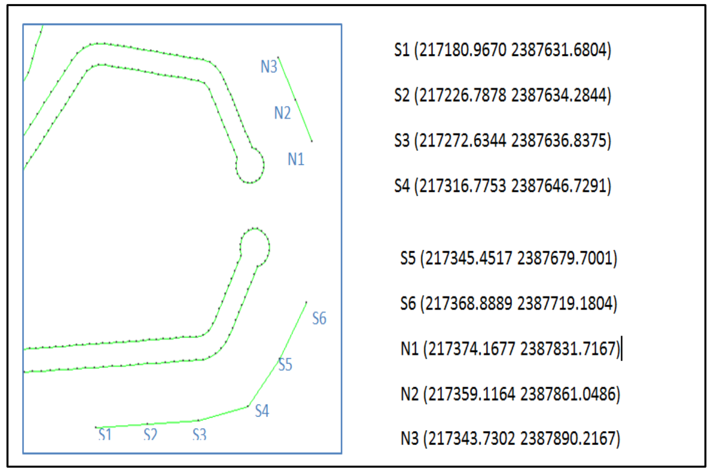

Figure 3: Near shore significant wave locations for SWI protection bund

The wave model results infer that a maximum significant wave height (Hs) of 1.2m with peak period of 6.7sec would prevail at the seawards side of the SWI protection bunds, for which bunds are required to be designed. The waves at SWI have direction from Southeast while the offshore incident waves are from south. Figure 4 shows distribution and approach of significant waves near the bund structure.

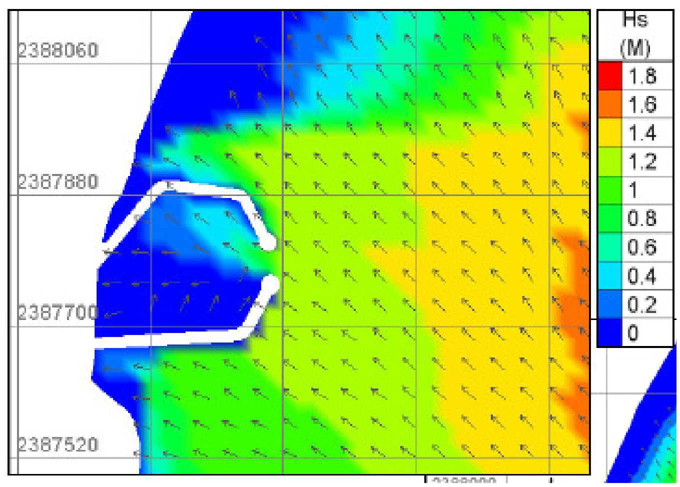

Figure 4. Distribution and direction of significant wave height

Thus, based on the available results from wave modeling studies, the significant height (Hs) was considered as design wave height for the design of bunds.

DESIGN OF BUND STRUCTURE:

To arrive at design of bund structure, wave modeling studies has provided with the dynamics of waves in offshore area approaching the site. The results of model run provided design wave height which becomes the basis of selection of breakwater type and framing the structure of bund. Following points were considered during the design stage for finalizing the bund structure.

- Based on the guidelines given in Owner's specification in tender document, the bund structure was required to be designed as Rubble mound protection bund using Armour stones.

- Availability of Armour rock from local quarries

- Cost is the most important factor for such structure. Armour stone bund structure incur low costing as compared to the other types of bund i.e. using concrete blocks, caissions or specially designed tetra pods, dolos, acropods, etc.

- This type of bund structure is not sensitive to differential settlement on account of their sloping faces and wide base and often foundation requirement is less for a comparable vertical structure placed directly on sea bed.

- Working on the design sections in line with discussion with approving authority and simultaneously keeping in view cost statistics interest of client.

The principle aim of rubble mound bund is to reduce wave action in the lee side of the structure. Wave action is reduced through a combination of reflection and dissipation of incoming wave energy. The principle aim of rubble mound bund is to reduce wave action in the lee side of the structure. Wave action is reduced through a combination of reflection and dissipation of incoming wave energy.

With reference to CIRIA C683 report, amongst the several types of the rubble mound bunds available, conventional rubble mound bunds has been selected for our design type. This type of structure consists of simple trapezoidal cross-section, armour layer covering the crest of the bund and part of the lee slope as well as the sea side of the bund.

The main body of rubble mound bund comprises of the core, usually built of wide-graded dredged or blasted material such as quarry run, one or more under layers, and the cover or armour layer. The crest is generally protected by the armour layer. The toe and scour protection at the seaward face of the bund, when built on sandy bed material, is needed to maintain stability of the slope, in case of erosion of the seabed.
Based on the distribution of significant wave height available from the wave modeling study model run, the bund structure was designed in four sections. As shown in figure 1, the bund structure is termed as Northern bund and Southern bund. Thus, based on the distribution of wave heights, the southern bund is designed as rubble mound protection bund, while the northern bund is designed as combination of rubble mound bund for the sea facing front portion and rear 150m portion as earthen bund.

The rubble mound protection bund design has been carried out in reference to ***Coastal Engineering Manual (CEM), 2006***, ***CIRIA C683*** and ***Shore Protection Manual (SPM) 1984***.

The design of Earthen embankment is done in accordance with ***IS 12094:2000, IS 12169:1987, IS 8408 and IS 7894 - 1975.***

Based on the guideline given by the approving authority, It was suggested to consider a factor of safety to ensure the damage is confined to 10% only. An appropriate factor of 1.5 as per ***EAU 1996*** was suggested to apply on the significant wave height determined from wave modeling studies to arrive at design wave height.

The critical design sections and their design data information is as below:

Section I: Northern bund front portion of 100m stretch and southern bund front portion of 180 m stretch are designed as rubble mound bund structure considering significant design wave height of 1.8m.

Section II: The rear 100m trunk portion of southern bund is designed as rubble mound bund structure considering significant design wave height of 1.20m.

Section III: Intermediate 100m trunk portion of northern protection bund is designed as rubble mound protection bund considering design wave height of 0.9m.

Section IV: Rear trunk portion of 150m stretch of northern protection bund is designed as earthen bund considering significant design wave height of 0.75m.

Section I with highest significant design wave height is designed first. Based on the design, the cross section is finalized. This cross section becomes the base for finalization of all other sections. The variation and transition from one design section to another is taken care during the detailing stage while preparation of working drawings.

Table 2 provides information on basic cross section parameters considered for design.

Figure 5 shows typical cross section of rubble mound bund along with bedding beneath and toe protection on seaward side. The details of each layer for each section i.e. Section I, Section II and Section III are provided in Table 3, 4 and 5 respectively.

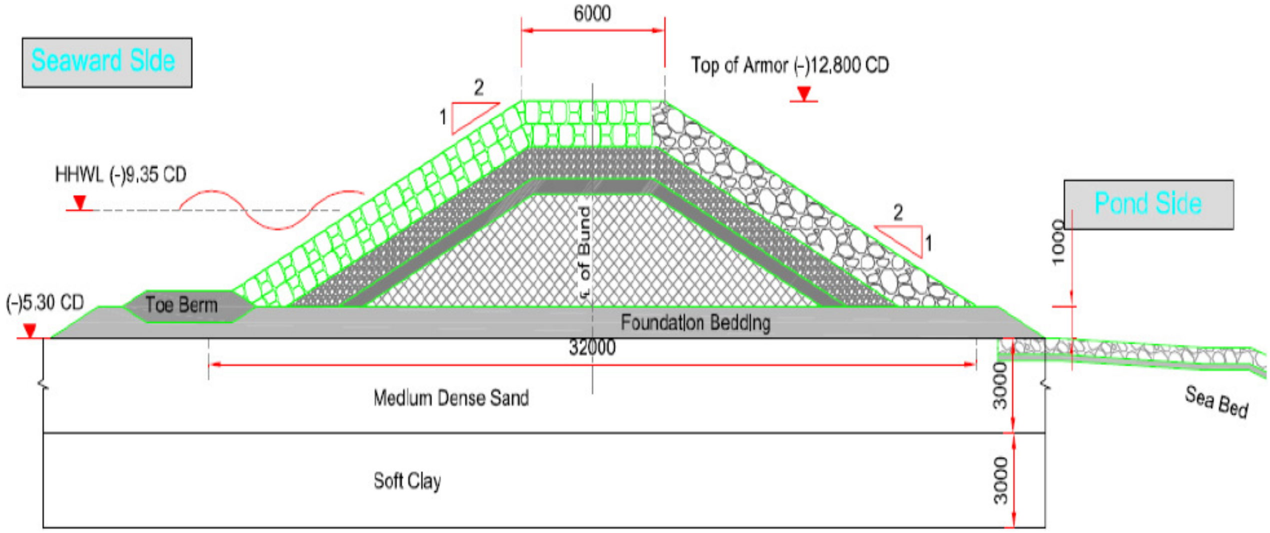

Figure 5. Typical cross section of Rubble mound bund with bedding and toe protection

Table 2. Dimensional parameters

Sr. No.	Parameters / Description	Value	Units
1	Total protection bund height	6.00	m
2	Structural slope (tan α)	1:2	-
3	Crest width (B)	6.00	m
4	Free board (Rc)	0.59	m
5	Total bottom width (including toe berm)	38	m

Table 3. Details of each layer of section I

Sr. No.	Parameters / Description	Value	Units
1	Armour layer	-	-
	W50	1.00	T
	No. of layers (n)	2	Nos.
	Layer thickness, average	1.45	m
	Volume per unit length (V/L)	44.00	m^3/m
2	First under layer (Wul1)	0.08-0.15	t
	Volume per unit length (V/L)	18.92	m^3/m
4	Second under layer (Wul2)	4 – 6	kg
	Volume per unit length (V/L)	5.90	m^3/m
5	Core (Wcore)	0.15-.35	kg
	Volume per unit length (V/L)	49.60	m^3/m
6	Toe (Wtoe)	0.15 – 0.25	t
	Volume per unit length (V/L)	16.10	m^3/m
7	Bedding (Wbed)	6.0 – 7.5	kg
8	Design significant wave height	1.8	m

Table 4. Details of each layer of section II

Sr. No.	Parameters / Description	Value	Units
1	Armour layer	-	-
	W50	0.32	T
	No. of layers (n)	2	Nos.
	Layer thickness, average	1.00	m
	Volume per unit length (V/L)	28.00	m^3/m
2	First under layer (Wul1)	0.03	t
	Volume per unit length (V/L)	13.50	m^3/m
4	Second under layer (Wul2)	1.81	kg
	Volume per unit length (V/L)	4.91	m^3/m
5	Core (Wcore)	0.1	kg
	Volume per unit length (V/L)	55.10	m^3/m
6	Toe (Wtoe)	0.09	t
	Volume per unit length (V/L)	11.90	m^3/m
7	Bedding (Wbed)	2.27	kg
8	Design significant wave height	1.2	m

Table 5. Details of each layer of section III

Sr. No.	Parameters / Description	Value	Units
1	Armour layer	-	-
	W50	0.14	T
	No. of layers (n)	2	Nos.
	Layer thickness, average	1.00	m
	Volume per unit length (V/L)	27.00	m^3/m

2	First under layer (Wul1)	0.02	t
	Volume per unit length (V/L)	9.90	m^3/m
4	Second under layer (Wul2)	0.91	kg
	Volume per unit length (V/L)	3.42	m^3/m
5	Core (Wcore)	0.04	kg
	Volume per unit length (V/L)	48.00	m^3/m
6	Toe (Wtoe)	0.02	t
	Volume per unit length (V/L)	9.80	m^3/m
7	Bedding (Wbed)	0.45	kg
8	Design significant wave height	0.90	m

Section IV of bund is designed as earthen embankment and the same is covered under next section.

DESIGN BASIS AND DESIGN OF EARTHEN EMBANKMENT:

Based on wave distribution shown in Figure 4, the northern protection bund from chainage 0 to 150m is exposed to wave heights less than 0.1m. This section of bund is not facing the waves. Earthen embankment is considered for this portion of bund which forms section IV of bund. However, considering the critical design condition, design wave height of 0.75m is considered for design of earthen bund. The design of Earthen embankment is done in accordance with Indian codal provisions framed in IS 12094:2000, IS 12169:1987, IS 8408 and IS 7894 - 1975. Table 6.0 shows extent and design parameters of section IV.

Considering the constructability aspect, availability of material in vicinity of site, a homogeneous type earthen embankment was finalized. According to IS 12169 – 1987 (Table 1 & Table 2), the properties of the embankment material should follow the properties given in the table 6.

Table 6: Properties required for homogeneous type embankment material

Sr. No.	Engineering classification of the soil IS 1498 - 1970	Value	Units
1	Earthen embankment		
	MDD	18-19	kg/m^3
	OMC	14.5-15.5	%
	Cohesion	1100-1700	kg/m^3
	Tan φ	0.51 – 0.65	-

With respect to review of soil investigation report taken at bund site and having in knowledge that huge excavation to be will carried out for forming the storage pond area, the material selection for earthen bund was considered in line to get benefit of the excavated material. For the design of the earthen embankment the critical properties of the **SC type** of soil is considered for formation of cross section and are as given in table 7.

Table 7: Properties considered for homogeneous type embankment material

Sr. No.	Engineering classification of the soil IS 1498 - 1970	Value	Units
1	Earthen embankment (SC type)		
	MDD	17	kg/m^3
	OMC	15	%
	Cohesion	1100	kg/m^3
	Tan φ	0.58	-

Further, in line with the cross section designed for rubble mound protection bund based on CIRIA guidelines, the earthen bund cross section especially, structural slope for the desired height was considered as per guideline framed in IS 12169 – 1987 (Table 1). The provided free board of 2.35m is also checked with codal requirement i.e. free board requirement due to wave action and free board requirement due to 2% settlement allowance. Sea side slope protection with dumped rip rap is considered as per IS 8237 – 1985. The thickness of rip rap is considered as 600mm as per minimum consideration framed in codal provision and minimum average rock size (D50) taken is 300mm. The full thickness of dumped rip rap is considered to be dumped in two layers. the riprap rock weight for this average size ranges from 40 kg to 55 kg.

Based on Cl. 5.0 of IS 8237 – 1985, the filter layers are provided for the seaward slope of embankment. The two layers of filter (Coarse and fine) are provided to prevent the waves from eroding and washing out the underlying embankment material. The thickness of filter layer is provided as 200mm for finer and coarser filter layer. Gradation requirement for the coarse filter material with respect to riprap material should confirm to the criteria that D85 size of the coarse filter material shall not be less than l/10 of D15 size of the riprap material. The gradation requirements for the fine filter with respect to embankment material should confirm to the criteria that D15 size of the fine filter material shall not exceed 5 times the D85 size of the retained embankment material.

As per requirement of Cl. 8.0 of IS 8237 – 1985, the downstream slope protection is suggested as providing turfing. Considering the importance of structure, the leeward side of earthen embankment is protected by providing hand placed riprap without filter layers. Based on Cl. 7.2.1 of IS 8237 – 1985, the minimum average rock size (D50) for maximum wave height up to 1.5 m shall be 300mm. Thus, the riprap rock weight for this average size ranges from 40 kg to 55 kg.

Scour protection and Bedding:

Scour protection is provided for prevention of the undermining of the seaward side bund structure, it is provided to have a sufficient depth of the protection layer, beneath the structure, before the undermining starts scouring the main structure itself, which may lead to the failure of the structure in future days of severe wave attacks. Thus, minimum scour depth of 0.5 times design wave height has to be provided. In this case study, the highest significant design wave height hitting the structure is 1.8m. Thus, bedding height requirement comes out to be 0.90m. For uniformity 1.00m depth of bedding layer is considered.

The general practice of providing the bedding layer is excavation / removing the superficial bed layer and placing the bedding layer. But in this case study, it is decided to put bedding layer above the existing sea bed after profiling. Thus, the main advantage of this goes to the execution team. They do not have to excavate / dredge in the sea during tidal conditions and the bund height increases by 1m. This increase will help when the initial settlement of structure takes place after overall building up. Even after initial settlement, which may be maximum of the order of 500mm, will keep the bund size more than the required as the bedding of 1m is kept above sea bed. Thus, keeping bedding layer above the profiled sea bed is proving advantageous in terms of safety along with saving of huge cost of dredging.

For the purpose of preventing the scouring effect the bedding layer has been extended 1.00m horizontally beyond the toe cover on the sea ward side and leeward side of all the design sections of bund structure.

Toe design provides protection against scouring and undermining of a structure and support against sliding to the structure armour/face. The toe therefore needs to be designed to prevent the occurrence of these two possible failure modes.

Armour stones are often considered to be the preferable choice for the stones/rocks in toe protection, as because of its flexibility and inter-locking, while in this case it is separately designed to have optimization of the design as per the requirement from the water depth at the seaward side of the bund. The toe is designed to be placed at the anticipated scour depth for the bund as per the total depth of water at the toe on seaward side, in such a fashion that the bund shall be protected from scouring through an extra layer of toe continuously throughout the total length of bund on seaward side.

GEOTECHNICAL ASPECT OF PROTECTION BUND:

Geotechnical design of the embankment structures is required to prevent failures or excessive deformations of the structure or its foundation. Protection bund is combination of rubble mound bund and earthen embankment. The geotechnical risks may be summarized as follows for both types of bunds i.e. Rubble mound and earthen bund.

Geotechnical Risks for Protection Bund:

- Bearing capacity failure of the ground
- Settlement
- Stability of the slope for
    - Normal Loading
    - Seismic and
    - Hydrodynamic Wave force
- Liquefaction

Other than above checks the earthen embankment also checked for basic design requirement as below:

- Stability Analysis
- Seepage Analysis

Bearing Capacity

The plastic failure of the ground under a rock structure is a mode of failure that may occur even when the internal and global stability of the structure is verified. The verification of the ground bearing capacity must therefore be performed for each structure or part of structure: it should be verified that the calculated bearing capacity is larger than the maximum load on the foundation. The ground bearing capacity under Rubble mound Bund is calculated by using analytical methods based on laboratory test results. For geotechnical design critical subsoil profile and soil parameters are taken from the critical borehole data which is falling in the bund area. We have considered the sand as the medium dense sand and clay as the soft clay considering the critical conditions for design.

The sub base layers, underneath the bund structure, are medium sand up to 3m and clay layer of 3m below sand layer as shown in figure 5. Bearing capacity is checked at both the layers for the type of failure is considered as general shear failure.

The calculated factor of safety for both types of soil layers is greater than 4 and hence the bund structure is safe for bearing capacity for the sub base on which it is resting.

Settlement Check

For Settlement analysis, net loading intensity qn is obtained by using the physical characteristics of the foundation and the relevant compressibility characteristics of the Underlying soil. The value so obtained ensures that the foundation shall not settle more than that which is permissible. For Settlement analysis, total settlement is considered as summation of three different component, namely Immediate or elastic settlement, consolidation settlement and secondary settlement. Net loading intensity "qn" is been obtained using the physical characteristics of the foundation and the relevant compressibility characteristics of the Underlying soil. The value so obtained, ensures that the foundation shall not settle more than the permissible limit.

Total Settlement S = Si+Sc+Ss

where Si= Immediate settlement
Sc = consolidation Settlement
Ss = secondary settlement

<u>For Sand Layer:</u>

Settlements of structures on cohesion less soils take place immediately as the foundation loading is imposed on them. Schmertmann's method is used, where in triangular relative strain diagram to model the strain distribution with respect to 0B, 0.5B and 2B. When Es is not constant, schmertmann proposed to plot the strain profile and obtain influence factors Iz at the centre of each change in Es over a depth increment Δz to obtain settlement.

$S_i = C_1C_2 (q' – q) \Sigma (I_z / E_s)\Delta z = 0.0026$ m

<u>For clay layer:</u>

5)

Primary Consolidation settlement (S_c) occurs in saturated, clayey soils when these are subjected to increased loads caused by the foundation pressure, while the secondary consolidation settlement (Ss) occurs after completion of primary settlement. Here the settlement is calculated for locations A, B, C and D at point 1 and 2 as shown in Figure 6. The value of primary consolidation settlement (S_c) is

$$\text{Sc} = \quad S_c = \frac{C_c}{1+e_0} \times H_0 \times \log_{10}\left(\frac{\sigma_0 + \Delta\bar{\sigma}}{\sigma_0}\right) \quad = 0.07 \text{ m}$$

Secondary Consolidation settlement (Ss):

$S_s = C\alpha\ H \log 10(t_2\text{-}t_1) = 0.016$ m

Where, time taken for secondary compression (t_2) = 25 yrs
time taken for primary consolidation to complete (t_1) =2 yrs is considered.

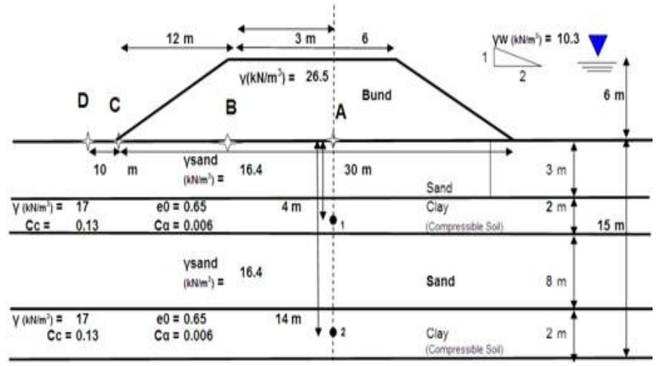

Figure 6: Figure showing location where primary consolidation settlement is calculated

Total Settlement:

Total Settlement for Clay = Sc + Ss = 0.07+0.016 = 0.09 m

Hence overall settlement in clay and sand
= 0.0026 + 0.09 = 0.093 m = <u>93 mm</u>

The allowable settlement for the bund structure shall not be greater than 300mm. This allowable settlement criteria is based on guideline given by "Port work design manual – Part 4, The Government of Hong Kong Special Administrative Region".

SLOPE STABILITY ANALYSIS

One of the important geotechnical checks for any type of embankment is the slope stability analysis. This analysis ensures the stability of slope for the embankment made of selected material for various intended loading and construction stages. The conventional approach for doing such analysis is by graphical method. This method involves numbers of iteration to arrive at the critical slip surface required for ensuring the stability of slopes.

Considering the importance of structure and variation of material in rubble mound bund and earthen embankment, slope stability analysis is evaluated using limit equilibrium methods as implemented in the SLOPE/W software, a product of GEO-SLOPE International Ltd. Calgary, Alberta, Canada (www.geo-slope.com).

<u>Stability analysis of Bund</u>

<u>Software and Model:</u>

SLOPE/W is a special-purpose computer code designed to analyze the stability of slopes using two-dimensional, limit equilibrium methods. Slope/W model generated for bund is as shown in figure 4.4

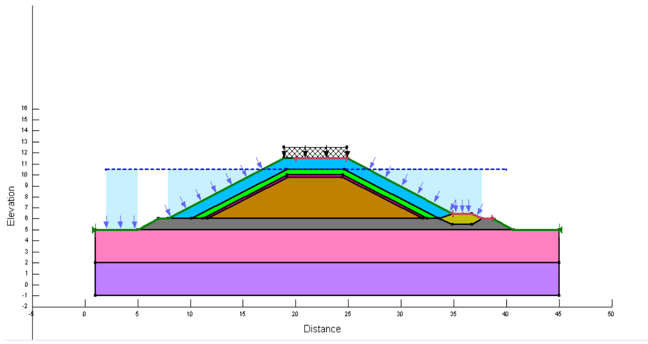

Figure 7: Software model for southern protection bund

Model for the bund is prepared based on cross sections of bund and sub soil profile. The slope of 2H: 1V is provided on seaward and leeward side. The effect of wave on sea side is considered using pore pressure line. Surcharge loading is considered over the top of bund. Bishop Method is used for the analysis.

Considering the top of crest may be utilized for vehicular movement in future or during execution, surcharge load is considered as 30 kN/m^3 (equivalent to 70 R loading) is applied as uniformly distributed load on top of bund as per specification given in IRC 70R.

Stability analysis is carried out for following cases:

a) Non-Seismic Condition
b) Seismic Condition
c) Hydrodynamic wave force

<u>Non-Seismic Condition:</u>

The model is generated for non-seismic condition. HHWL is taken as pore water pressure on the bund and surcharge load is considered as 30 kN/m3 (equivalent to 70 R loading) is applied as uniformly distributed load on top of bund as per specification given in IRC 70R.

<u>Seismic Condition:</u>

It is generally agreed, based upon analytical study and instrumental records, that earthquake magnitude, distance from the hypocenter and local subsurface conditions are the three major factors that affect the seismic intensity at the site. The larger the magnitude or shorter the distance from the earthquake focus, the stronger is the seismic intensity at a given site. In addition, the level of shaking intensity in rock is generally different from that in a soil deposit at ground surface or at any depth below the ground surface. Other factors being equal, local subsurface conditions alone can both amplify and attenuate earthquake forces. During small earthquakes and microtremors, the ground surface accelerations on soil deposits, especially on soft compressible clay layers and alluvial deposits, are usually higher than those occurring on bedrock. However, as earthquake magnitudes become greater, the horizontal accelerations on soil sites may be equal to or lower than those on rock sites.

The earthquake impact on the bunds is been found out using **Slope/w** software.

The horizontal and vertical seismic coefficients are calculated as per IS 1893 – 1984 and the values of these coefficients are provided in the software.

<u>Hydrodynamic wave force:</u>

Protection Bund is dimensioned such that no significant wave impact loads are to be expected. The force generated in the protection Bund due to Wave impact is calculated & applied to the model in Geo Slope software to check the stability of the Protection Bund.

<u>Recommended Factor of safety:</u>

The slope stability for bund is analyzed and checked with minimum factor of safety 1.3 for non seismic condition, 1.1 for seismic condition and 1.1 for hydrodynamic wave force. The protection bund is safe against failure if the min. factor of safety is achieved.

<u>Results of Slope Stability Analysis:</u>

Using the strength parameters (c and φ), in conjunction with the loading, the bund configurations were analyzed at most critical cross-section. Geo-Slope's Slope/W computer program was used for the analyses including pore water pressure. For the Bishop's simplified method analyses, circular failure surfaces with optimization were conducted. The stability analyses focused on the potential for failure along the seaward and leeward side of bund. A SLOPE/W failure surface from these analyses for all three cases is done. Figures 8 shows slip circle formation and factor of safety for seismic condition on seaward side.

Similar approach is taken for all the defined cased on seaward and leeward side of slopes and factor of safety is determined. From the results of SLOPE/W analysis for various conditions, the factor of safety determined for rubble bund structure and earthen embankment structure is satisfactory and above the permissible value.

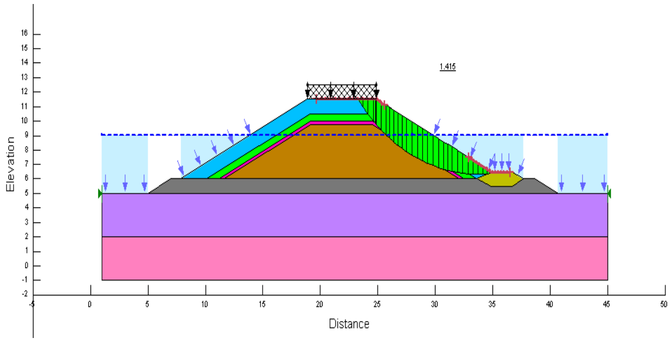

Figure 8 Slip circle for seismic condition on seaward side

Liquefaction Potential of the soil

Liquefaction refers to the decrease of shear strength and/or stiffness caused by the increase in pore water pressures in saturated non-cohesive materials during earthquake ground motion, such as to give rise to significant permanent deformations or even to a condition of near-zero effective stress in the soil (EN 1998-5:2004). Non-cohesive soils include layers or thick lenses of saturated loose sand, with or without silt/clay fines. A state-of-the-art paper is Youd *et al* (2001).

The evaluation of the liquefaction susceptibility must be performed for the ground surface elevation and the water table elevation prevailing during the lifetime of the structure. The reference method for this purpose consists of using the results of in situ Standard Penetration Tests (SPT) or of cone penetration tests (CPT); for information about SPT and CPT penetration tests see Section 4.4. Based on work by Seed and Idriss (1971), Seed *et al* (1983) and Seed (1983), the criterion for liquefaction is expressed in EN 1998-5:2004 as the set of curves of Figure 5.129, which define limiting values of the ratio of the earthquake-induced cyclic shear stress, τe (kPa), to the effective vertical stress, $\sigma' v0$ (kPa). These curves depend on the normalised SPT blow count value, $N1(60)$, defined by Equation given below

$$N_1(60) = N_{SPT}\sqrt{\frac{100}{\sigma'_{v0}}}\cdot\left(\frac{E_R}{60}\right)$$

Where $NSPT$ is the measured value of the SPT blow count, expressed in blows per 300 mm (-); 100 is the overburden pressure (kPa), $\sigma' v0$ is the initial effective vertical stress at the depth and time of the SPT measurement (kPa); and ER is the energy ratio, specific for the testing equipment (%).

Based on the above method the liquefaction potential is calculated and it is noted that the soil below the protection bund is not susceptible to liquefaction.

Seepage analysis of earthen embankment:

According to Casagrande, the phreatic line or seepage line for homogeneous fill with no filter is drawn as shown in Figure 9. The location of the phreatic line is necessary in order to analyze the stability of the embankment. Its position is not influenced by the permeability of the material composing the embankment as long as the material is homogeneous.

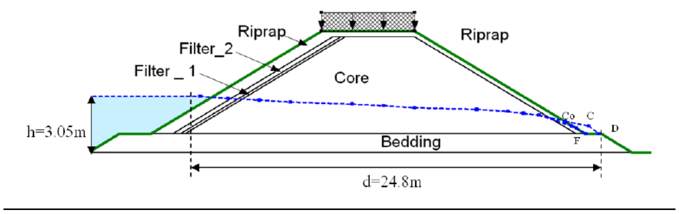

Figure 9 Phreatic line for homogeneous earthen bund with no filter

The point CF is known as the discharge face and the value ‘a’ (see Figure 3.11) is used to construct the corrected phreatic line. To determine the value of ‘a’ Schaffernak and Van Iterson method is used for α<30º. In this case α= 26.56 º.

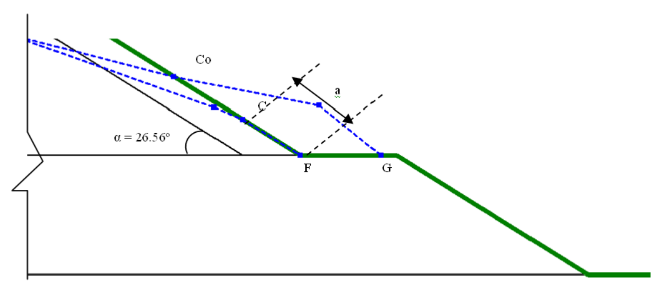

Figure 10 Enlarged view showing Phreatic correction line

Thus, based on the seepage analysis, the phreatic line passes through 0.67m above the point F of bund. To check the stability for steady seepage condition the phreatic line as shown in the above figures is constructed in the Slope/W model and accordingly the stability analysis has been done.

Stability check for Homogenous earthen embankment:

As per Clause – 5.1.2.3 of IS 12169:1987, slope stability check is not necessary for earthen embankment, where the height is 5m to 10 m. However, to ensure stability of embankment for different loading conditions, stability analysis was done in SLOPE/W software. The stability of seaward slope is computed for the following conditions, with and without earthquake

- Sudden drawdown condition
- Just after construction

The Stability of the leeward slope is computed for the following conditions, with and without earthquake

- Steady seepage condition
- Just after construction condition

CONCLUSION:

- The bearing capacity is more than the load coming from the protection bund.
- The stability analysis of rubble mound protection bund using the software code GEO-SLOPE is carried out for non-seismic, seisimc, hydrodynamic wave forces and Sudden drawdown condition. The FOS achieved for each case is more than the permissible values of FOS. Thus, the rubble mound protection bund structure is safe and stable.
- The earthen embankment was also checked for its slope stability for just after constrcution, Sudden drawdown and steady seepage condition with and without seimic condtion. The factor of safety was achieved against the permissible FOS suggest that the bund structure is stable and safe for various loading conditions.
- Geomembrane is provided on bedding layer and seaward side slope in homogeneous earthen bund to ensure that the bund slopes does not get affected due to seepage.
- The settlement of bund and liquefaction criteria are also under permissible condition.

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