Commemorate the Legacy of Ralph B. Peck
April 29 - May 4, 2013 CHICAGO
Seventh International Conference on Case Histories in Geotechnical Engineering
and Symposium in Honor of Clyde Baker

ANALYIS AND DESIGN OF BREAKWATER FOR SEA WATER INTAKE FACILITIY ON SOUTH EAST COAST OF INDIA

Jaykumar Shukla	**Laxman Rajwani**	**Dhananjay Shah**
L&T- Sargent & Lundy	Royal Haskoning India Pvt. Ltd.	M.S. University of Baorda
Baroda, India- 390019	Mumbai, India-	Baroda, India-390009
Jaykumar.Shukla@Lntsnl.com	L.rajwani@haskoningindia.co.in	dr_dlshah@yahoo.com

ABSTRACT

The proposed power project site on the South East coast of India required the construction of breakwaters to prevent detrimental effects owing littoral drift along the coast and also to protect the intake channel lining against sea attack. This paper describes the geotechnical design and analysis for breakwater facility proposed for the site. The preliminary design of breakwater was carried out using raw geotechnical data made available in form of the few boreholes investigation and SPT N-values. The general soil profiles suggest that the stratum consist of 37 metres of soft clay below sea bed resting on medium dense to dense sand formations. Ground improvement was inevitable for the site and attempts were made to review the techno commercial feasibility of ground improvements in the staged construction framework for vibroreplacement and band drain installation. Slope stability analysis and time dependent settlement analysis were implemented for suggested staged construction sequence to ensure the desired stability at each construction stage. The partial treatment scenario was considered owing to practical limitation for carrying out the ground improvement along the full clay depth. The study describes the possible strength gain approach as geotechnical solution for such difficult sites to achieve techno commercial tradeoff. The stability analyses are addressed in the present study for each construction stage along with settlement analysis. The good quality geotechnical investigation carried out for the site has revealed that sound geotechnical judgmental engineering from the insufficient preliminary data can lead to converge better in this competitive environment.
.

INTRODUCTION

Recognizing that electricity demand in India was increasing at a rate faster than overall energy supply, in the year 2004, the Government of India formulated a major policy to enhance the production of electricity so that overall development is not hampered on account of a power shortage. To implement the policy, a scheme was initiated for setting up large 4000 MW units termed as Ultra Mega Power Projects (UMPP). One of these UMPP is planned to set, along the coastal areas of villages Ayyavaroppakhandriga and Shambhunithopu, north of the Krishnapatnam Greenfield harbor in Andhra Pradesh, India. Fig. 1 describes the geographical location of the proposed project.

The developer, having gone through the various aspects for the project, identified the need for make up water to meet the cooling water requirement of turbines. For make up water requirements building up the sustabable and robust sea water intake facilities was considered to be most important element and lifeline for keeping power plant operation throughout its life. To decide the optimum scheme, technical feasibility of various offshore and onshore intake structures were explored. Keeping in mind important aspects such as maintenance dredging, shore protection, hot water recirculation, and design of protective structures, it was decided to implement offshore intake structure with breakwater protection option for cooling water intake system.

In this alternative, an open channel of the required dimensions was proposed to be dredged in order to bring make-up water to an inshore pump house. Since such a channel was proposed to be connected to the required depth of water to ensure that the supply remained uninterrupted, even at the lowest astronomical tide, the channel would be exposed to wave action which could transmit to the sump, thereby affecting the pump performance. Some protection would therefore have to be provided, which would also prevent entry of sediment to the exposed part of the intake channel. Schematic layout of the intake channel is reproduced and shown in Fig. 2.

In general, the littoral drift along the east coast of India is restricted to about 7 m contour and approximately 90 % of the littoral transport of sand being restricted to within the 5 m contour. Breakwaters are proposed to be built on either side of the channel to prevent the drift and thereby damaging the mouth of intake channel. Scour Protection in addition to breakwater is shown on either side to arrest accreation and erosion effects on the outer side of north and south breakwater as shown in Fig. 2. The sea bed near shore has a mild slope as could be seen from the extracts from Navigational Chart no 3031 (Fig. 1). As can be seen from the chart (Fig. 1), the 5 m contour is about 500 and the 10 m contour about 5 km from the shore line.The shore connected breakwaters were proposed to be provided up to 5 m water depth. The length of each of the breakwaters was approximately 500 m. By providing proper shape to the breakwater alignment it would be possible to bypass some of the littoral sand naturally so that, the erosion of the down drift shore line is very limited. Accordingly, the shape of the breakwaters is designed as an inverted cup with an opening of about 50 m on the seaward side which will help in self bypassing of coastal drift material.

.

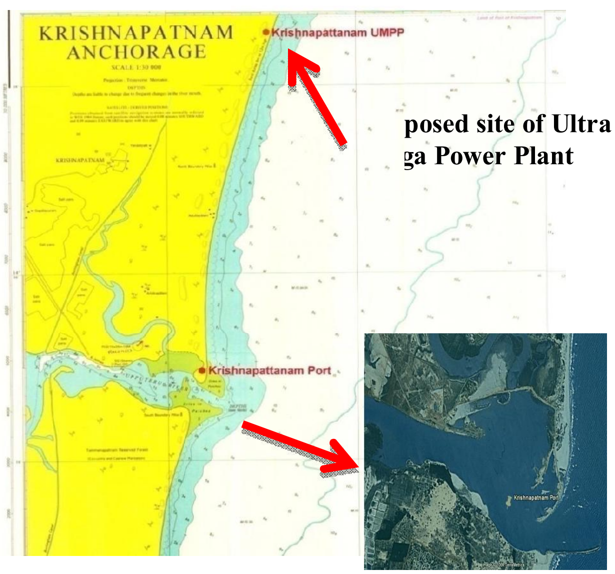

Fig.1 Krishnapatnam Port location (source – Google Earth)

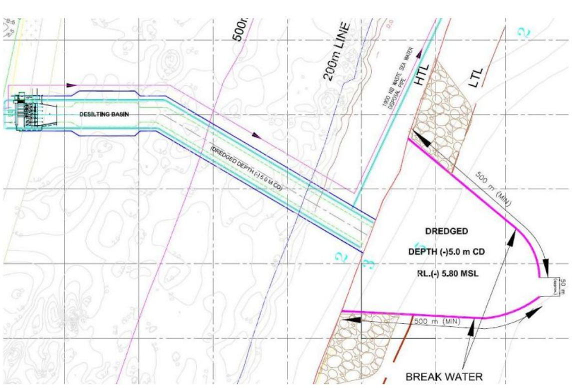

Fig. 2 General Layout and Alignment of the Intake Channel & Breakwaters

BREAKWATER SIZING

For the design of breakwaters various physical factors which need to be considered are wave climate, tidal levels, availability of stones, bed material, and bed slopes etc. The breakwaters are subjected to dynamic forces of waves and need to be safe against the maximum wave action expected at the site. At the same time the design of the breakwater should be optimized so as not to become too costly without of course sacrificing safety. In general the life of the breakwater structure expected is even more than 100 years. In order to make an optimal design, the choice of the design wave height is very important.

As described in the Section on Site Conditions, the wave height during SW monsoon could be 2.5 to 3 m from SE and 2.0 m from NW direction during pre and post-monsoon periods for the site. Under the cyclonic condition a wave height of 9 m with a return period of 1 in 100 years could occur in a water depth of 15 m from SW or W direction. Since overtopping of the breakwater could be tolerated for a few days in the year when cyclonic conditions are experienced, the design wave for overtopping was considered as 2.5 m.

With regard to the stability of the armour units, the design wave height could be taken as that wave height which can be sustained in the given depth of water. A wave breaks at the structure when the depth of water is 1.3 times the wave height. Putting inversely, the maximum wave height that can be sustained in a depth of water ‘d’ is equal to 0.78d.

The Mean High Water Springs at site is + 1.20 m CD (Chart Datum) and Highest High Water is + 1.4 m C.D. Since the breakwater round head is located at the 5 m CD contour, the depth of water at high water would be 6.4 m, which would sustain a wave height only up to 5 m, and the nose (round head) would have to be designed for this wave height regardless of the high waves generated by cyclones. Similarly the trunk of the breakwater, at say the 3 m CD contour, would have to be designed for a wave height of 0.78 x 4.4 = 3.4 m. Accordingly the design conditions considered are given below.

The type of structure to be considered for the breakwater depends upon the construction material available economically near the site, effect of structural damage and maintenance requirements. Rubble mound structure is generally favored because damage to the rubble mound is gradual and the force due to wave action has to act for a longer period of time to cause any major damage. Depending upon the quarries available and the biggest size of stone that could be quarried and the quarry yield, a rubble mound structure could be designed economically. In case large stones are not available in sufficient quantities, artificial concrete blocks in the armour could be used.

The parameters generally considered in the design of the armour unit are unit weight (of rock, in the instant case), wave height at the structure, specific gravity of armour unit, angle of seaward breakwater slope and a factor which is indicative of the interlocking property of the armour unit. The details of the breakwater section are decided on various considerations, such as method of construction, whether crest level be such that no overtopping of waves would take place, crest width requirements, necessity for lee side reclamation or otherwise, bedding requirements and the largest size stones which could be available from the quarries. It is assessed that quarries considered for proposed breakwater would produce stones with a maximum weight of 10 to 12 tonnes.

In the rubble mound section, use of stones in the armour layer is always economical. However the required size (weight) may be neither easily quarriable nor possible to be transported to the breakwater site. In such a case artificial concrete blocks need to be cast near the site and used in the armour layer. There are many types of concrete blocks of different shapes that have been developed by researchers. Tetrapods have been very extensively used in breakwaters all over the world. Some new blocks of recent origin like Accropode and Core-loc are being used due to the specific advantage that these blocks can be used in a single layer instead of in double layer as is the case with most of the other blocks. Thus the cost of the breakwater could be reduced; however, these types of blocks need to be placed in a specific manner to achieve the required interlocking property which otherwise would not be possible if laid. One more advantage of these new blocks is that their 'K_D' value is higher than that of tetrapods and due to their geometrical shape they are more effective in dissipating wave energy and as such the weight or the concrete volume required for each block would also be less.

Based on various considerations mentioned in the earlier paragraphs, breakwater sections for various sections for the south, west and north breakwaters have been worked out. It could be noted that the artificial concrete armors in form of tetrapods finally proposed to be used in the armour layer for the entire breakwater except the section nearest to lands. The details of the breakwater section at different contours are given in Table 1. A typical cross section of Breakwater is shown in Fig. 3.

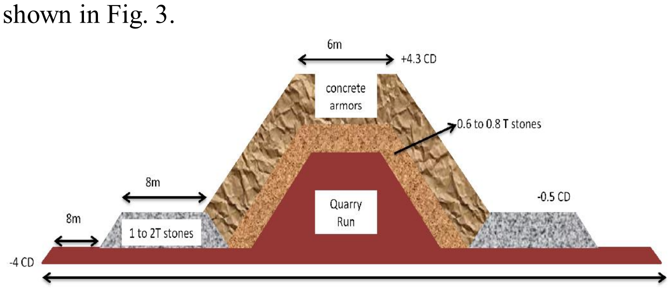

Fig.3 Typical cross section of proposed rubble mound breakwater close to round head section.

PREIMINARY GEOTECHNICAL INVESTIGATIONS

Preliminary geotechnical investigations were carried out in form of drilling boreholes for offshore and onshore locations. Two marine boreholes (MBH 9 & MBH 10) have been drilled in the vicinity of the breakwaters as shown in Fig. 4. The comparison of few boreholes drilled close to offshore along with MBH 9 and MBH 10, are given in the Fig. 5. The foundation soil (MBH 9) consists of very soft clay up to about -20.0CD (SPT = 2) followed by a layer of soft clay (SPT 3-4) up to about -33.0CD. Further down, SPT value of the clay improves from 8 to 12 at about - 43.0 m CD. The clay layer is underlain by sandy layer up to the drilled depth of - 63.5 CD, with SPT varying from 28 to 42 between RL - 44 m CD and - 53 m CD followed by very dense sand & gravel mixture (SPT > 100) up to the drilled depth. In absence of extensive geotechnical explorations, the analysis was mainly focused on the SPT values and Plasticity Index (Fig. 6) as the identified upper layers is consisting of soft clay. Without clear shear strength data (lab test/vane shear test) on the undisturbed samples of the clay, shear strength (cohesion) of the clay layer was approximated from the SPT values (Cohesion= 5N to 6N kN/ m^2 where N = SPT).

Table 1. Details of proposed breakwater

Contour (m)	K_d	Length (m)	Slope (V:H)	Top level of BW (m) CD
1.4 to 0	3	141.59	1:2	1.9
0 to -1	3	59.53	1:2	1.9
-1 to -2	3	70.52	1:2	2.7
-2 to -3	3	343.06	1:2	3.5
-3 to -4	3	153.11	1:2	4.3
-4 to -5 (round head)	3 2	144.27 75.1	1:25 1:3	5 5

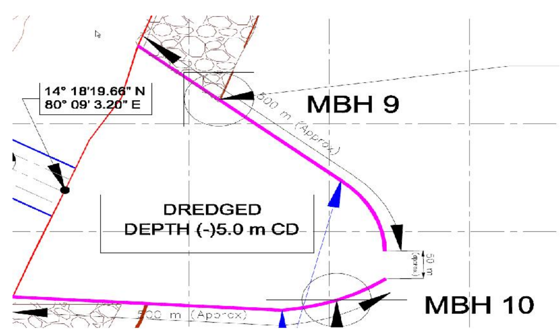

Fig. 4 Locations of the important boreholes (preliminary soil investigation) for breakwater design.

GROUND IMPROVEMENT SCHEME

The stability calculations are performed using the soil model derived based on the observed SPT N-values. The assumed soil model is given in the Table 2. The subsequent stability run revealed that the sub soil conditions are not suitable to support the proposed breakwater sections. Since the specified construction time is only 1.5 years, initially the breakwater supported on stone columns is proposed to get sufficient strength against embankment load of 11.5m high breakwater.

1m diameter bottom feed vibroreplacement stone columns is selected for the proposed ground improvement. Stone columns will be installed with 1.65 m c/c (15 to 18 m length) triangular grid spacing before Construction of Breakwater.

Table 2. Proposed Soil Model and Strength Properties considered for Breakwater slope stability analysis

Soil Layer No	Thk (m)	Description	Soil Strength	C_c	C_v in m^2/day
1 (0.0 to 6.5 m from G.L.	15	Soft Clay	10 kPa	0.365	0.0032
2 (6.5 to 20 m from G.L.	10	Medium Clay /Intermediate Clay	15 kPa		
3 (20.0 to 50.0 m from G.L.	20	Deep Clay	40 to 50 kPa and modeled as C_u/P_c = 0.22	Not considered in settlement calculations	
4	As per Fig. 3	Break Water Fill	C_U=0 , Phi = 40^0 considered		

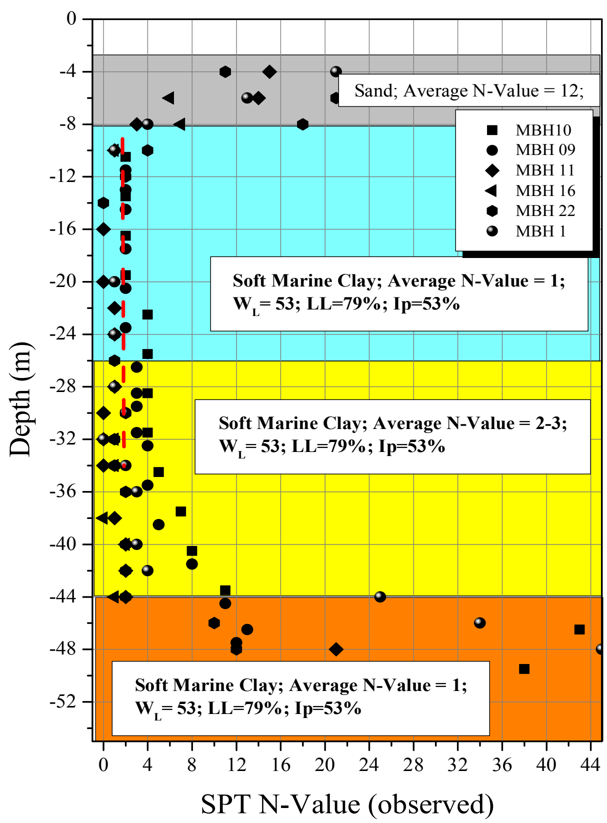

Fig.4 Geotechnical properties at site as per preliminary soil investigations.

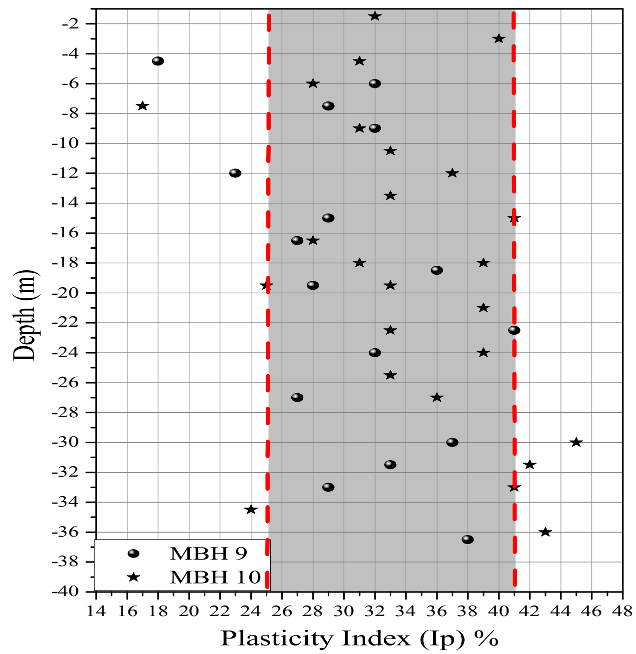

Fig. 5 Distribution of Plasticity Index for the site (MBH 9 & MBH 10)

The staged construction method was adopted for the construction of Breakwater. It was proposed to place the breakwater fill in the three stages. In the first stage 6.11 m height of the fill will be constructed and then left for the two month for allowance of the consolidation of the treated Ground. The installation of Geo-Grid at the Elevation 2.75m from the present ground level is considered. The Breakwater fill should be placed up to height 6.11 after the installation of Geo-grid. In the second stage the fill height will be raised upto 8.3 m fill height (i.e. second stage fill height = 8.3 – 6.11 = 2.19m). After finishing the second stage again 2 month waiting time will be given to stabilize the pore pressures and thereby achieving additional consolidation and strength gain. Finally in the third stage the breakwater will be raised up to 11.5m height (i.e. third stage fill height = 11.5 – 8.3= 3.2 m).

Although the proposal of installation would resolve the stability issue to the certain extent, the settlement computation is rather a challenge. Since the preliminary soil investigation failed to provide sufficient information on the consolidation properties of the underlying soft clay, the consolidation parameters were estimated through the plasticity index (Fig. 5, Table 2).

Barksdale and Bachus (1983) procedure was used to estimate the settlements for the stone column installed ground conditions. The estimated settlement reduction ratios are presented in the Table 3. It is important to note that the estimated settlement reduction ratios are within the range of 0.67 to 0.7 for the triangular stone column configurations (1.65 m c/c; 15m to 18 m length). However, subsequent discussions raised the issue regarding the settlements of the untreated ground below 15m.

Table 3. Total Cumulative Settlement of Clay stratum for the staged construction

Fill Height (m)	Settlement of upper 20m clay stratum in m		Settlement reduction ratio S_{wc}/S_C
	Without Stone Column (S_{WC})	With Stone Column installed (S_c)	
6.11	1.099	0.737	0.67
8.3	1.37	0.941	0.69
11.5	1.703	1.202	0.70

The stability of the breakwater embankment is modeled assuming stage wise construction. The profile method described by Barksdale and Bachus (1983) is used for stability modeling for stone column installed ground conditions. The strength gain is assumed at the end of each stage before proceeding for the next stage of construction and strength gain in clay is estimated using procedure given by varuiys researchers like Koutsoftas and Ladd (1985); Ladd (1991). The factors of safety obtained at the start of each construction stage are illustrated in Fig.6 to Fig. 8.

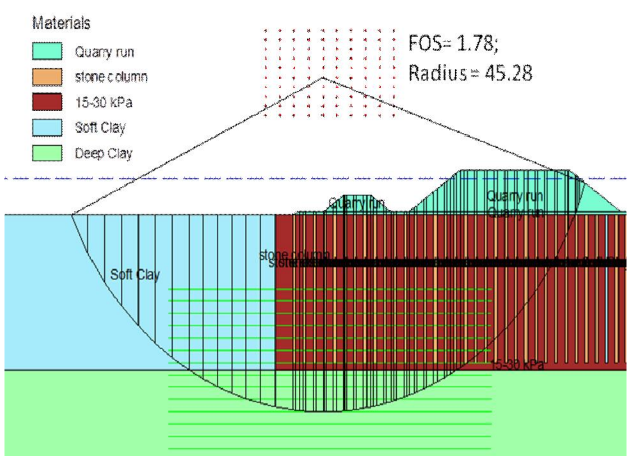

Fig.6 Calculated factor of safety for first stage breakwater construction (Bishop method)

There was practical difficulty associated with stone column installations at the stretch where the ground is -3m to -5 m CD. The installation rig did not have enough length to install the intended stone columns from the marine environments and contractor proposed to import the rig which in turn elevated the estimated cost. The scheme of installation of stone column at the foot print of the breakwater was selected as feasible option since the construction time associated with contract was limited i.e. 1.5 years. However, looking to the elevated cost associated with the installation of the stone columns, client was reluctant in the implementing the proposal. It was later agreed to allow more construction time i.e. 3.5 years to build the breakwater since the entire project was delayed due to coal linkage problem. Client has asked for some cost effective ground improvement solution which can be implemented within the increased time span. However, it is expressed that enough geotechnical information are not available to confidently model the in situ ground conditions. It was suggested to obtain in situ strength of soft clay stratum from fresh geotechnical investigations.

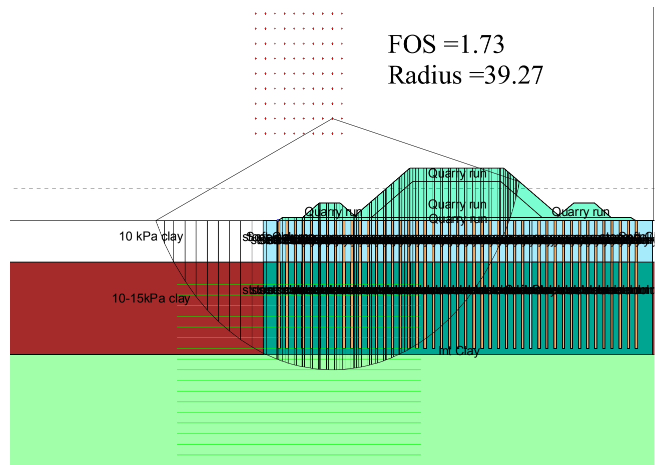

Fig.7 Calculated factor of safety for second stage breakwater construction (Bishop method)

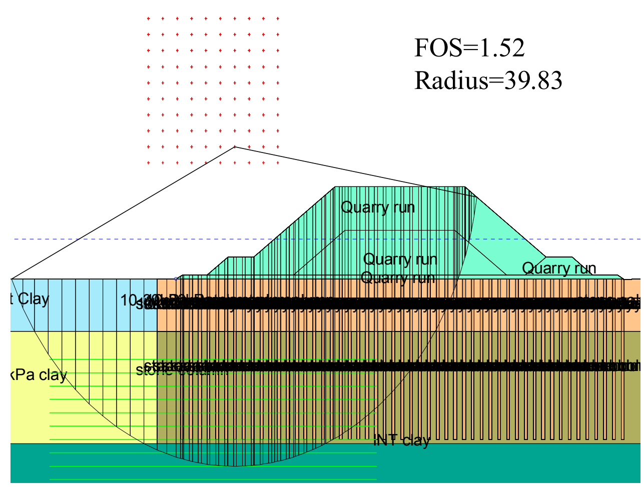

Fig.8 Calculated factor of safety for third stage breakwater construction (Bishop method)

FRESH GEOTECHNICAL INVESTIGATIONS

Fresh geotechnical investigations have been carried out in form of 4 Borehole investigations and 2 nos of SCPT tests. The test results of the Unconfined Unconsolidated triaxial test and Consolidated Undrained triaxial test data are presented in the Fig. 9. The variation of the coefficient of consolidation and initial void ratio are also presented in the Fig. 10 from which parameters are selected for settlement computations. One of the significant outcome of the fresh geotechnical investigation is confirmation of the existence of the 4m thick sandy/ silty sand layer on the top the the soft marine clay. Due to existence of this layer the stability scenario has changed completely. Additionally the measurement of clay strength via SCPT gave some confidence on the in situ clay strength. It is observed that the measured laboratory clay strength were reported to be less compared to the clay strength measured by SCPT test. It is interesting to note that the undrained clay strength (Cu) measured through SCPT data are in good agreement with SHANSEP model (Ladd, 1991) giving Cu = 0.18 to 0.24 times the effective overburden pressure (Fig. 11).Taking the advantage of the various findings in the fresh geotechnical investigations, the ground improvement scheme was revisited considering the increased construction time to optimize the cost of the breakwater construction. The revised soil model is presented in Table 4.

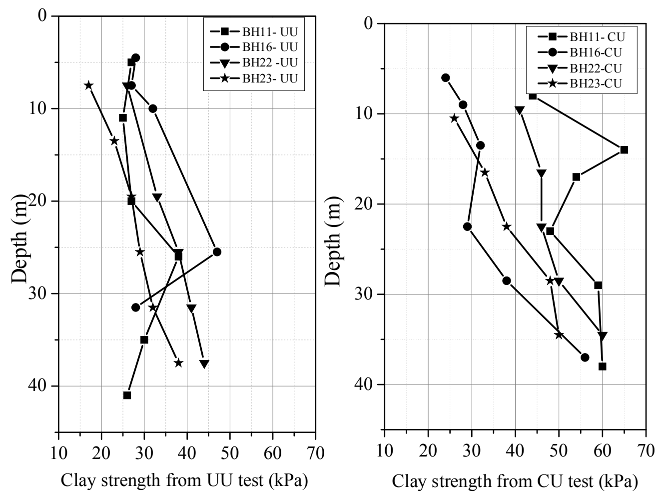

Fig.9 Measured clay strength from fresh geotechnical investigations (UU- Unconsolidated Undrained test; CU- Consolidated Undrained test)

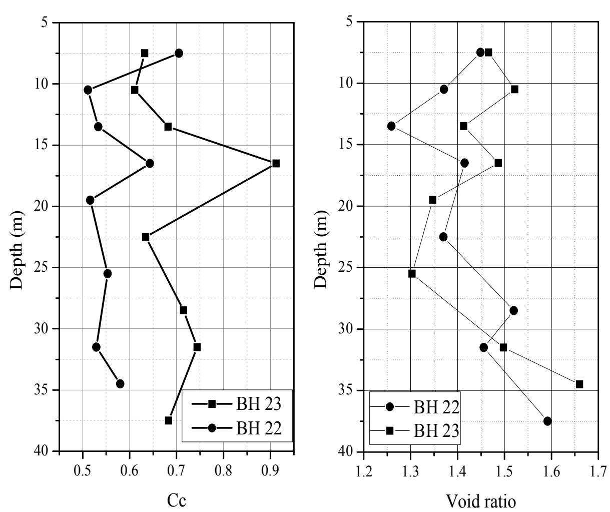

Fig. 10. Measured coefficient of consolidation (Cc) and initial void ratio from fresh geotechnical investigations

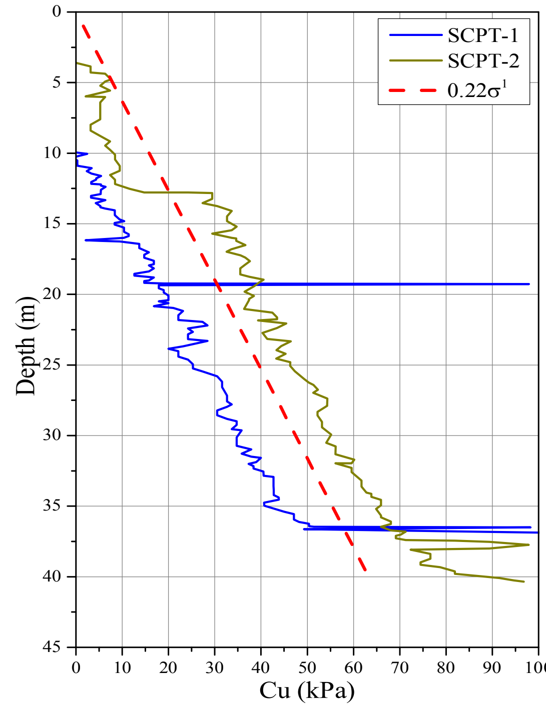

Fig. 11. Measured coefficient of consolidation (Cc) and initial void ratio from fresh geotechnical investigations

Table 4. Revises soil model for stability modeling

Soil Layer No	Description	Shear Strength parameters C_u (kPa)/ (Ø) (degrees)	Thk. (m)	C_c	C_v in m^2/day
1	Loose to Medium Sand	C_U=0 , Phi = 30^0	2.5	0.65 (e_0= 1.4)	3.2 e-3
2	Soft Clay	10-15 kPa	7.5		
3	Clay Layer 1	15-25 kPa	10		
4	Clay Layer 2	25 to 50 kPa and	15		
5	Clay Layer 3	modeled as C_u/P_c = 0.25 with base C_u=10	7.5		
6	Break Water Fill	C_U=0 , Phi = 40^0 considered	As per stage under consideration		

GROUND IMPROVEMENT USING PVDs

Since the given construction time is comparatively large, the ground improvement using Prefebricated Vertical Drains (PVDs) is explored. However, the stability modeling in given construction stage was challenging task since the strength gain in clay is purely depend upon the degree of consolidation. It is proposed to place the breakwater fill in six stages for the most critical full height section (i.e. at round head) and the number of stages will be reduced for the lower height sections within the stretch of breakwater.

Stage 1 Construct a 4 m high section of breakwater fill and then leave for 5-6 months to allow greater than 90% consolidation of the treated Ground (soft clay) and the associated strength gain.

Stage 2 The breakwater fill height will be raised by 2.5m to 6.5 m (i.e. second stage fill height = 6.5 – 4 = 2.5m). After completion the fill will again be left for 5-6 months to allow greater than 90% consolidation and the associated strength gain.

Stage 3 The breakwater fill height will be raised by 2m to 8.5m height (i.e. third stage fill height = 8.5 – 6.5= 2 m). After completion the fill will again be left for 5-6 months for the same consolidation stage as with stage 2.

Stage 4 The breakwater fill height will be raised by 1.5m to 10m height (i.e. third stage fill height = 10 - 8.5 = 1.5 m). After completion the fill will be left for 3-4 months for the consolidation period.

Stage 5 The breakwater fill height will be raised by 1m to 11 m height (i.e. third stage fill height = 11 – 10= 1 m). After completion the fill will be left for 2-3 months for the consolidation period.

Stage 3 Finally, the breakwater fill will be raised by 0.5m to 11.5m height (i.e. third stage fill height = 11.5 – 11= 0.5 m). After completion the fill will be left for 2-3 months for the consolidation period.

Typical Time vs Staged height (embankment loading) curve as proposed is shown in the Fig. 12. After consolidation of the clay layer under the application of the applied stage 1 loading, there will be strength gain in the insitu soft clay. The possible strength gain in the clay layer is estimated after application of each stage of embankment loading and revised strength is then applied for the stability analysis to verify the factor of safety under application of the next stage embankment load. A value of 90% of the maximum strength gain is assumed at the center of the embankment and averaged reduced strength gain is assumed for the clay soil under sloping portion of the embankment. The 90% relates to the degree of consolidation achieved during the loading period of 5 to 6months.

The possible variation of the undrained cohesion (C_U), in the second stage of construction is estimated using the spread sheet and recommendations made by Tavenas and Leroueil (1980); Bergado et al. (1991); Ladd (1991); Chai et al. (1994) are referred to for the strength gain calculation with an assumed maximum Cu value due to loading given by $C_u/\sigma' = 0.25$. A geotextile layer (tensile strength 300 kN/m) is proposed to laid at the elevation of 4m to counter the stability issue in the second and subsequent staged construction. Appendix A describes possible variations of the effective stresses and strength gain in each construction stage. Considering the strength variation of the subsoil profile given in the Table A-1, slope stability analysis has been carried out and the obtained Factors of safety are presented in Table A-2.

The estimated cumulative consolidation settlements are presented in Table 5. Since the depth of PVD installation is very high i.e. 35m from the sea bed level, the construction aspect of installation was explored by discussing the feasibility with various contractors. In order to It was felt that installation of the PVDs in marine environment up 37m is uncommon in India and reliable case history is not available for its performance. The partial depth of improvement is explored for its suitability and serviceability requirements. However, it was really a challenging task to recommend the exact depth and exact spacing to ensure the desired degree of consolidation at each stage of construction (starting and ending) within the specified time span. In case of partial depth improvement scenario, the post construction settlement must be within the limit to obtain the stable shape of breakwater.

Two alternatives are finally derived after exploring several possibilities. Alternative 1, in which PVDs are installed upto full depth (37m) with staged constructions (Fig. 13). Alternative 2, in which PVDs are installed upto 17m only (Fig. 14). Ensuring the stability at each construction stage, the settlement scenarios are compared in the Table 7. The overall influences on the alternatives are highlighted in Table 8.

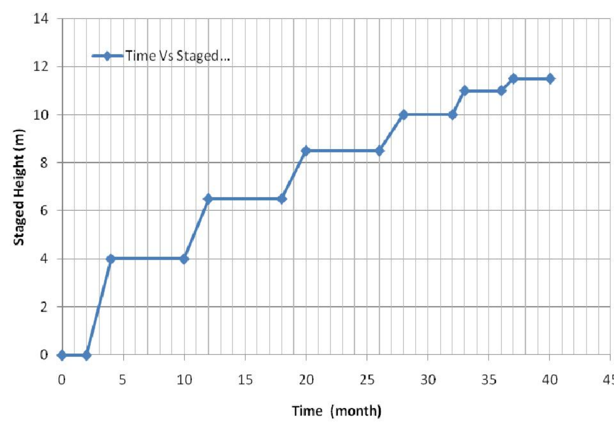

Fig. 12. Time vs Staged construction height scenario

Table 5. Total Cumulative Settlement of Clay stratum for the staged construction

Construction Stage	Cum. fill height (m)	Cum. consolidation settlement (m) (IS:8009 part I)
Stage 1	4	1.109
Stage 2	6.5	1.964
Stage3	8.5	2.633
Stage 4	10	3.046
Stage 5	11	3.280
Stage6	11.5	3.389

Table 6. Estimated Time required (in month) to achieve

specified combined degree of consolidation with various PVD spacing

Combined Degree of Consolidation (%)	PVD Spacing (m)				
	0.8	0.9	1	1.1	1.2
	Time (month)				
80	1.128	1.520	1.980	2.510	3.111
85	1.329	1.792	2.334	2.958	3.667
90	1.613	2.175	2.833	3.591	4.450
95	2.099	2.830	3.686	4.671	5.790
97	2.457	3.312	4.314	5.468	6.777

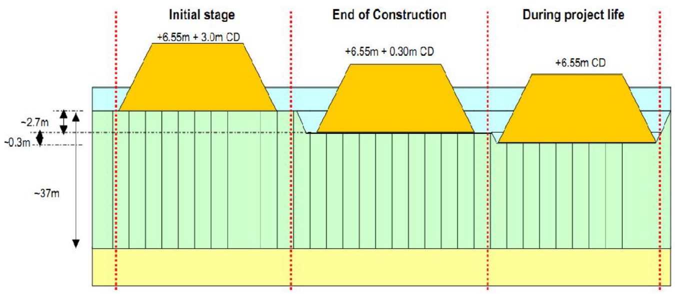

Fig. 13. PVD installation Alternative -1; PVDs installed up to full depth i.e. 37m

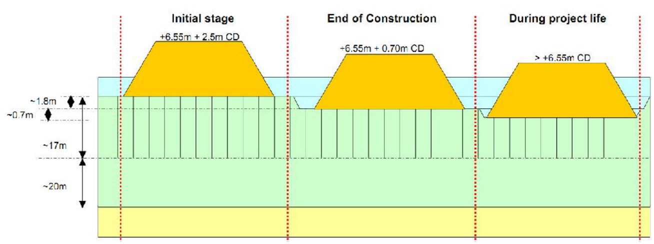

Fig. 14. PVD installation Alternative -2; PVDs installed up to partial depth i.e. 17m.

Table 7. Calculated settlements for alternative PVD installation scheme.

Layer thk. (m)	% consolidation*	Settlement (m)*	% consolidation**	Settlement (m)**
Alternative 1 (PVD installed up to full depth)				
17	90	1.8	10	0.2
20	90	0.9	10	0.1
	Total	2.7	Total	0.3
Alternative 2 (PVD installed up to 18m – partial depth)				
17	90	1.8	10	0.2
20	0	0	50	0.45
	Total	1.8	Total	0.65

*- during construction
**- post construction 30 year scenario

The comparison of the the alternative PVD installation scheme revealed that the partial installation of PVD can result in substantial cost saving compared to full depth installation. However, post construction settlement for the partial depth alternative may give higher post construction settlements (0.65-0.3 = 0.35m). The amount of increase in post construction settlement is very less than that for the efforts required to install PVDs for full depth of clay layer. Further to facilitate the decision making, the comparison is highlighted in the Table 8 which suggests that partial depth installation alternative has many advantage over the full depth of PVD installation provided that the post installation settlement limit is relaxed. For example, for full depth of PVD installation, it is required to bring very specific equipment since the depth of installation is 40m (plus water height from where PVDs are installed) whereas partial depth installation can be carried out using normal equipment available in Indian market. Similarly there is substantial saving on the installation length of PVDs (almost double qty required for full depth of PVD installation) which will also save considerable construction time as well.

Table 8. Comparison of the PVD installation alternatives.

Description	Alternative 1	Alternative 2
Residual Settlements (m)	0.3	0.65
Additional qty. of breakwater material due to settlement	3.3 m^3	3.5 m^3
PVD running length per installation	40	20
PVD installation equipment	Very specific equipment capable to install upto 40m	Normal equipment capable to install upto 25 to 28m

Finally the total cost of both alternatives were calculated and presented to the client to get the concern over the post construction settlement limits. Since the total cost of the construction was substantially less for the partial depth of the ground improvement, same was proposed as possible solution for construction of stable breakwaters in such difficult sub soil conditions. However, stability analysis was based on the staged construction method suggested by Koutsoftas and Laad (1985); Ladd (1991); which requires consolidation parameters from oedometer tests and shear strength derived from field vane tests and triaxial tests. Therefore, more detailed, structural location specific site investigations were recommended to obtain sound information on the consolidation parameters, pre-consolidation pressures and shear strength of the in situ ground. Since the stability of each stage construction is based on the strength gain assumptions, detailed instrumentation program was suggested to verify the degree of consolidation and corresponding strength gain at each construction stage.
The measurement of the settlement can provide a reasonable indication of consolidation achieved in the field and measurement of pore water pressure ensures that subsequent stages of construction are not undertaken until the required consolidation for clay strength gain has been achieved. This method will greatly reduce the risk of catastrophic slip failure

during any stage of construction.

The settlement calculated to occur both during and after construction must be considered as they notably increase the quantity of breakwater material needed to reach the full height required by the design. Based on the assumptions made in this study, it was recommended that ground improvement can carried out by installing pre-fabricated vertical drains (PVDs) up to partial depth and with a staged construction approach, it is possible to construct the breakwater up to 11.5m height. The low height sections i.e less than 8.5m high was proposed to built within 24 (±3) months whereas the critical section of 11.5m height is likely to take longer due to more stages required to reach this height with the subsequent time required for consolidation and placing the fill material. .As shown in the analysis the construction of this breakwater on the poor ground conditions at the site will require a slow staged construction approach with careful instrumentation and monitoring to safeguard against any slip failures that may jeopardize the project.

CONCLUSIONS

The proposed power project site on the South East coast of India required the construction of breakwaters to prevent detrimental effects owing to littoral drift along cost and also to protect the intake channel lining against sea attack. Present study describes the geotechnical design and analysis for breakwater facility proposed for the site. Geotechnical site conditions were very poor and ground improvement was inevitable to support 11.5m high breakwater (round head). To get the techno commercial trade off, various ground improvement scheme were explored and finally it was decided to recommend installation of PVDs as feasible alternative. From the study following conclusions can be drawn.

- In sufficient geotechnical data may result over conservative design solutions. The good quality geotechnical data enables engineer at early stage of the project to explore various alternative schemes to achieve techno commercial balance in competitive environments.

- The strength gain approach can be suitably applied for geotechnical solution for such difficult sites implementing the stage construction sequence for stability modeling of breakwater and similar structures. Post construction settlement is considered to be most important and sensitive parameters that need proper evaluations as it can lead to complete collapse of high embankment type structure in long term if no assessed properly at design state. Needless to mention, design adjustment based on monitoring strength gain and settlement during construction is essential to validate the final design issued for construction.

- The alternative comparison of the alternative ground improvement scheme revealed that in given subsoil conditions in the present project, it may be possible to implement partial depth ground improvement to achieve economy for construction of stable structures in marine environments provided that the post construction stability issues warrant it.

- The construction time for the large project is quite important to minimize the project cost. Minimizing the project execution time may imbalance the project cost-benefit ratio and hence suitable equilibrium between the time-cost-benefit shall be ensured for large projects.

REFERENCES

Chai J.C., Sakajo S. and Miura N. [1994]. *Stability analysis of embankment on soft ground.* Soils and Foundations, Vol. 34, No. 2, pp. 107-116.

Barksdale R. D. and Bachus R. C. [1983]. *"Design and Construction of Stone Columns"*. Vol I, Report No. FHWA/RD-83/026.

Bergado, D.T., Alfaro M.C. and Chai J.C. [1991]. *The granular pile: its present state and future prospects for improvement of soft Bangkok clay.* Geotech. Eng., Southeast Asian Geotechnical Society, Vol. 22, No. 2, pp, 143-176.

IS: 8009 Part I- [1979]. "*Code of practice for calculation of settlement of foundation (shallow foundation subjected to static vertical loads)"*. Bureau Indian standards. New Delhi, India.

Koutsoftas, D.C. [1981]. "*Undrained shear behaviour of a marine clay"*. Laboratory shear strength of soil, ASTM STP 740, Yong & Towsend (eds.), pp. 254-276.

Koutsoftas, D.C. and Ladd, C.C. [1985]. "*Design Strengths for an Offshore Clay*". Journal of Geotechnical Engineering, ASCE, Vol. 111, No. 3, pp. 337-355.

Ladd, C.C. [1991]. "*Stability evaluations during staged construction*". Journal of Geotechnical and Geoenvironmental Engineering, ASCE 117 (4), 540–615.

Tavenas F. and Lroueil S. [1980] "*The behavior of embankments on clay foundations*". Can. Geotech. J., Vol. 17, pp 236-260.

APPENDIX -A

Table A-1. Estimated strength variation of the in situ clay layer under each stage of embankment loading.

Elevation from Sea bed (m)	At the End of Stage 1			At the End of Stage 2			At the End of Stage 3			At the End of Stage 4			At the End of Stage 5			At the End of Stage 5		
	σ' (kPa)	Total cohesion available Cu(kPa)		σ' (kPa)	Total cohesion available Cu(kPa)		σ' (kPa)	Total cohesion available Cu(kPa)		σ' (kPa)	Total cohesion available Cu(kPa)		σ' (kPa)	Total cohesion available Cu(kPa)		σ' (kPa)	Total cohesion available Cu(kPa)	
		Under triangular portion	Under central portion		Under triangular portion	Under central portion		Under triangular portion	Under central portion		Under triangular portion	Under central portion		Under triangular portion	Under central portion		Under triangular portion	Under central portion
A	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**	**J**	**K**	**L**	**M**	**N**	**O**	**P**	**Q**	**R**	**S**
-2.5	55.13	11.60	12.13	95.32	17.98	23.83	135.3	23.83	33.83	165.28	37.57	41.32	185.24	43.81	46.31	195.21	47.56	48.80
-10	54.77	16.69	18.37	94.59	24.61	30.84	133.72	35.73	40.62	162.48	44.22	47.81	181.08	50.13	52.46	190.08	53.58	54.71
-20	52.86	25.00	25.00	90.81	31.04	37.08	126.35	41.53	45.97	151.05	49.06	52.14	166.07	54.02	55.89	172.99	56.76	57.63
-35	49.64	50.00	50.00	81.03	50.00	50.00	109.68	50.00	50.00	128.4	51.84	53.67	139.23	55.02	56.37	144.08	56.98	57.59

Table A-2. Obtained Factors of Safety at each stage of embankment loading.

Stage	At Start of Stage 1	At Starti of Stage 2	At Start of Stage3	At Start of Stage 4	At Start of Stage 5	At Start of Stage 6	At end of Stage 6 (during hand over of Breakwater)
Clay strength used (Ref)	Table 1	Table 6 Column (C,D)	Table 6 Column (F,G)	Table 6 Column (I,J)	Table 6 Column (L,M)	Table 6 Column (O,P)	Table 6 Column (R,S)
Factor of Safety	1.79	1.24	1.26	1.3	1.27	1.22	1.33