Offshore Stone Columns Under Embankment With Embedded Pipes

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OFFSHORE STONE COLUMNS UNDER EMBANKMENT WITH EMBEDDED PIPES

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ABSTRACT

A breakwater embankment was constructed as part of a settling basin for the intake of a thermal power plant in the north of Egypt. The 440-m long breakwater surrounds the intake basin which has an area of about 18000 m2. Sea water is supplied to the intake basin through 50 concrete pipes embedded in the breakwater. Subsurface soil conditions in the area indicated that the embankment would be founded on soft silty clay underlain by a layer of silty sand. Results from the settlement analysis showed that the embedded pipes could not tolerate the predicted differential settlement in the embankment. Accordingly, it was determined that the embankment could not be founded on natural soil and that this soil would require improvement using stone columns.

Offshore stone columns were constructed using the blanket method utilizing two vibro flotation probes. The layout was designed such that the stone columns were arranged in a triangular pattern below the embedded pipes, and in a rectangular pattern elsewhere. This paper presents the design method using numerical modeling to show the amount of expected settlement with and without stone columns. The method of construction is discussed showing how the stone columns were constructed using the blanket method. The full-scale load test that was constructed offshore to validate the design is described. The results from the load test and post-construction settlement readings indicated that the stone columns proved to be an efficient and economical solution in reducing the differential settlement to tolerable limits for the embedded pipes.

INTRODUCTION

A breakwater embankment is constructed with an average length of about 440m to surround the intake basin which has an area of about 18000 m^2. It is used to enter the sea water inside the basin through 50 concrete pipes embedded in the front side of the breakwater embankment (north side). The area inside the breakwater was dredged to level -4 MSL (Mean Sea Level).

The zone under study is zone (B), where the sea bed level is varying from -3.50 to -4.75MSL. The location of this zone is enclosed in Figure 1. The subsurface condition for this zone comprises a top soft silty clay layer of variable thickness followed by silty sand layer. The results of site investigations in terms of soil formation and strength/stiffness parameters shows that for this area the breakwater if founded on natural soil without improvement will experience excessive total and differential settlement that could break the embedded pipes after installation.

OUTLINES OF THE SOIL IMPROVEMENT SYSTEM

The main targets of the soil improvement system can be summarized as follows:
1- Increase the factor of safety for bearing capacity failure to the allowable value.
2- Increase the factor of safety for the slope stability to the allowable value.
3- Decrease both the total and the differential settlement to the allowable values.

Stone Columns

Vibroflotation (Vibro compaction) / Vibro replacement techniques are famous techniques used to reduce the settlement and also improve the bearing capacity and the embankment slope stability safety factors. VIBRO Replacement is a combination of Vibroflotation with a gravel backfill resulting in stone columns, which not only increases the amount of densification, but provides a degree of reinforcement and a potentially effective means of drainage. Spacing of the probe compaction points depends upon soil type, density requirements, and probe/vibrator characteristics. Typical spacing range from 1.5 to 4 m (5 to 12 ft). The vibro replacement causes the reduction of settlement and increase of shear strength. The technique offers a double benefit by reducing the settlement in addition to improving the ground sufficiently to allow foundation support on rafts spread footings.

The use of stone column for such a project is mainly to decrease the total settlement and differential settlement; also it contributed for the stability requirements. The rule of the stone columns is to decrease the settlement due to the following:

• Increasing in lateral stress
• Increasing in stiffness modulus
• Improved drainage.

The stone columns are arranged in equilateral triangular and rectangular pattern. The spacing between the columns “S” shall be determined based on the allowable stress and the induced settlement. The stone columns are 1.0 m in diameter. The stone columns must have a minimum penetration of 1.0 m inside the lower silty sand layer.

Geogrid Reinforcement

The use of basal geogrid reinforcement for embankments on soft clay is a recent technique described and defined thoroughly in British standards8006-1995-section 8 entitled “Design of embankment with reinforced soil foundation on poor ground". The main rule of the basal geogrid reinforcement is to ensure an acceptable margin of short term stability during different stages of construction with a minimum holding period between different construction stages. For area of zone where the pipelines entering the breakwater, the basal reinforcement layer shall be placed at just below the bottom level of the intake pipes. For other areas of zone B, the geogrid shall be placed at level +0.5 MSL. The geogrid layer shall assume to have long term strength of 275 kN/m.

BASIC OF GEOTECHNICAL DESIGN

The geotechnical analysis for the breakwater should consider safety of the breakwater body against soil shear failure with suitable safety factor for short and long term conditions. Also the geotechnical design must address the settlement analysis of the breakwater which must be considered in the design of pipes so as the used pipes must be designed to tolerate the anticipated values for both total and differential settlements.

Geotechnical Soil Parameters

The idealized geotechnical formation and the idealized geotechnical parameters of clay can be summarized as follows:

Table 1. Soil Parameters

Top/ bottom level (MSL)	Soil unit weight (kN/m^3)	Case of during construction		Case of Final condition	
		Shear strength parameters	Deformation parameters	Shear strength parameters	Deformation parameters
-3.50 to -4.75 / -11.00	16	c_u = 15 kN/m^2 ϕ = 0	E_u = 3300 kN/m^2 μ= 0.49	c_d = 0 kN/m^2 ϕ = 25^o	E_d = 3000 kN/m^2 μ= 0.35 C_v=0.90m^2/year
-11 / -14	17.5	c = 0 kN/m^2 ϕ= 36^o	E = 18000 kN/m^2 μ= 0.33	c = 0 ϕ= 36^o	E = 18000 kN/m^2 μ= 0.33

External load

For the purpose of the geotechnical analysis a distributed load of 33 kN/m^2 shall be considered affecting on the top of the top of breakwater to simulate the traffic loading.

Water level

The water level shall be considered as the zero level (MSL).

Seismic Parameters

According to contract specifications, for design of permanent structures, a ground acceleration of 0.15g shall be considered in the analysis. For Plaxis finite element analysis, the PIANC (Seismic design guidelines for port structure, international navigation association) had been used in order to evaluate the percent of sliding mass to be considered in the lateral analysis using Pseudo static method. In addition, the Eurocode 8 (part 5) had been used to evaluate the percent of sliding mass to be considered in the lateral analysis using Pseudo static method. The two methods had led approximately to the same result which indicates that for finite element Pseudo static analysis, seismic horizontal coefficient Kh shall be about 0.12g of the sliding mass.

Embankment geotechnical parameters

The embankment materials shall be stones. The angle of shearing resistance of the stones shall be considered as 40 degree while the saturated unit weight shall be considered as 18 kN/m^3. The deformation modulus for the embankment materials shall be considered as 70000 kN/m^2.

Stones geotechnical parameters

The angle of shearing resistance of the stones used for columns shall be considered as 40 degree while the saturated unit weight shall be considered as 18 kN/m^3. The deformation modulus for the stone materials shall be considered as 100000 kN/m^2.

DESIGN OF STONE COLUMNS FOR SETTLEMENT CONTROL

The most critical area is the area where the pipes are penetrating the embankment. The maximum load of the embankment will be under the crest of the embankment while the minimum load shall be at the sides. This condition may create a large differential settlement if equals spacing between stone columns had been used. This differential settlement can be reduced by changing the spacing between the stone columns. Under embankment crest stone columns with triangular arrangement at spacing 1.5 m is used while a spacing of 2.0 m in rectangular arrangement shall be used below the slopes of the embankment.

For the area where no pipes, the stone columns is arranged at spacing of 2.0 m. Stone columns are arrangement is shown in Figure 1.

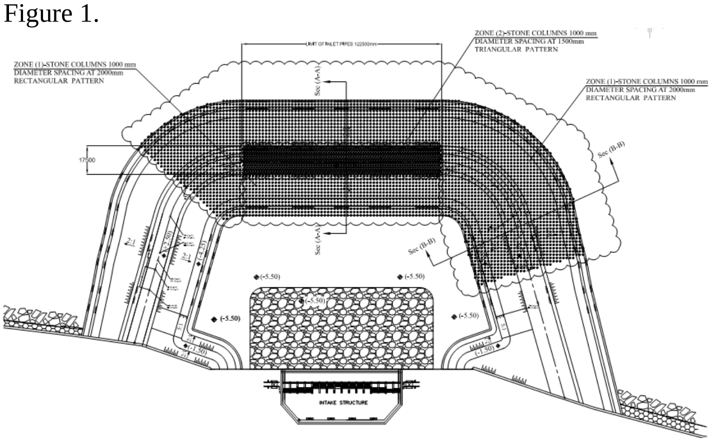

Fig. 1. General Lay out of Stone Columns.

NUMERICAL MODEL

The breakwater structure is located on Mediterranean Sea. The high water level in the sea is (0.63 MSL) while the minimum water level is (-0.73 MSL). As previously discussed three models shall be addressed for the geotechnical analysis.

The geotechnical analysis shall consider different soil and loading condition.

Methods of analysis

The breakwater structure shall be analyzed for all construction stages and all anticipated loading conditions. The main analysis shall be carried out using the computer programs PLAXIS 8.6, which are widely, used finite element package that accounts for the coupled soil-water behaviour. A 15 node plain strain elements shall be considered. The analysis shall be performed on step by step in order to simulate different stages of construction. For each state the program calculates the maximum deflection as well as the overall stability safety factors. Figure 2 shows the general configuration of the Plaxis model development.

Stages simulated in the FEM Methods of analysis

In the finite element analysis, the following stages shall be simulated:

1- Installation of the stone columns.
2- Backfilling up to water level.
3- Installation of the Geogrid layers.
4- Complete backfilling up to final level.
5- Applying the external surcharge load.

For selected stages, the factor of safety using C-Φ reduction method shall be calculated.

Plane strain modeling of stone columns

Actually, the circular stone columns arranged in either rectangular or triangular pattern are a 3-D problem. In 2-D analysis (plane strain analysis), the stone columns are modeled as trench. In general, the plane strain properties need to be adjusted to account for the geometrical change many methods had been presented in literatures in order to simulate the stone columns in 2-D problems. The method proposed by S.A.TAN 2008 which shall develop for stone columns arranged in rectangular pattern shall be modifies to suit both rectangular and triangular pattern shall be used. The plane-strain materials stiffness is given by the following relationship based on the matching of the column-soil composite stiffness.

$$E_{c,pl}*as+E_{s,pl}*(1-as)= E_{c,3d}*as +E_{s,3d}*(1-as)$$

Where "as" is the area replacement ratio, E_c denote deformation modulus for stone, E_s denotes deformation modulus of natural soil, "pl" denotes for plane strain condition and "3d" denotes 3-D condition.

For rectangular pattern, this relation can be put in the following form for a trench of stone columns with width "D" arranged at spacing "S":

$$E_{c,pl}*D*S+E_{s,pl}*(S-D)*S= E_{c,3d}*as +E_{s,3d}*(1-as)$$

For rectangular pattern, this relation can be put in the following form for a trench of stone columns with width "D" arranged at spacing "S":

$$E_{c,pl}*D*0.86\ S+E_{s,pl}*(S-D)*8.86S= E_{c,3d}*as +E_{s,3d}*(1-as)$$

The same formula can adopted for the angle of shearing resistance ϕ by replacing the deformation modulus E with $\tan\phi$

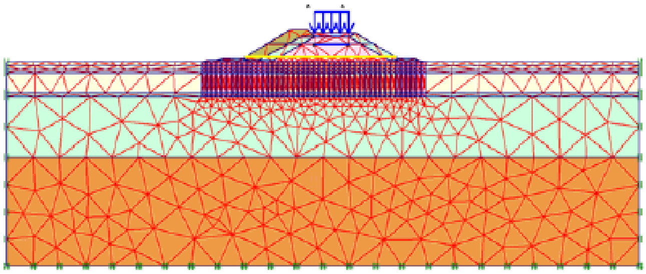

Fig. 2. General Configuration of the PLAXIS Model

Modelling of geogrid layers

In finite element analysis, the geosynthetic material shall be modelled as a membrane (geogrid) element allowing only for axial tension force with appropriate normal stiffness. The geogrid element modelled with elasto-plastic behaviour, the plasticity (failure) load had been considered, conservatively, as the long term strength of the geogrid in both short term and long term condition. For required long term strength is 275 Kn/m and the allowable strain as the same standard is 5%, the properties of the geogrid layers had been considered as follow:

1-Plastic load: 275 Kn/m.

2-Normal stiffness EA = Plastic load/allowable stain = 5500 Kn/m.

Constitutive models and soil parameters.

Constitutive models can can be grouped into linear elastic, non-linear elastic (hyperelasticity, hypoelasticity), variable moduli, elasto-plastic, elastovisco-plastic, cap models and hypoplasticity. Several references can be cited in connection with the formulation of constitutive soil model (Duncan and Chang 1970; McCarron and Chen 1987; Kirkgard and Lade 1993)

The soil layers shall be modeled using Mohr Coulomb model (MC)which is appropriate for this type of problems (embankment loading). This shall be applied for all layers except the top concrete cap and the deep silty sand layer.

For the top cap, it shall be modeled as linear elastic (LE)with appropriate value for Elastic modulus.

The lower deep silty sand layer shall be modeled using hardening soil model (HSM) in order to represent the increase in soil stiffness with depth and to reach a reasonable vertical deformation value. The power (m) for HSM was assumed as 0.5 for clay (Brinkgreve 2002) The soil parameters used in the analysis can be summarized as follows

Material	Constitutive model	C Kn/m2	Φ	E_{ref} MN/m^2	E_{oed} MN/m^2
Cap	LE	------	----	21000	
Core and shell	MC	5	40	50	
Silty clay	MC	0	25	3	
Silty sand	HSM	0	36	50	45

Type of analysis.

The analysis of the breakwater shall consider both undrained and drained conditions. The undrained analysis represents the short term condition which simulates the different stages of construction while the drained analysis represents the long term analysis which simulates the final embankment condition. In the undrained analysis, it is permitted for the built of pore water pressure in cohesive soils and consequently decreases the effective stress which lead to less Figure 3- Distribution of vertical displacement- Short term

soil strength. This type of analysis shall be used only for the soft clay layers, while for non-cohesive layers the built up of bore water pressure is not allowed as these layers dissipate any excess in pore pressure due to the high permeability. Indeed, this type of analysis shall be critical for stability analysis where minimum factor of safety is achieved.

In drained analysis, the cohesive soil is considered fully consolidated under the weight of the breakwater and hence no excess pore pressure is produced and consequently the soil rebound its effective strength. Indeed, this type of analysis shall be critical for deformation analysis where maximum settlement is anticipated in long term condition.

Results Of Numerical Model

The results of the analysis showed that for undrained analysis (short term condition), the settlement shall be in range of 10 cm while the factor of safety upon backfill to sea level and before installing the geogrid shall be 1.69. Figures 3 and 4 show the distribution of the vertical deformation and the anticipated failure surface

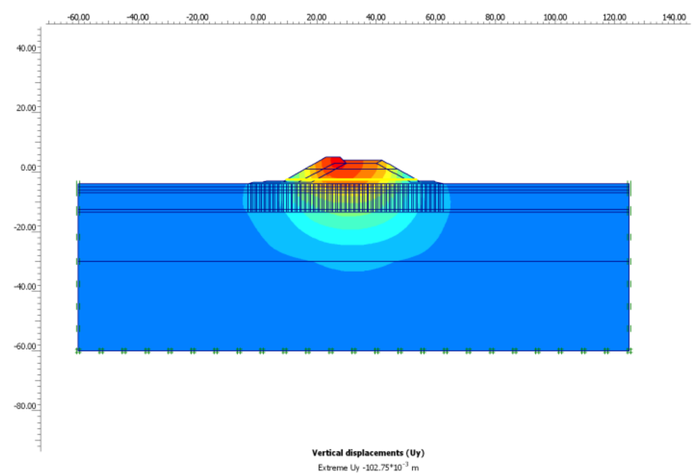

Fig. 3. Distribution of vertical displacement- Short term

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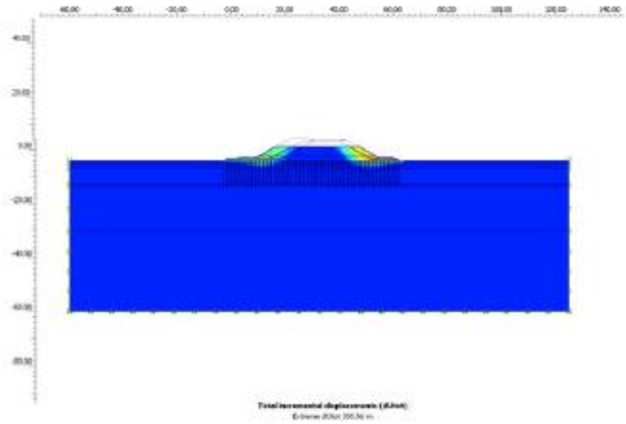


Fig. 4. Anticipated failure surface- Short term

For long term condition, the settlement shall be in range of 16.5 cm while the factor of safety in final drained condition shall be 1.69. Figures 5 and 5 show the distribution of the vertical deformation and the anticipated failure surface

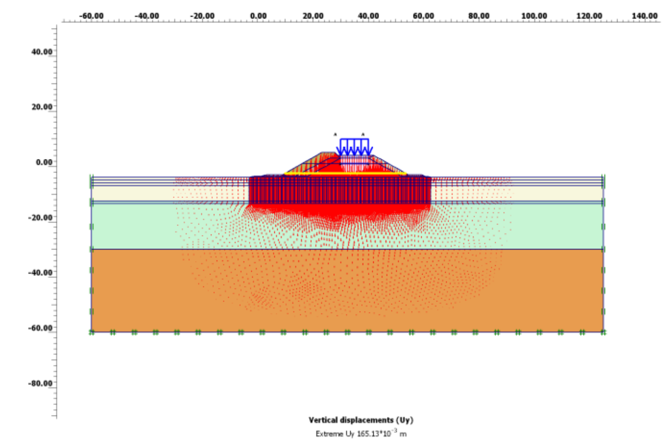


Fig. 5. Distribution of vertical displacement- long term

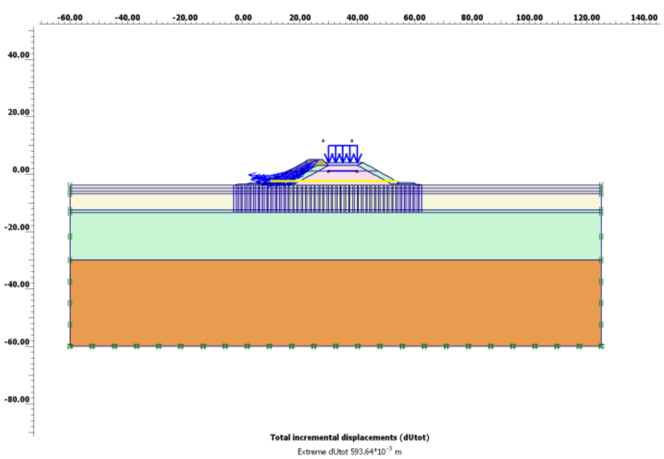


Fig. 6. Anticipated failure surface- long term.

The above results of analysis showed mainly that factor of safety for all cases of loading are within the allowable values. In addition the results of the analysis had been used to plot the settlement profile in both short term and long term as shown in figure 7

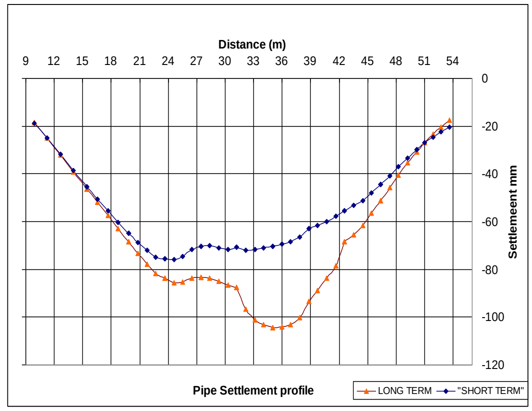


Fig. 7. Settlement profile for pipes

The above profile had been used to calculate the angle of rotation between the between the different segments of the pipes 3.5 m length. These calculations showed that the maximum differential angle of rotation is less than the allowable value as per vendor requirements.

FIELD LOAD TEST

<u>Test General Configuration</u>

In order to verify the theoretical design, a field test had been performed on the constructed stone columns using placement of concrete blocks represents the required tests load. The used blocks are 100 blocks of 1.55x1.55x1.55 m in dimensions which form 4 rows (each row has 25 blocks) as well as 1 row of another 25 blocks of 1.35x1.35x1.35 m in dimensions. The tested area is 7.75x7.75 m.

The blocks had been placed starting from level higher than the sea bed by 1.5 m. The 1.5 m had been filled with crushed stones layer which was used for levelling and load distribution, considering that the bed level in the area of study is (-4.10) MSL, the bottom level of the blocks had been (-2.60) MSL.

Figures (8) shows a plan and a cross section for the test load arrangement.

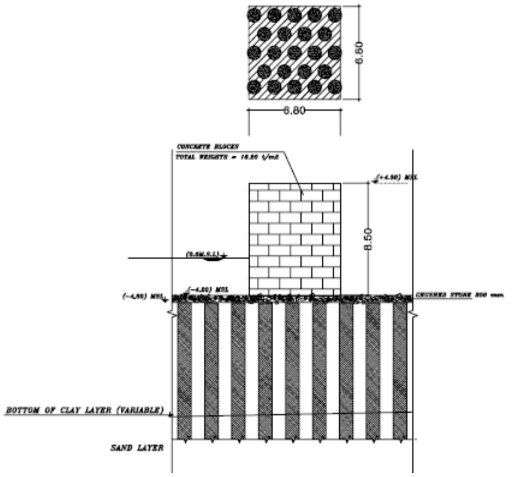


Fig. 8. Load test plan and section

Load Test

The used test loads are composed of concrete blocks of unit weight 2.40 t/m3. In addition, the unit weight of the crushed stone (levelling) layer is considered 1.80 t/m3.

Monitoring

The duration for the test had been 3 weeks The test has been stopped after this period due to the steady of the recorded readings. The monitoring process was carried out via choosing 9 blocks at the top surface and fixing 5 elevation reference points per each block. This monitoring system constitutes 45 points which represent most of the top surface of concrete blocks.. The layout of the measuring points is shown in figure 9

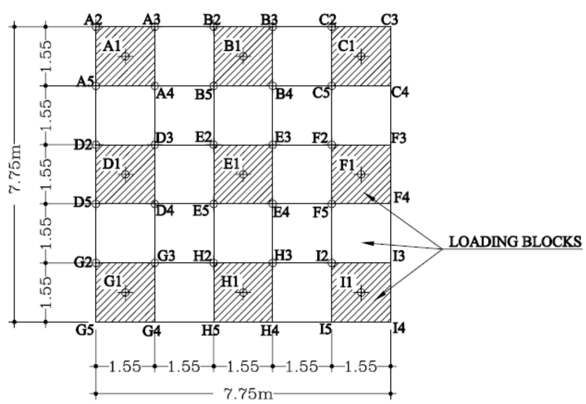
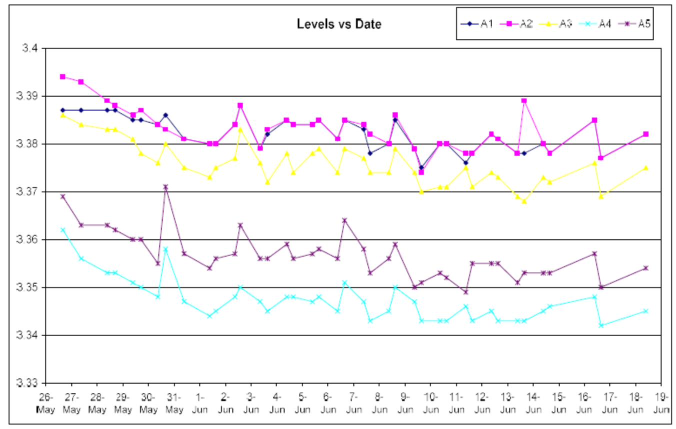
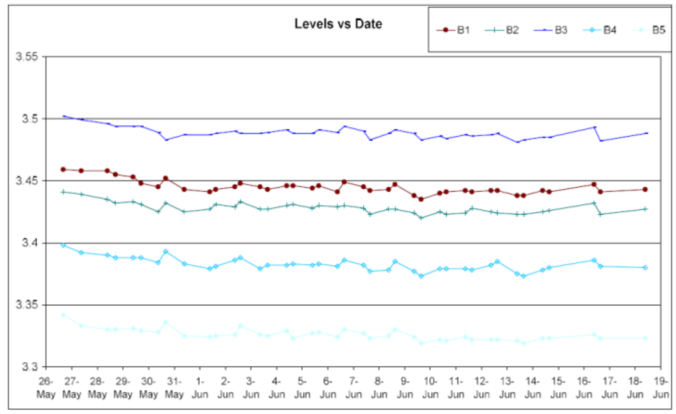


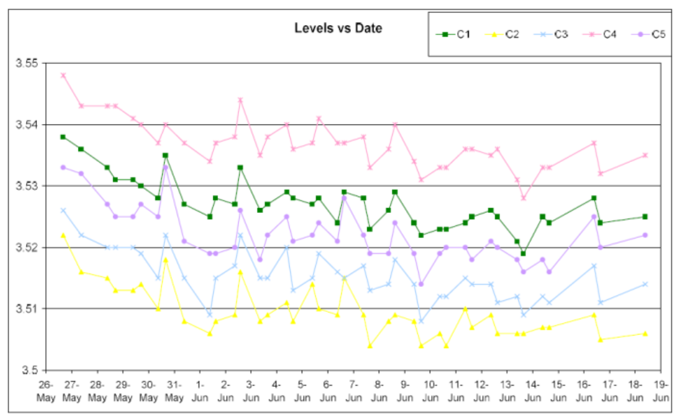
Fig 9: Location of measuring points

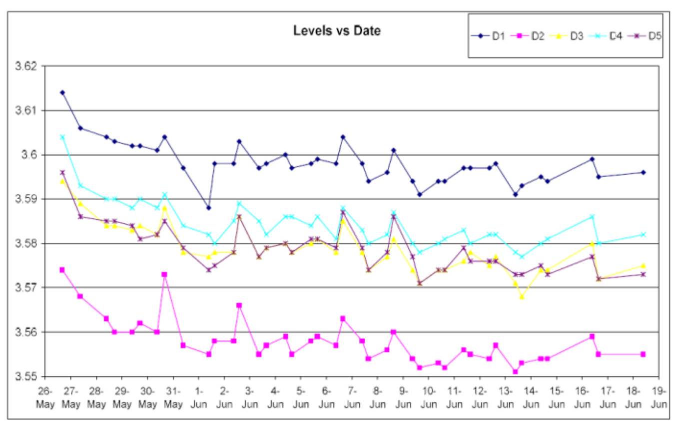
Test readings

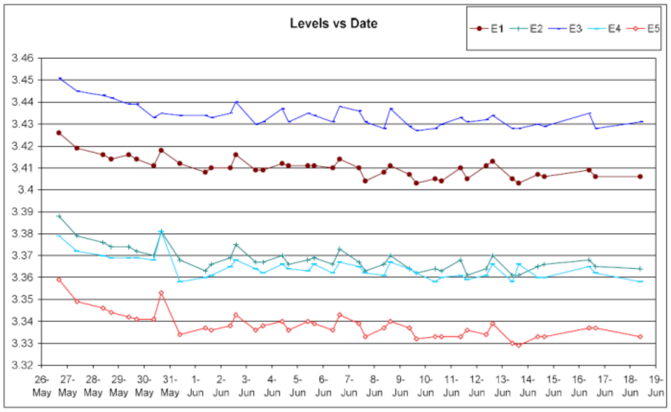
Test records are presented in Figure 10.











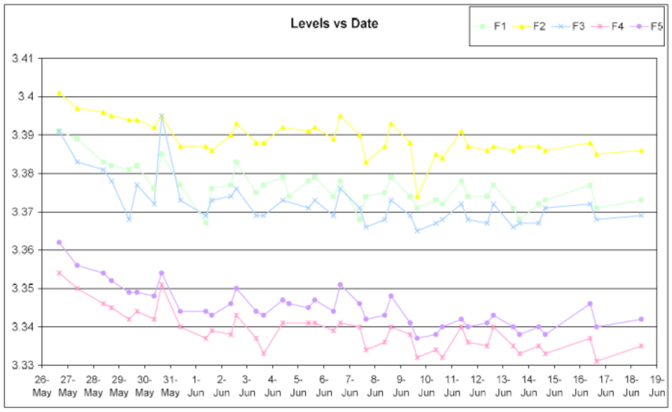


Fig 10.Test results

Analysis

The main objective from the geotechnical analysis is to validate the theoretical design for the stone columns in terms of settlement values and rate of settlement under the design loads. Asaoka's method had been used to evaluate the anticipated final settlement and the equivalent coefficient of consolidations based on the results of the test. Using Asaoka's method, the anticipated final settlement and the equivalent coefficient of consolidations at location of each block have been calculated. These calculations showed that the anticipated final settlement for the clay-stone columns system is 40 mm which is lower than the theoretical value settlement for the embankment for the case of long term condition (100 mm). In addition, that calculated equivalent value for the coefficient of consolidations is 3.26 m2/day. This is much higher than the original value for vertical coefficient of consolidation. This reflects the rule of the stone columns in accelerating the settlement of the soft clay layer. The load-settlement curves presented in figure 10 show that the curves became almost flat for the last period of loading. This means that the rate of settlement is almost zero and an average degree of consolidation of 90% had been reached.

CONCLUSION

Based on the analysis and testing of stone columns the following is concluded:

1- Stone columns have been proven to be an effective solution to reduce both long term and short term settlement.
2- A numerical model was generated to model stone columns under an embankment with steel pipes.
3- Results where validated by conducting a full scale load test on a group of stone columns.
4- A method is presented to conduct load test on stone columns offshore.
5- Full scale load test indicated lower values than what was concluded from the numerical analysis.

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