Visakhapatnam Chapter

Proceedings of Indian Geotechnical Conference 2020
December 17-19, 2020, Andhra University, Visakhapatnam

Study of Maliya Marine Clay for a Highway Embankment

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Abstract. This paper presents study of the soft to very soft marine clay deposit, very thick and having challenging geotechnical engineering properties for a road embankment over it. These include a very low shear strength, high compressibility, low permeability, sensitivity, etc. It is the Maliya marine clay deposit in tidal swamp at Little Rann of Kachchh, an extremely flat coastal alluvium plain. This clay deposit is of 6.5 km width and up to 15m thickness. Its engineering properties are determined through the elaborate laboratory testing program comprising of consolidated undrained CU circular direct shear box tests, vane shear tests, consolidation tests, unconfined compression tests - all these tests on the undisturbed samples obtained at every 1.5m from the borehole by the author up to 12m depth. The field investigation consists of undisturbed sampling. The results of all these tests are presented in this paper for shear strength, strength v/s strain characteristics, sensitivity, compressibility, consolidation characteristics c_v. The soil is mostly of CH group. Field moisture content exceeds liquid limit [60-70%]. Liquidity Index is compared. The values of the ratio of undrained shear strength to effective overburden pressure S_U/p [$S_{u/P}$], v/s plasticity index are determined and discussed. The top portion of this deposit, up to 2.4 m depth and relatively stiff. Using $\phi=0$ analysis, stability analysis of a typical road embankment having 12m top width and 7.5m height and with berms and vertical sand drains is done incorporating the strength at 3% strain for the compacted embankment.

Keywords: Soft marine clay deposit, undisturbed sampling laboratory testing, highway embankment, sand drains.

1 Introduction

An alignment of a highway embankment linking Ahmedabad to Kandla port passes through the Little Rann of Kachchh, where it encounters a 6.5km wide belt of marine clay deposit of thickness up to 15m+. The Rann of Kachchh is an extremely flat, coastal alluvium plain - originating from Gulf of Kachchh it runs WSW-ENE and then S-N for 96 km with varying widths of 8 to 50 km. The Rann of Kachchh was formerly a sea basin. Silting, upheaving and man- made & natural bunds, turning and thinning away of rivers created a Rann.

Along the highway embankment the Rann constitutes a tidal swamp, a soft marshy ground, and is very flat with ground level at almost 2.4m R.L., and is within the reach of daily sea tides of 2.8m R.L. The high tide level of the year is 3.3m R.L. and the maximum flood level is at 5.1m R.L. The maximum height of the highway embankment is kept at 6.9 m at an approach of the bridge over an intruding creek. It is 15m+ thick, normally consolidated, soft [0.25 kgf/sq. cm.< q_u <0.5 kgf/ sq. cm.], to very soft [q_u < 0.25 kgf/sq.cm, ~ 25kPa]. A man can hardly walk over it. It is of low shear strength, low permeability and high compressibility with $0.6 < C_c < 1.05$. This paper describes an engineering design for a typical embankment considering 7.5m height and 12m top width on this marine clay deposit incorporating sand-drains and berms.

1.1 Site and soil profile

The nearest villages to this area are Maliya in the south, 40km from Morbi, and village Surajbari in the north. The thickness of the deposit varies from 3m at chainage 4800m (from Maliya side) to 15.5m+ at chainage 10800m. Then onwards it reduces to 3m thickness at chainage 13995m. With facilities from Gujarat Engineering Research Institute, the author made one bore hole up to 12m depth below G.L. at chainage 10500m. The upper portion of the deposit is relatively stiff and penetrates below G.L. for a depth up to 2.8 m.

1.2 Properties of Maliya marine clay deposit

Presence of sufficient quantity of electrolyte (salt) in sea water, gives rise to edge to face structure and flocculation during sedimentation process of fine-grained soil. Actually, marine clay deposits are arranged in a more random three-dimensional orientation [1]. This concept was substantiated by [2], through electron micrography. In addition, slowly increasing geological loading simply pushes particle edges to other particle faces and bonds particles in a natural space frame [3]. This process allows slowly increasing load to be carried without substantial reduction of voids ratio or water content of sediments below.

Properties of Maliya Clay in brief. Marine clays are of high moisture content of 80% or more and near to or above liquid limit, except for upper relatively stiff crust portion with 40 to 50%.

Liquid limit w_L	Plastic Limit w_P	Plasticity Index I_P	Liquidity Index I_L
50-70%	22-35%	25-35%	0.4-1.7

Mechanical Analysis shows that silt size particles are predominant. Silt size particles: 60-75%, clay size particles: 22-35%, sand size particles: 2-13%

Accordingly, Maliya marine clays are inorganic clay of high compressibility. They mostly fall in CH group on Casagrande's Chart. A few fall in CI and MH group. As per PRA classification they fall on A-7 and A7-6 group. Standard Proctor maximum dry density MDD is 16.35 kN/m^3 at Optimum Moisture content OMC of 18%, Compression index C_c= 0.6 to 1.05. Coefficient of consolidation is from (50 to 100) x 10^{-5} cm^2/sec. From circular box shear strength test, cohesion intercept, C_{cu} = 0, and an angle of shearing resistance, ϕ_{cu}=18°. Average shear strength, as determined by laboratory vane shear tests and unconfined compression tests, all on undisturbed samples, is 16.6 kPa to 22.5 kPa.

Stiff crust. Hard surface layer overlying softer soils is generally the case with soft marshy ground [4]. According to them it is formed by the effect of desiccation at upper surface that is not permanently flooded. [5] state it to be the result of weathering effect. Maliya marine clay deposit is in a marshy swamp, under effect of daily tides in a semi-arid region. Accordingly, desiccation has caused an upper stiff crust of thickness from 1.2-3m.

Leaching. Leaching reduces salt content from pore water. As per [6], the reduction of salt content largely reduces remolded strength. [7] states that it reduces shear strength of both the remolded samples, and undisturbed samples, and holds out the reduced strength due to leaching to be the cause of natural or otherwise undisturbed Norwegian clay slopes. Maliya marine clays do not seem to have been subjected to leaching.

Sensitivity. Sensitivity is the loss of strength on remolding at an unaltered water content. It is more pronounced in marine clays. It is measured by the ratio of undisturbed strength to remolded strength at constant water content. In very soft samples, the strengths may be determined by vane shear tests. Current concepts attribute loss of strength partly to orientation of particles and partly due to reduction in inter-particle forces. Laboratory vane shear tests on undisturbed samples of Maliya marine clay were performed. Sensitivity values were seen to be in range 3-8, in one case 10.

Activity. Activity is defined by [8] as the ratio of plasticity index to clay fraction. Value for Maliya marine clay varies from 0.94-1.21, and it is termed as normal clay.

Activity value	<0.75 inactive	0.75-1.4 normal	>1.4 active

$S_{u/P}$, Ratio of undrained shear strength to effective overburden pressure. It was only after 1945 with refined sampling techniques and vane shear tests, it was known that for normally consolidated soft deposits also S_u (undrained shear strength) did increase with depth and that the ratio $S_{u/p}$ was approximately constant where p is an effective overburden pressure. [7] reported it to be increasing with plasticity index. [8] gave relation as

$$S_{u/p} = 0.11 + 0.0037 I_p$$

[9] state that the above statistical relation so far has been found applicable over a wide range types of sedimented clays. For the normally consolidated Maliya marine clay, the value of $S_{u/P}$ ranges from 0.25 to 0.28 while the plasticity index is in range of 25-35%.

2 Shear Strength Tests

The undisturbed samples were collected in sampling tubes, one tube per 1.5 m depth up to 12 m below ground. The tubes were carefully sealed and brought to the laboratory.

2.1 Circular box direct shear CU consolidated test

Consolidated, undrained circular box direct shear tests were performed using undisturbed samples. In order totally avoid the observed compression disturbance on pushing sample out of sampling tube, the tubes were cut into pieces of the size just sufficient to get required best samples of 2 cm height. The circular shape of the box mold ensured that the disturbance, if any, to the sample while transferring to the mould did not occur. Also the change in area of the contact area of the enforced plan of shear up to failure would, at all strains, be less in case of circular mold compared to rectangular mold.

Procedure. The volume of samples was 57.14 cm^3. Load was applied in increments of 25lb, varying from 0 to 125lb. These correspond to increments of 5.65psi, or 39kPa, normal stress σ being (39, 78, 117, 156, 195) kPa.

The time required for 100% consolidation:

$T = T_v [H^2/c_v] = 1.3[(1)^2/(50x10^{-5})] = 43.3$ minutes

The samples were consolidated under normal load for 45 minutes and then sheared at fast rate of 0.1275cm/min, $dL/L_0 = 2.105$, say 2%. The vertical dial gauge readings were recorded for obtaining the changes in the volume in the sample during the consolidation and also the shearing process. Bulk unit and dry unit weight at the beginning of the test for each sample and the moisture content at failure for a few samples were determined.

Analysis of test results and discussion. Stress v/s horizontal strain curves are presented here for the undisturbed sample designated Maliya/1256 obtained from depth 10.5m below G.L. The curve with ‘yield point’ is observed. [14] demonstrated that ‘not only does the cohesion component develop very rapidly, but in some clays the cohesion reaches maximum and then decreases measurably at decreasing rate while friction tends to steadily increase’. [16] states that ‘being destroyed’ cohesion is not being fully compensated by friction, the cohesion passes its peak and decreases more rapidly. Pronounced peaking effect exhibits a ‘noticeable yield point’ in the stress-strain curve.

Lines for shear strength τ v/s normal stress σ pass through origin for normally consolidated clays when samples are tested in the laboratory at the normal pressure σ greater than the existing effective overburden pressure in the field. But in the case of the same samples when tested in the laboratory under normal stress which is less than the corresponding effective overburden , cohesion intercept appears on y-axis and it is found to be proportional to existing effective over burden pressure to which the sample in the field has been consolidated. In short, as per [16],

$\tau = \sigma'_p \tan \phi'_c + \sigma'_f \tan \phi'_r$

Now for normally consolidated clays, $\sigma'_p = \sigma'_f$, hence

$\tau = \sigma'_f (\tan \phi'_c + \tan \phi'_r)$

$= \sigma'_f \tan \phi'_s$.

And strength passes through for normally consolidated clays, fig 7. In this C_{cu}=0 and ϕ_{cu}= avg18.5°. In other cases, corresponding these average values are: C_{cu}=21 kPa and ϕ_c=4°.

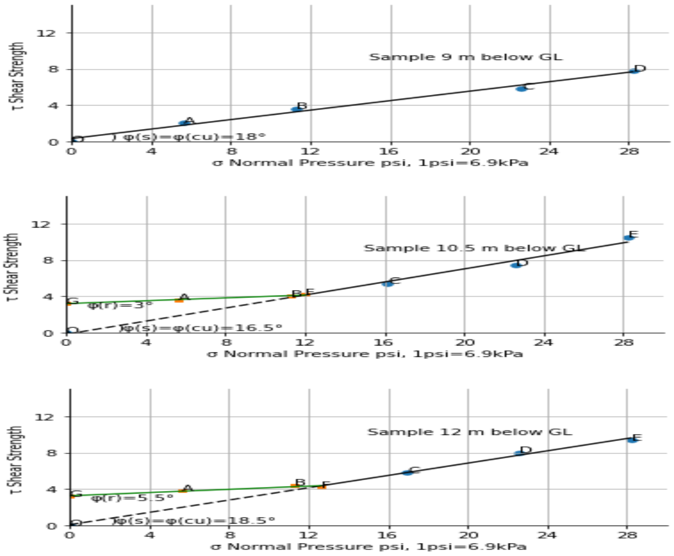


Fig.1. Box Shear Test

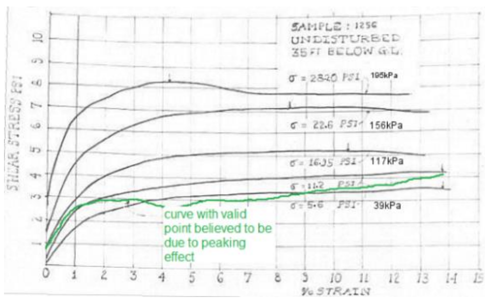


Fig. 2. Peaking Effect (Box Shear Test)

2.2 Vane shear test

[8] concluded that ‘the strength of normal consolidated clays does increase with depth.’ This finding became possible due to refined sampling techniques and vane shear tests. According to [10] and [11], there is an increase in shear strength values obtained at higher rate of rotational speed. To find the shear strength of Maliya marine clay, the samples were subjected to rotational speed of (5-7) ° per minute, the minimum rotation rate that can be uniformly applied by hands in the laboratory. Shear strength develops at small strains and assuming constant shearing resistance over the plane with only one circular surface (bottom) together with cylindrical surface total torque

$$T = T_1 + T_2 = 2\pi r^2[L + r/3]\ \tau.$$

Also torque T = θ x (spring factor)/180 where θ is in degrees.
Substituting values of the length and radius of the vanes and of the spring factor:
T = 0.0795θ psi = 0.548θ kPa.

Table 1. Avg. Shear strength of soil at various depths

Depth in below G.L. (m)	Avg Shear Strength
0	41.4
5	68.3
10	90.1
15	14.5
27	15.8
30	17.25
35	17.9
40	20.7

2.3 Unconfined compression tests

In place shearing resistance of foundation soil is found by an unconfined compression test on specimens. The shearing resistance is equal to half the unconfined compression strength q_u. In case of unconsolidated undrained tri-axial test, the strength is found to be half of the maximum deviatoric stress. [12] states that it can be found out from a direct shear test in which no consolidation is allowed under a normal load and is sheared immediately after putting it in position. Unconfined compression tests were carried out on undisturbed specimens collected from 4.5m and 9m depth below G.L. Results are shown in Figure 6. The strength values are (1.937psi) 13.35kPa and (2.073 psi) 13.99 kPa respectively. They are comparable to the values (2.05psi) 14.1 kPa and (2.5psi) 17.22 kPa as obtained from the laboratory vane shear tests on the corresponding undisturbed samples. The strength values obtained from laboratory vane shear tests are utilized for stability analysis of the embankment and the strength value adopted is 12 kPa.

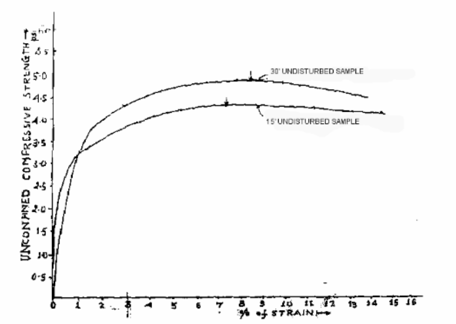
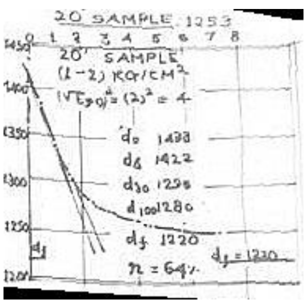


Fig.3. Unconfined compression test

3 Consolidation tests

Consolidation tests were performed on undisturbed samples from every 1.5m depth up to 12m below G.L. Diameter of the mold was 60.325mm and consolidation pressure ranged from 1/16-8kgf/cm^2 (96.26-800kPa). The load increment was 1 and the time duration for action of each of these was 24hrs. Dial gauge reading were recorded to make a plot of these readings vs. square root of time t. One such typical plot for pressure range from 107-214kPa is shown in Figure 5. Coefficient of consolidation c_v, compression index C_c, and consolidation ratio r are calculated. Casagrande's graphical method is used to find pre-consolidation pressure for each sample. These graphical values are usually found to be smaller than actual ones. One reason for this discrepancy might be that in the field the pressure increment ratio is smaller, and also that the duration increases the value of the pre-consolidation load.

The initial void ratio varies from 1-2.4, c_v from (22-160) x10^{-5}cm^2/sec and compression index C_c from 0.66-1.05. It is observed that when values of c_v are plotted on log scale against the depth of the samples, a straight line is seen.



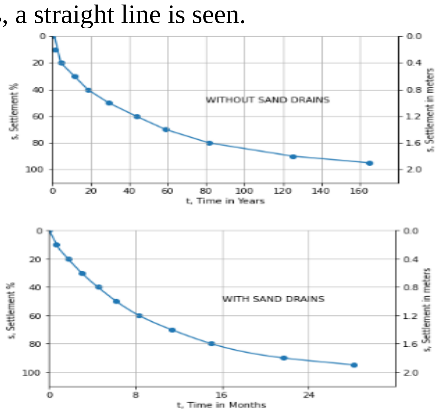


Fig. 4. Consolidation test

Fig.5. Load vs Settlement Plot

4 Embankment and Stability Analysis

In the case under consideration, the embankment is to rest on soft to very soft normally consolidated thick marine clay deposit of low permeability, in marshy swamp at tidal alluvial flat Rann of Kachchh. The ground level is 2.4 R.L., and maximum flood level is 5.1 R.L. The embankment has to be built only in one season at least above this flood level for its safety. This means rapid construction over saturated clay sub-soil without any chance for the dissipation of pore pressures. For such situation the end of construction condition is well accepted to be the most critical, as in due course of time there will be an increase in shear strength consequent of consolidation. As a result, the factor of safety will increase with time. The shear strength is determined by unconfined compression test and it is equal to half of the unconfined compressive strength, $\tau = q_u/2$. This strength is used in stability analysis of natural saturated slopes with their bases. The analysis is called ϕ_u=o analysis. In this case lesser value obtained using laboratory vane shear test is adopted for stability analysis purpose.

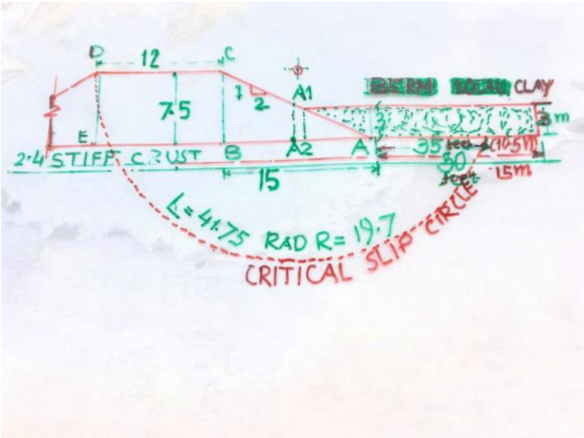


Fig.6. Cross section of the Embankment

4.1 ϕ_u= 0 analysis

[13], in analyzing the stability of natural slopes conclude that $\phi_u = 0$ analysis applies only for end of construction type condition. [9] state that the results of an unconfined compression test on a perfectly undisturbed sample are approximately same as those of a consolidated undrained test performed under same confining pressure p_3 where $p_3 = (0.7 \text{ to } 0.9)\ p_v$.

4.2 Stability of slopes of embankment

The upper portion, a relatively stiff crust, of 15m+ soft marine clay deposit varies in thickness from 1.8m-2.4m. Its shear parameters in molded state are:

1) At standard Proctor density MDD 1.676 g/cm^3, 16.35 kN/m^3., OMC 18.7%: C_{CU}=3psi (20.7kPa), ϕ_{cu}=23°
2) At Proctor density and OMC+3%: C_{CU}=1.8psi (12.24kPa)
   Shear strength develops at about 7-8%. Minimum shear strength = $q_u/2$ =7psi (48.3 kPa) at 7% strain. This strength is considerably higher than the undisturbed strength of underlying soft foundation material.

4.3 Stability analysis

The embankment with 7.5m height and 12m top width is examined for stability of slopes of 1.5:1 and 2:1. 12 slip circles for former, and 8 slip circles for the latter are considered. Shear strength required for equilibrium is 22.7kPa and 18.9kPa respectively. It is known that it is the base in such cases which governs the design of the embankment slopes.

4.4 Stability of the base

Mode of failure. Base failure may occur in several different situations. The fill may bodily sink in the supporting soil – failure by sinking; failure by spreading, failure by piping [4]. For the fills on soft homogeneous clay they state that "if the surface of the clay is very close to the base of the fill, the thickness of the clay stratum is at least half as great as the base width of the fill and the stratum is fairly homogeneous. The failure of a fill on such a base has a general character of a base failure along a midpoint circle. The average undrained resistance for the base be determined". For the real fill having cohesion or only friction, equivalent cohesion (ϕ=0) is used. Maliya marine clay base is homogeneous, close to the base of the fill, and it has the thickness nearly half of the base width of the embankment with 2:1 slope. 15-16m thickness nearly meets these criteria. In Figure 4, we present the midpoint circle (ϕ_u= 0) analysis. In all 5 circles were examined.

Shearing resistance by embankment and by base. The material utilized for the examined embankment is considered to be a local material at MDD with omc=3%. Its stress strain curves are a bit flatter and its peak strength is developed at 7% strain as compared to 2-4% strain in undisturbed soft clay. To avoid plastic state in foundation material and consequent heaving, and extreme subsidence, it will not be desirable to strain material in the foundation base, and hence in the embankment beyond 3%strain. The contribution of the embankment towards shearing resistance at this strain is 7.1psi (49kPa) say 48 kPa against average ultimate strength of 15psi (103.4kPa). Average shearing resistance by the foundation is taken as 12kPa.

Critical midpoint circle analysis. Two slopes were examined with values 2:1 and 3.6:1. The analysis for slope 2:1 is presented in **Error! Reference source not found.**. Factor of safety is calculated and it is 0.916 which is not ok. [4] state that “it is commonly required that the factor of safety with respect to base failure should be at least 1.5. Considering the unavoidable errors in estimating the average shearing resistance of the clay, this factor is very low. Nevertheless, in order to satisfy the requirement, high fills on soft clay must be provided with very gentle slopes. Hence, if the fill is very long, it may be economical to further reduce the factor of safety still further, to 1.2 or 1.1” and then monitor to detect any impending failure, if any, during construction.

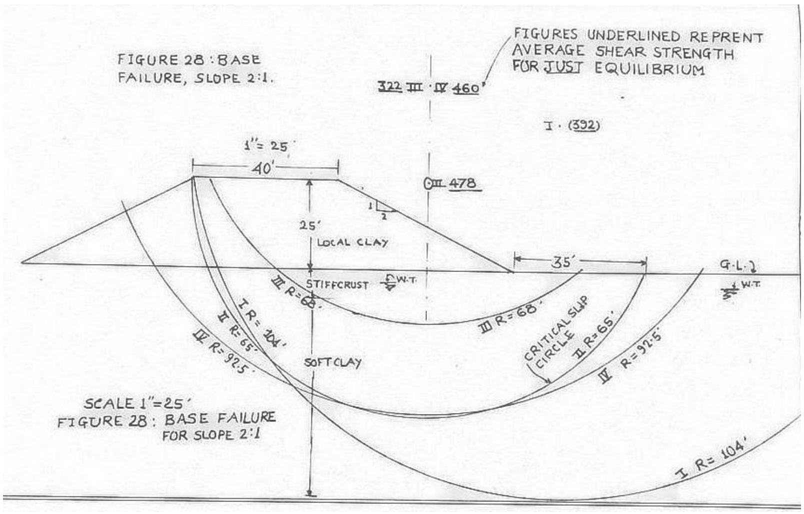


Fig. 7. Analysis for Mid point circle base analysis

In this case, it is noticed noted the shear resistance has, fairly well, been arrived at. The berms, 3 m thick, made from local clay and compacted 100 lb. per cft=15.72 kN/ m^3, are incorporated in design of an embankment. Factor of safety respectively with 10.5 m, 15 m, and 24 m is 1.28, 1.47 and 1.99. The embankment with 15 m berm satisfies the safety criteria and hence it is ok.

FOS CALCULATION IN CASE OF10.5 BERM:

12.5 m x 7.5 m x1 m x16.35kN per cu m x13,5 m = 19865.25 kN-m….[1]

(1/2) x7.5 m x3.75 m x 1 m x16.35 Kn per cu m x 5 m =1149.8 kN-m ….[2] 7.5 m x3.75 m x 1 m x 16 35 m kN per cu m x3.75 =172441 kN-m …..[3] total of [1] +[2]+ [3] =22739.4 = $M_{D= \text{Driving}}$ moment … [4]

Resisting moment: (41.576 m x 1 m x 12 kPa +4.876 m x 1 x 48 kPa +7.62 m x 1 m x 48 kPa) = 1100.7 kN . 1100.7 KkN x 19.7 m lever arm = 21683.79 kN-m … [5]

(1/2) 7.25 x 3.75 16.35 kN per cu m x2.5 m lever arm = 574.8 kN-m … [6]

[5] + [6] = 22258.6kN-m … [7]

Deduct for 1.5 m cracking 1.5 m x 1 m x 48kPa x 19.7 m = 1418.4 kN-m … [8]

[7] – [8] =20840.2 kN-m …… [9]

Now add for berm of 10.5 m x 3 m:

Small triangle (1/2) x 6 m x 7.5 m x 15.72 kN per cu m x 5.5m lever = 1945..35 kN-m …. [10]

[9] + [10] = 22785.5kN-m = total resisting moment…….[11]

10.5 m x 3 m x1 m x 15.72 kN per cu m x 12.75 lever arm 6313.5 kN-m [12]

Total resisting moment M_R = [11] + [12] =29099.0 Kn-m … [13]

Fos = [13] / [4] =1.279 say 1.28

Settlement prediction

Findings from consolidated tests on undisturbed samples are used. Charts are used to find vertical pressure at any depth. Vertical settlements are then computed using equation:

$S = H(e_1-e_2)/(1+e_1)$

in usual notation. The settlement is calculated for only center line of the embankment. The coefficient of consolidation c_v is taken as 50 x $10^{-5}cm^2/sec$ and height as 15m. With time in seconds given by:

$$t = TH^2/c_v$$

it works out to 147T years, where T is Time Factor whose numerical value corresponds to U%, degree of consolidation S/S_{final}. Total settlement is 1.95m (6.49 ft). For 50% settlement (0.925m) the time required is 29.4 years in case of no sand drains.

4.5 Sand drains

Let us consider vertical sand drains of diameter d=0.3m and at triangular spacing 2.8m as show in Figure 2. The equivalent circular diameter D is 2.85 x 1.05 = 3m. Hence the system can be thought of as a cylinder of diameter D'=3m with an internal drain of diameter d=0.3m. As per [13] time factor $T = c_v t/H^2 = c_v t/D'^2$ which is 0.132. For comparison of time settlement of the embankment foundation, with parameters as given above and writing

$T = c_{vr} t/(D')^2$

With $c_{vr} = 1.5c_v$, we arrive at t=47T. For 50% settlement, the time t required is 47 x 0.132 = 6.2 months as shown in Figure 3. This is a considerable gain, 57X, improvement through the use of sand drains. Sand blanket of 0.6 m below the embankment is incorporated to discharge the drained water.

Note: This work was submitted as a dissertation to the M.S. University of Baroda by the author in partial fulfillment of the requirements for the degree of Masters of Engineering in 1967.

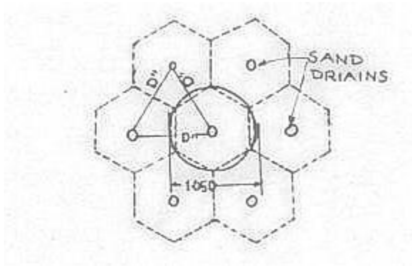


Fig.8. Pattern of Sand Drains

Acknowledgments

The author is highly indebted to late H. P. Oza, M.Sc. (Bom) S.M. (M.I.T), A. I.MAR. E. (Lon.), M.I.E.(India), DIRECTOR, Engineering Research Institute, Baroda, now Gujarat Engineering Research Institute (GERI) Vadodara, the Guide for the dissertation, for his esteemed guidance, invaluable help and continuous encouragement throughout the work. The author is much thankful to the staff of GERI for their help at the time of collecting undisturbed samples.
The author also owes to Late Prof. R. S. Dighe, Head of Applied Mechanics and Structural Engineering Department, for his very kind encouragement throughout the work. He was the basic incentive for the work. The author is also thankful to late Prof. N. Y. Hiryur. The author expresses his sincere thanks to Shri A.C. Shah, ARO (Assistant Research Officer), for his help and suggestions from time to time.

5 Conclusions

1. The Rann of Kachchh was formerly a sea basin. The huge amount of fine-grained soil brought by rivers filled it up. As a result of filling up due to silting, partially upheaving of the sea floor, man-made mounds and moving away of rivers a sea basin got convertd into the Rann.
2. The Maliya Marine clays are of marine origin at the gulf end in a Little Rann of Kachchh. They mostly are of CH group. A few fall in CI and MH group. The thickness of deposit varies from 3 m to 15 m plus. Its moisture content is at 80%, greater than liquid limit which is from 50 to 70%. The laboratory vane shear test gave the sensitivity value varying from 2-8.

3. From box shear test, it can be inferred that the clay is normally consolidated. The yield point is observed in stress-strain curve. This may be due to pronounced "peaking effect".
4. In the box shear test, shear strength line passes through origin when samples are tested under normal load intensities greater than in-situ effective burden the average angle of shearing resistance, ϕ_{cu} 18.2. But it gives cohesion intercept when tested under normal load intensities smaller than in-situ effective burden. In that case $C_{cu} =$ 21kPa average and $\phi_r = 4.3°$.
5. "Yield point" is observed in stress- strain curve during box shear test. This is explained by the phenomena of pronounced peaking effect.
6. Undrained shear strength is seen to increase with depth of a deposit. It increases from (2 psi=) 14 kPa to(3 psi =) 21 Kpa. For a depth range of 4.5 m and 12 m.
7. The ratio of undrained shear strength $S_{u,}$ to p, an effective over burden pressure is constant over the depth. Its value is found to be average 0.27 as determined by laboratory vane shear tests.
8. The upper portion of the deposit is relatively stiff as a result of desiccation.
9. It is seen that it is the base that governs the design in this case. Mid-point circle failure mode has been examined for the embankment with berms and 1:2 slope.

   The use of sand drains reduces the time from 29 years to 6.2 months for 50% primary settlement which is calculated to be (3.25 ft.) nearly 1 m

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