Int. J. of GEOMATE, Sept., 2013, Vol. 5, No. 1 (Sl. No. 9), pp. 653-659
Geotec., Const. Mat. &Env., ISSN:2186-2982(P), 2186-2990(O), Japan

LOWLAND ENVIRONMENTAL GEOTECHNOLOGY OF SEISMOSEDIMENTS OF KANDLA PORT ININDIA

J.Rajaraman[1] and K.Thiruvekatasamy[2]

Department Of Harbour And Ocean Engineering, Amet University, Chennai, India

ABSTRACT: The Gujarat coast in India is vulnerable to various natural disasters such as earthquakes, cyclones, tsunamis and agricultural droughts. The nearby main town on main land is Gandhidham.Kandla is located at a distance of about 4 km from Gandhidham. A 20 m wide road connects Gandhidham and Kandla. Geographically, Kandla could not be considered as a port because, there is water on almost 340 ^{0} of this port. Kandla is almost an island. There is only one escape route via the bridge. In case the bridge gets damaged due to natural disasters, then there is no alternate route for rescue, evacuation and escape. At present huge amount of dredging is regularly being undertaken at the Kandla port. On an average about half million cubic meters of dredged materials is excavated, carried and is dumped in deep Sea. In this paper, an integrated approach to the clay behavior (including seismosediments) is considered mainly to relate cohesion and friction under different environmental conditions in lowlands, to shear strength by different types of clay.

Keywords: *Lowlands,Seismosediments, Lateral Earth Pressure, Clay Minerals*

1. INTRODUCTION

In flood affected lowland areas the loss of stability due to drastic variation in clay behavior and lateral pressure is a problem that needs separate treatment. The accumulated sediments due to huge amount of dredging adds more complications to the already existing problems. On an average about half million cubic meters of dredged materials is excavated, carried and is dumped in deep Sea. The heterogeneity of the sediments is due to many factors in which seismosediments are one of the important factors.

1.1Seismosedimentation And Seismosediments

Geological materials which are crushed, fractured, loosened or displaced directly or indirectly as a result of an earthquake induced processes such as landslides, landslips, slumps, mudflows, avalanches, liquefaction, rockfalls, rockslides, tsunamis and soils moved under gravity in the coastal areas or in the vicinity of the river are defined as seismosediments and the process is defined as seismosedimentation

1.2Textural Inversions

Textural inversions also cause mixing of different clay sediments to create more heterogeneity in soils. For example quartz arenites are more poorly sorted and many contain high percentage of sub angular and angular grains. Some quartz arenites exhibit textural inversions such as a combination of poor sorting and high rounding, a lack of correlation between roundness and size, such as small round grains and larger angular grains, or mixtures of rounded and angular grains within the same size fraction. These textural inversions probably result from mixing of grains from different sources, erosion of older sandstones, orenvironmental variables such as floods, tsunamis and wind transport of rounded grains into a quiet water environment. Angular grains may result also from diagenetic development of secondary overgrowth.

1.3Clay Minerals and Sediments

The dredged material in Kandla port consists of fine sandy clay or clayey sand. In these sediments mainly three types of clays are involved. They are Kaolinite, Illite and Montmorillonite. Due to the mixing factors discussed above the soil becomes highly heterogeneous. The Geotechnical properties vary from pocket to pocket in an erratic manner. Aseparate treatment of geotechnical methods for coastal lowlands is necessary.

2.PROPERTIES OF CLAYS

Plastic property of clay is due to adsorbed water that surrounds the clay particles. This property is affected by type of clay minerals and their proportional amounts in a soil. The ultimate effect is that different clay soils possess different liquid and plastic limits.

2.1 Activity of Clay

Skempton defined a quantity called Activity i.e. the slope of the line correlating PI (Plasticity index) and percent finer than 2μ therefore
Activity A (as shown in Table 1).

$$A = \frac{PI}{percentage\ of\ claysizefraction\ by\ wt}$$

Seed and others studied the plastic property of several artificially prepared mixtures of Bentonite and Kaolinite. The results are shown in figure 1

Table1 Activity (A) of Clay Minerals.

Mineral	Activity(A)
Smectites	
(Montmorillonite)	1-7
Illite	0.5-1
Kaolinite	0.5

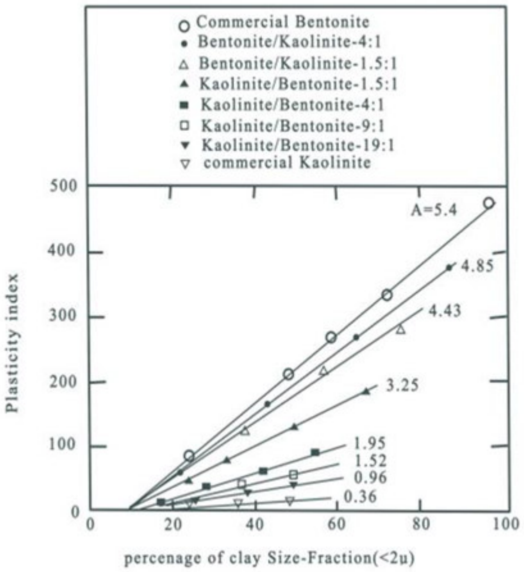

Fig.1 Relationship between plasticity index and clay size fraction by weight for kaolinite/ bentonite clay mixtures [1]

2.2 Clay Minerals and Angle of Internal Friction φ

For normally consolidated clays cohesion C = 0. The equation of the Mohr – Coulomb failure envelope is given by S = σ' tan φ. Where φ is angle of internal friction and S is shear strength.Then

$$\boldsymbol{sin\varphi = \frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'}} \quad (1)$$

The modified forms of Mohr's failure envelope for pure clay minerals are shown in the figure 2. In a consolidated drained test the total stress is equal to the effective stress, since the excess pore water pressure is zero. At failure, the maximum effective principal stress is $\sigma_1' = \sigma_1 = \sigma_3 + \Delta\sigma_f$, where $\Delta\sigma_f$, isthe deviator stress at failure. The minimumeffective principalstress is $\sigma'_3 = \sigma_3$The modified form of Mohr's failure envelope of pure clay minerals and quartz with typical values of φ is grouped as inTable 2.

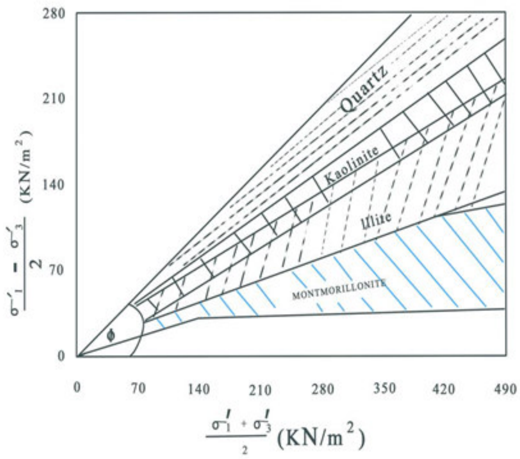

Fig 2Friction angle variation with minerals/quartz [2]

Table2 Variation of**φ**with types of clay minerals/quartz

Sl No	Minerals	Types	Values
1.	ClayMinerals		
	Montmorillonite	Type1	25-30
	Illite	Type2	30-35
	Kaolinite	Type3	35-40
2.	Quartz	Type4	40-45

2.3 The Random Nature of Clay

Kaolinite and Illite have nonexpansive lattices whilst that of Montmorillonite is expansive. There is no particular value of plastic limit that is characteristic of an individual clay mineral type. Similarly there is no single liquid limit which ischaracteristic of a particular clay mineral. The liquid limits for Illites fall in the range of 60 – 90 % whilst those forKaolites vary from 30 -75%. The presence of 10% Montmorillonite in an Illitic or Kaolinitic clay can cause a substantial increase in their liquid limits. The plasticity indices of Na and Li Montmorillonite clays have exceedingly high values, ranging between 300 to 600 %. TheMohr-Coulomb envelope only indicates an ultimate shear strength value. It is the final compromised state of equilibrium of many random variables.

3. STRESS PATH AND LATERAL EARTH PRESSURE

Consider a normally consolidated clay specimen subjected to a consolidated drained triaxial test Fig.3 (a). At any time during the test, the stress condition in the specimen can be represented by Mohr's circle as shown in Fig.3(b).

In a drained test, total stress is equal to effective stress so $\sigma_3 = \sigma'$ (minor principle stress).
$\sigma_1 = \sigma_3 + \Delta\sigma = \sigma_1'$ (major principal stress).
At failure, Mohr's circle will touch a line (Mohr-Coulomb failure envelope). This makes an angle φ with the normal stress axis.

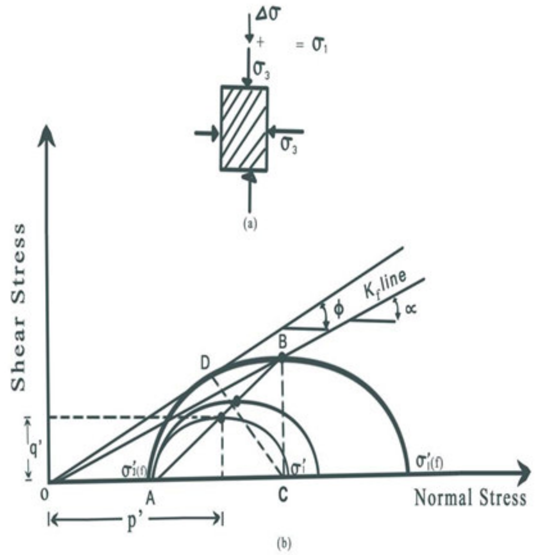

Fig.3Definition of stresspath [3]

Now considering the effective normal and shear stresses on a plane making an angle of 45^{0}with the major principal plane:

Effective normal stress,

$$p' = \frac{\sigma_1' + \sigma_3'}{2} \quad (2)$$

Shear stress,

$$q' = \frac{\sigma_1' - \sigma_3'}{2} \quad (3)$$

The points on Mohr's circle having coordinates *p′ and q′* are shown in Fig.3b. If the points with *p′ and q′* coordinates of all the Mohr's circles are joined, will result in the line AB. The line is called a stress path. The straight line joining the origin and point B is defined here as the $K_f$line. The $K_f$line makes
an angle ∝ with the normal stress axis. Now

$$\tan \propto = \frac{BC}{0C} = \frac{\left(\sigma'_{1(f)} - \sigma'_{3\,(f)}\right)/2}{\left(\sigma'_{1(f)} + \sigma'_{3\,(f)}\right)/2} \quad (4)$$

Where $\sigma'_{1(f)}$ and $\sigma'_{3(f)}$ are the effective major and minor principal stresses at failure.
Similarly,

$$\sin \varphi = \frac{DC}{0C} = \frac{\left(\sigma'_{1(f)} - \sigma'_{3\,(f)}\right)/2}{\left(\sigma'_{1(f)} + \sigma'_{3\,(f)}\right)/2} \quad (5)$$

From equations (4) and (5) ,$\tan \propto = \sin\varphi$ (6)

4. SETTLEMENT CALCULATION USING STRESS PATH

From Lambe's stress path the followingconclusions are made
1. The stress paths for a given normally consolidated clay are geometrically similar, and
2. When the points representing equal axial strain say (ϵ_1) are joined, will be approximate straight lines passing through the origin. Now considering a soil specimen subjected to one dimensional consolidation type of loading we can write

$\sigma'_3 = K_0\, \sigma_1'$ (7)

This equation (7) is very important because it gives a relationship between σ'_1, σ'_3 and the coefficient of earth pressure at rest (K_0). According to Jaky, K_0 = 1- sin φ. This equation connects various types of clay and their respectiveangle of internal friction φ. From the following, Fig. 4 in the Mohr's circle the coordinates of point E is given by

$$q' = \frac{\sigma_1' - \sigma_3'}{2} = \frac{\sigma_1'\ (1 - K_0)}{2} \quad (8)$$

$$p' = \frac{\sigma_1' + \sigma_3'}{2} = \frac{\sigma_1'\ (1 + K_0)}{2} \quad (9)$$

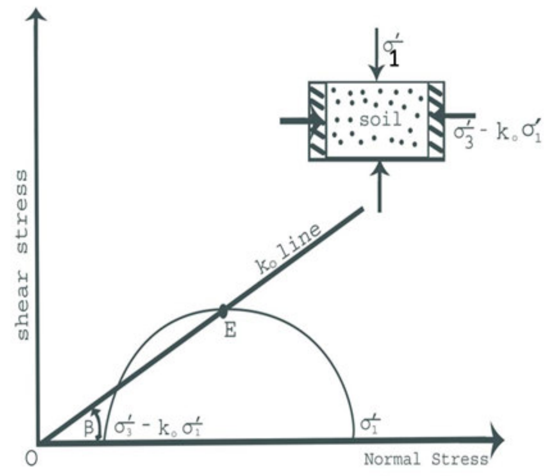

Fig. 4 Formation of the slope of k_0 line [3]

From the plot of p' versus q' with k_0 and $k_f$lines as shown in Fig. 5.
The angle,

$$\beta = \tan^{-1}\left(\frac{q'}{p'}\right) = \tan^{-1}\left(\frac{1-k_0}{1+k_0}\right) \quad (10)$$

Where β is the angle that the line OE (K_0 line) makes with the normal stress axis.

5. KANDLA PORT ENVIRONMENT

The Kandla Port is surrounded on all sides by water. There are two creeks viz Nakti creek and the Kandla creek surround the port and there is water on about 340 degrees. At the Kandla end of the bridge, the two creeks almost meet one another.

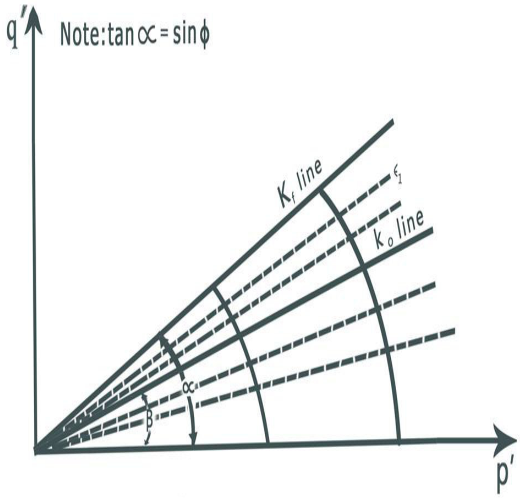

Fig. 5 Plot of p' versus q' with k_0 and k_f line.

The above-mentioned route is the only connecting link between Kandla and Gandhidham**.**

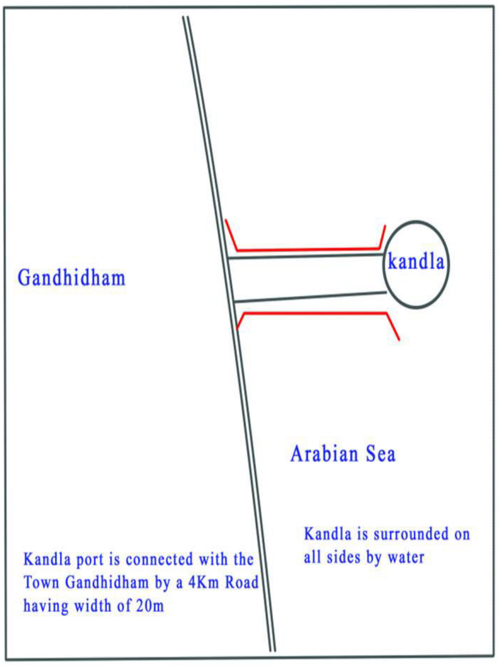

Fig.6 Kandla port environment[4]

Arun Bapat suggested for a construction of vertical evacuation facility (VEF). Instead of dumping into the deep sea the dredged material could be used to build a vertical evacuation facility. A vacant and large area of about 100 sq.km is available at Kandla. This land is in S.W. direction and south of Khari creek. This is very near the port. This land can be used for VEF. The facility will be a construction of small hill like structure with a shape of tapering conical or a flat top cone with one or two tops. For this purpose, retaining wall along the periphery of VEF will be required. A hillock of about 15 to 20 m high with a base of about 200 to 250 m diameter could accommodate about fifteen to twenty thousand people or more. The dredged material consists of fine sandy clay or clayey sand. This could be suitably compacted so that the structure will be in maximum stability conditions. It could have a wide ramp with gentle slope for rapid movement of large population. This ramp would be winding the small hillocks in serpentine way. Suitable shady trees could be planted at various terraces of the VEF so that it becomes stable. Construction of such VEF would help in saving several thousand lives in the case of tsunami attack at Kandla.

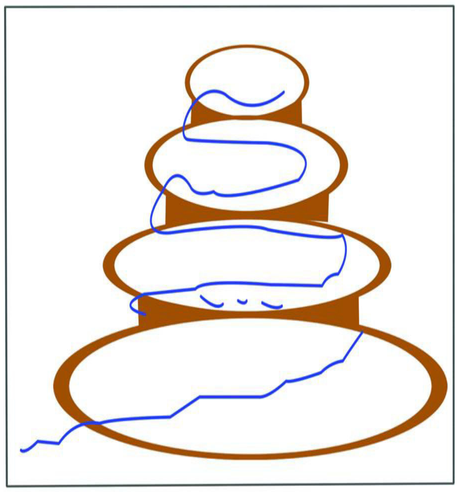

Fig.7 Sketch of VEF at Kandla[4]

According to the view of Arun Bapat the sketch of the VEF will appear as shown in the Fig.7

6. THE ENVIRONMENTAL GEOTECHNICAL PROBLEMS

The environmental Geotechnical problems are highly complex. These problems are associated with seismosediments, dredged materials, clay minerals. Basically the dredged soils will show different soil characteristics than the natural soils with natural compaction. Therefore a detailed micro-studies related to soil mechanics alone will yield good solutions. The settlement calculation will clearly indicate the complex nature of the problems.

6. 1 Settlement Calculation

An illustrative example is shown in Fig.8. Itrepresents the stress – strain contours for a given normally consolidated clay specimen obtained from an average depth of a clay layer. Taking compression index C_c = 0.25 and void ratio e_0 = o.9and the drained friction angle φ as $30^0$and from equation (10),

$$\boldsymbol{\beta = tan^{-1}\left(\frac{q'}{p'}\right) = tan^{-1}\left(\frac{1-k_0}{1+k_0}\right)}$$

And k_0= 1 – sin = 1 – sin 30^0 = 0.5

$$\boldsymbol{\beta = tan^{-1}\left(\frac{1-0.5}{1+0.5}\right) = 18.43^0}$$

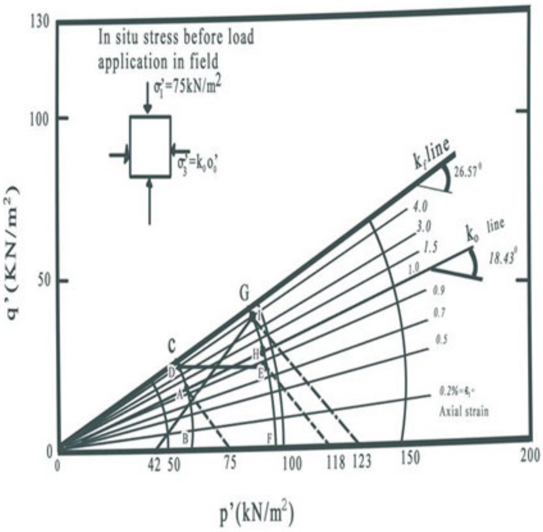

Fig.8 Use of stress path to calculate settlement [5]

Knowing the value of β the k_0 line is formed as shown in fig. 8.

Since tan∝ =sinφ, angle φ =30^0

tan∝ = 0.5. So ∝ = 26.57^0

The settlement is calculated for the clay layer from the following conditions :

1. Insitu average effective over burden pressure σ'_1 = 75kN / m2.
2. Total thickness of clay layer = H_t = 3m

Owing to the construction of a structure, the increase of the total major and minor principal stresses at an average depth are,

$\Delta\sigma_1$ = 40kN/m2

$\Delta\sigma_3$ = 25kN/m2

Assuming that the load is applied instantaneously. Insitu minor principal stress at rest pressure is σ_3 = σ'_3 = $k_0\sigma'_1$ =0.5(75) = 37.5kN/m2.

So before loading.

$$\boldsymbol{p' = \frac{\sigma_1' + \sigma_2'}{2} = \frac{75+37.5}{2} = 6.25\ kN/m^2}$$

$$\boldsymbol{q' = \frac{\sigma_1' - \sigma_2'}{2} = \frac{75-37.5}{2} = 18.75\ kN/m^2}$$

The stress conditions before loading can now be plotted in Fig.8 from the above values of *p' and q'*. This point is A. Since loading is instantaneous(i.e., undrained), the stress conditions in clay represented by the *p' versus q'* plot immediately after loading, will fall on the stress path BCA.

Immediately after loading,

σ_1 = 75+40 = 115kN/m2 and σ_3 = 37.5+25 = 62.5kN/m2. So

$$\boldsymbol{q' = \frac{\sigma_1' + \sigma_3'}{2} = \frac{\sigma_1 - \sigma_3}{2} = \frac{115-62.5}{2} = 26.25\ kN/m^2}$$

With this value of *q'*, we can locate point D. At the end of consolidation, $\sigma'_1 = \sigma_1$ = 115kN/m2 $\sigma'_3 = \sigma_3$ = 62.5kN/m2,

So

$$\boldsymbol{p' = \frac{\sigma_1' + \sigma_2'}{2} = \frac{115+62.5}{2} = 88.75\ kN/m^2}$$

And q' = 26.25$\boldsymbol{kN/m^2}$

The preceding values of *p' and q'* are plotted as point E. FEG is a geometrically similar stress path drawn through E. ADE is the effective stress path that a soil element, at average depth of the clay layer, will follow. AD represents the elasticsettlement, and DE represents the consolidation settlement. For elastic settlement (stress path A to D):

$$\boldsymbol{s_{\varepsilon} = [(\epsilon_1\ \ at\ D) - (\epsilon_1\ at\ A)]H_t = (0.004 - 0.001)3 = 0.09m}$$

For consolidation settlement (stress path D to E), based on previous valid assumption the volume metric strain between D and E is the same as the volumetric strain between A and H. Note that H is for point H, σ'_1 = 118kN/m2. So the volumetric strain, ϵ_v

$$\boldsymbol{\varepsilon_v = \frac{\Delta_e}{1+e_0} = \frac{c_c\ \log(118/75)}{1+0.9} = \frac{0.25\log(118/75)}{1.9} = 0.026}$$

The axial strain ϵ_1 along a horizontal stress path is about one-third the volumetric strain along the k_0 line, or

$$\boldsymbol{\epsilon_1 = \frac{1}{3}\epsilon_v = \frac{1}{3}(0.026) = 0.0087}$$

So, the consolidation settlement is

S_c = 0.0087 H_t = 0.0087(3) = 0.0261m and hence the total settlement is

$S_e + S_c$ = 0.09 + 0.0261 = 0.116m.

7. CLAY AND LATERAL PRESSURE

In the following Table 3 the variation of k_0, the lateral pressure and clay types is shown.

Table 3 Variation of $k_0$with clay types

Sl No	φ^0	Sin φ	k_0=1-Sin φ	Clay Type
1.	25	0.42	0.58	Montmorillonite
2.	30	0.50	0.50	Montmorillonite Illite
3.	35	0.57	0.53	Illite Kaolinite
4.	40	0.64	0.36	Kaolinite
5.	45	0.70	0.30	Quartz

K_0 values are different and therefore the settlement and differential settlement will be also different. The activity of each type of clay (Table 1) will further complicate the problem in the presence of flood water. The clay Montmorillonite and its swelling potential will be a major problem.

8. IMPORTANCE OF CLAY IN APPLIED ENVIRONMENTAL GEOTECHNOLOGY

Interparticlc friction is the principal property which permits soils and granular materials to resist load without deformation. Dry soils of virtually all types are highly stable. The suitability of soils for engineering purposes depends largely upon theirability to remain in place and support whatever loads may be placed upon them, either by a permanent engineering structure or by transient vehicle loads. A study of the properties which distinguish the more satisfactory from the less satisfactory soils indicates that in the majority of cases clays are detrimental to stability. It is apparent that wet clay has the effect of a lubricant in diminishing the natural resistance due to friction that would otherwise exist. It is necessary that the civil engineer responsible for construction of any form of earth work should be informed not only concerning the quantity of clay minerals that are present but also about the nature and their potential influence on the engineering properties of the soil.

As the ability of soils to resist deformation depends very largely on the internal friction, wet clay has the effect of reducing or canceling out the frictional resistance. It may also be pointed out that the so-called cohesive resistance is induced almost entirely by the clay fractions, and therefore that clean sands are non-cohesive. Again we must note the important part played by water, as finely ground dry clay particles exhibit no cohesive properties. If water is added to a dry soil, the cohesive resistance will normally increase with the addition of moisture and in most cases the frictional resistance will not be greatly impaired until a certain amount of moisture is added.

Beyond this point, the friction will diminish but the cohesive resistance may continue to increase up to some point of higher moisture content, after which both values will diminish as the soil approaches a completely fluid state.

Metals are typical substances having little or no internal friction; in them, resistance values are almost entirely a result of the cohesive or tensile strength.

Fig. 9(a) shows wheel load and sub-grade in road construction in lowland coastal area. Fig.9(b) is a chart showing characteristic curves illustrating loss in stability or internal resistance of a crushed sandy gravel due to the addition of increments of plastic clay.The combined figures 9(a) and 9(b) clearly indicates the instability created by Montmorillonite (Bentonite), Illite (local clay), and Kaolinite in the presence of water.Bentonite fails first followed by

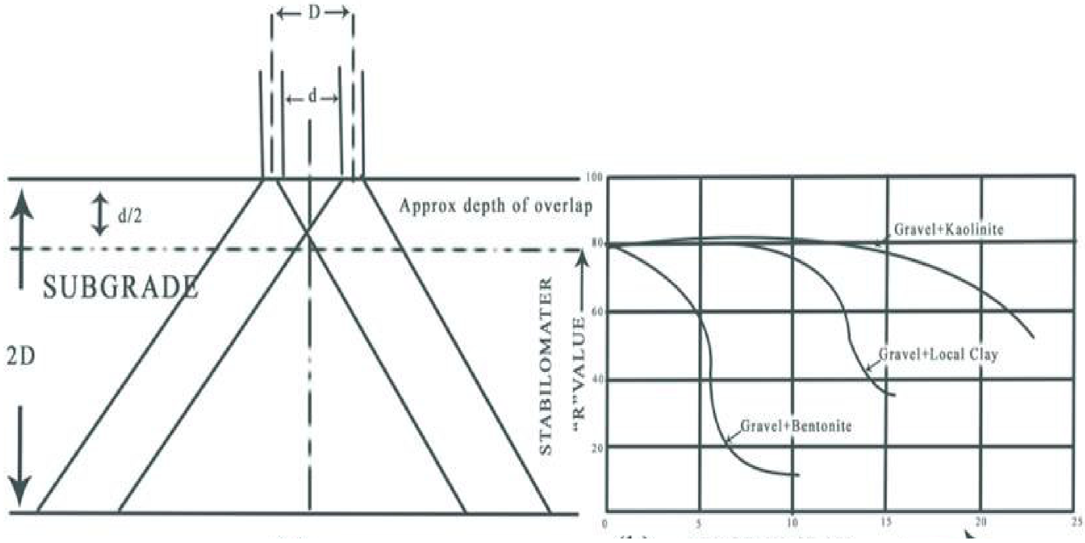

Fig.9 Relationship among sub grade, stability and percent of clay andStability variation with different clay types [6]

Illite. Kaolinite is more stable.

9. CONCLUSIONS

1. Every port has its own geological setup and environmental geotechnology.
2. Seismosediments create heterogeneous nature to the soil. They represent accumulated disturbed samples.
3. The φ value for clay varies with type of clay.
4. Calculation of settlement by using strain methods reveals more intrinsic nature of clay behavior than stress oriented methods.
5. Increase in stability of a soil is not a function of cohesion and angle of internal friction alone, because water introduces "turn around stages" and create instability especially in coastal area lowlands.
6. In a mixed sediments (seismosediments, flood sediments) a depth of 3m of clay (say Kaolinite 1m, Illite 1m and Montmorillonite 1m) KIM – will create 24 different geotechnical sequences which in turn will increase the complexity of the geotechnical problem.
They are :
KIM,MIK,IMK,KKK,III,MMM,KII,IKI,IIK,IKK, KIK,KKI, MII,IMI,IIM,KMM,MKM,MMK,KMI, IMM, MIM, MMI, MKI, IKM.
7. The final conclusion is that the construction techniques or site preparation should match with Geotechnical properties of different mixed types of clays.

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Int. J. of GEOMATE, Sept., 2013, Vol. 5, No. 1 (Sl. No. 9), pp. 653-659.
MS No. 114a received on June 19, 2012and reviewed under GEOMATE publication policies.

Pertinent discussion including authors' closure, if any, will be published in the Sept. 2014 if the discussion is received by March, 2014.
Corresponding Author:J.Rajaraman