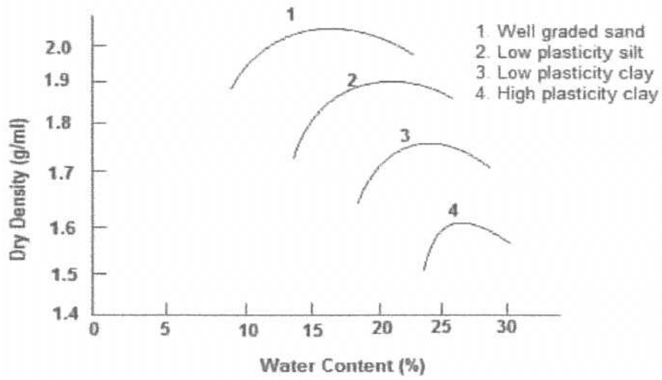
(14)

REVISION: ~~10~~ 15

COURSE TITLE: GEO TECHNICAL ENGINEERING — COUSRE CODE : ~~3027~~ 5013

Qt No	Scoring indicator	Split up	Sub total	Total mark
	PART A			
1.	Alluvial(water),collovial (gravity),aeolin(wind)	Any 2	2	10
2.	Water content, $w = W_w/W_d$ hence $1+w = (W_w+W_d) / W_d = W/ W_d$ $W_d = W/(1+w)$ $\gamma_d = W_d / V = W/(1+w) V$ $\gamma_d = \gamma /(1+w)$	2	2	
3.	Capillary water : It is that water which is lifted up by surface tension above the free water surface	1	2	
	Adsorbed water: 1.hygroscopic : water which the soil particles freely adsorb from atmosphere by the physical force of attraction and force adhesion 2.film water: It is attached to the surface of soil particles as a film on the layer of hygroscopic film	1		
4.	Undisturbed samples especially cohesion less soil	2	2	
5.	The load is transferred to the adjoining soil by friction between the pile and the surrounding soil. The load is transferred downward and laterally to the soil is called friction pile. A friction pile gets his bearing load mostly from friction along the shaft.	1	2	
	Typical end-bearing piles are driven through the very soft soil, such as a loose silt-bearing stratum underlain by compressible strata. A bearing pile gets his bearing load mostly form its resistance at its pile point, normally these are very short pile	1		
	PART B			
1.	**Bulk unit wt** $\gamma = W / V = (\gamma_s V_s + \gamma_w V_w) / V$ **Partially saturated samples ,** $V_s = 1$; $V_w = e_w$ **and** $V = (1+e)$	2	6	30
	$\therefore \gamma = (\gamma_s.1+ \gamma_w. e_w)/ (1+e)$ **but** $\gamma_s = G \gamma_w$ **and** $e_w = e_s$	2		
	$\therefore \gamma = (G \gamma_w+ \gamma_w. e. S)/ (1+e) = (G + e. S) \gamma_w / (1+e)$	2		
2.	Liquid limit ,plastic limit , shrinkage limit	1	6	
	Procedure for finding plastic limit : 1. Put 20 gm of air-dried soil, passed thorough 425 mm sieve into an evaporating dish. Add distilled water into the soil and mix it thoroughly to form uniform paste (the soil paste should be plastic enough to be easily moulded with fingers.)	5		

	2. Take one of the soil masses and roll it on the glass plate using your figures. The pressure of rolling should be just enough to make thread of uniform diameter throughout its length. The rate of rolling shall be between 60 to 90 strokes per min. 3. Continue rolling until you get the thread diameter of 3 mm. 4. If the thread does not crumble at a diameter of 3 mm, kneed the soil together to a uniform mass and re-roll. 5. Continue the process until the thread crumbles when the diameter is 3 mm. 6.Collect the pieces of the crumbled thread for moisture content determination. 7. Repeat the test to at least 3 times and take the average of the results calculated to the nearest whole number.			
3.	**seepage velocity** is the **velocity** of groundwater which is calculated using Darcy's law. It is the **velocity** at which ground water goes through a porous medium like soil and rocks. **Discharge velocity** is the rate of flow of a liquid across a given cross sectional area, though not necessarily into the ground	6	6	
4.	1.**Effect of Water Content on Compaction of Soil** At low water content, the soil is stiff and offers more resistance to compaction. As the water content is increased, the soil particles get lubricated. The soil mass becomes more workable and the particles have closer packing. The dry density of the soil increases with an increase in the water content till the optimum water content in reached. At that stage, the air voids attain approximately a constant volume. With further increase in water content, the air voids do not decrease, but the total voids (air plus water) increase and the dry density decreases. **2.Amount of Compaction** The compaction of soil increases with the increase in amount of compactive effort. With increase in compactive effort, the optimum water content required for compaction also decreases. At a water content less than the optimum, the effect of increased compaction is more predominant. At a water content more than the optimum, the volume of air voids become almost constant and the effect of increased compaction on soil is not significant. **3.Type of Soil:** The compaction of soil depends upon the type of soil. The maximum dry density and the optimum water content for different soils are shown in figure. In general, coarse grained soils can be compacted to higher dry density than fine-grained soils.	2 2	6	

	With the addition of even a small quantity of fines to a coarse-grained soil, the soils attain a much higher dry density for the same compactive effort. **4.Method of Soil Compaction:** The dry density achieved depends not only upon the amount of compactive effort but also on the method of compaction. For the same amount of compactive effort, the dry density will depend upon whether the method of compaction utilizes kneading action, dynamic action or static action. compaction curve obtained is different from that obtained from the other conventional tests in which an equal compactive effort is applied.  Fig: Compaction curves for different soils	2 2		
5.	For small areas and less important buildings, one borehole or trial pit at the center may be sufficient. For a compact building site, covering an area of about 0.4 ha (4000 m^2), a total of five boreholes or test pits, that is, one borehole or trial pit in each corner and one at the center, is adequate. For large and important structures, exploration is carried at important locations, as suggested by the geotechnical engineer, in addition to the corners of the site. The spacing between borings or test pits may be 10-30 m, depending on the variation in subsurface conditions and/or loading.	2 2 2	6	
6.	**Based on materials** **Timber piles:** Timber can be used for manufacture of temporary piles and also for permanent ones in regions where timber is readily and economically available. It's most suitable for long cohesion piling and piling under embankments. **Steel piles:** Steel can be used for both temporary and permanent works. They are suitable for handling and driving for piles with prolonged	 1 1	6	

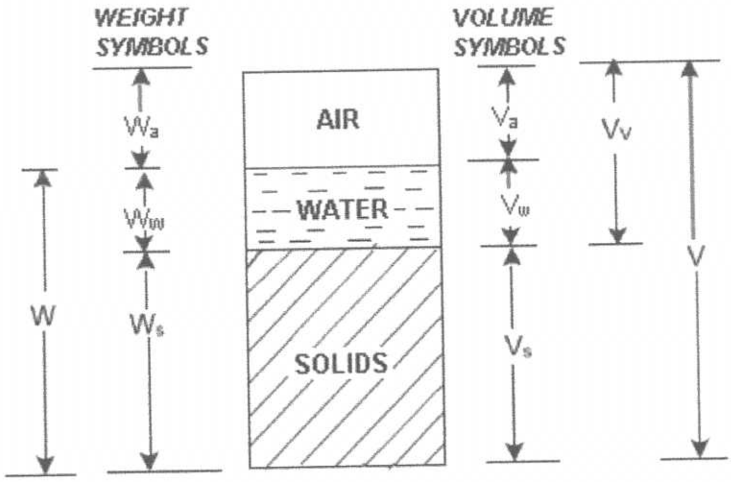
	lengths. Their relatively small cross sectional area along with the high strength makes penetration easier in firm soil. If it's driven in to a soil with low Ph value, there may occur a risk of corrosion which can be eliminated by tar coating or cathodic protection. **Concrete Piles** Concrete piles may be precast, prestressed, cast in place, or of composite construction Precast concrete piles may be made using ordinary reinforcement or they may be prestressed. Precast piles using ordinary reinforcement are designed to resist bending stresses during picking up & transport to the site & bending moments from lateral loads and to provide sufficient resistance to vertical loads and any tension forces developed during driving. Prestressed piles are formed by tensioning high strength steel prestress cables, and casting the concrete about the cable. When the concrete hardens, the prestress cables are cut, with the tension force in the cables now producing compressive stress in the concrete pile. It is common to higher-strength concrete (35 to 55 MPa) in prestressed piles because of the large initial compressive stresses from prestressing. Prestressing the piles, tend to counteract any tension stresses during either handling or driving. **Method of Installation of piles** **Driven or displacement piles** They are usually pre-formed before being driven, jacked, screwed or hammered into ground. This category consists of driven piles of steel or precast concrete and piles formed by driving tubes or shells which are fitted with a driving shoe. The tubes or shells which are filled with concrete after driving. Also included in this category are piles formed by placing concrete as the driven piles are withdraw **Bored or Replacement piles** They require a hole to be first bored into which the pile is then formed usually of reinforced concrete. The shaft (bore) may be eased or uncased depending upon type of soil	1 2 2		
7.	1.The outer surface of well curb steining should be regular and smooth as possible 2.The radius of curb should be kept 2 to 4cm larger than the outside radius of well steining 3.The cutting edge of curb should be uniform thickness and sharpness 4.The dredge should be done uniformly on all sides in circular well and both pockets of twin well	2 2 2	6	

<table>
<tr><th colspan="5">PART C</th></tr>
<tr><td>III.a</td><td>Bulk unit weight $\gamma = (\gamma_w G(1+w)) / 1+e$
$19.62 = (10*2.7*1.22) / 1+e$
$e = 0.678$
$e = wG/S$
$S = (0.22*2.7) / 0.678 = 87.61\%$
Saturated unit wt $\gamma_{sat} = \gamma_w (G+e) / (1+e)$
$= 10*(2.7+0.678) / 1.678$
$= 20.13\ kN/m^2$</td><td>3

2

3</td><td>8</td><td>15</td></tr>
<tr><td>III.b</td><td>Soil is not a coherent solid material like steel and concrete, but is a particulate material. Soils, as they exist in nature, consist of solid particles (mineral grains, rock fragments) with water and air in the voids between the particles. The water and air contents are readily changed by changes in ambient conditions and location.

As the relative proportions of the three phases vary in any soil deposit, it is useful to consider a soil model which will represent these phases distinctly and properly quantify the amount of each phase. A schematic diagram of the three-phase system is shown in terms of weight and volume symbols respectively for soil solids, water, and air. The weight of air can be neglected.



The soil model is given dimensional values for the solid, water and air components.

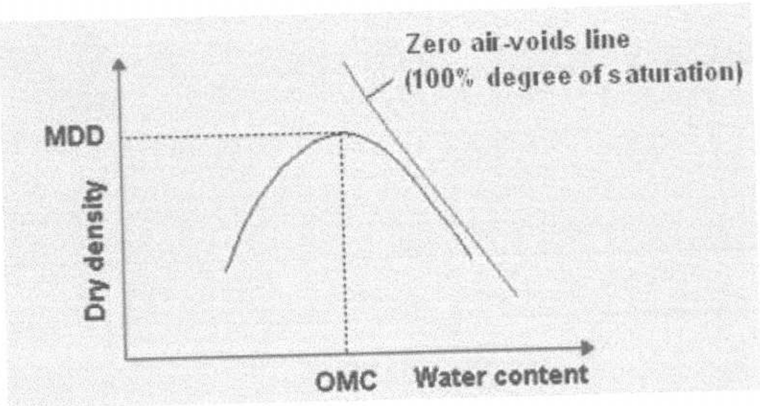
Total volume, $V = V_s + V_w + V_v$</td><td>7</td><td>7</td><td></td></tr>
</table>

IV.a	1. Clean and dry the Pycnometer. Tightly screw its cap. Take its mass (M_1) to the nearest of 0.1 g. 2. Mark the cap and Pycnometer with a vertical line parallel to the axis of the Pycnometer to ensure that the cap is screwed to the same mark each time. 3. Unscrew the cap and place about 200 g of oven dried soil in the Pycnometer. Screw the cap. Determine the mass (M_2). 4. Unscrew the cap and add sufficient amount of de-aired water to the Pycnometer so as to cover the soil. Screw on the cap. 5. Shake well the contents. Connect the Pycnometer to a vacuum pump to remove the entrapped air, for about 20 minutes for fine-grained soils and about 10 minutes for coarse-grained soils. 6. Disconnect the vacuum pump. Fill the Pycnometer with water, about three-fourths full. Reapply the vacuum for about 5min till air bubbles stop appearing on the surface of the water. 7. Fill the Pycnometer with water completely upto the mark. Dry it from outside. Take its mass (M_3). 8. Record the temperature of contents. 9. Empty the Pycnometer. Clean it and wipe it dry. 10. Fill the Pycnometer with water only. Screw on the cap upto the mark. Wipe it dry. Take its mass (M_4). $G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)}$	8	8	15
IV.b	**$\gamma_{sat} = \gamma_w (G+e)/(1+e) =$ 18.64 = 10*(G+0.3G) /1+0.3G** **G = 2.518** **Shrinkage limit Ws = (γ_w / γ_d) – (1/ G)** **= 0.169 = 16.9 %**	7	7	
V.a	For laminar flow through saturated soil the discharge per unit time is proportional to the hydraulic gradient Q = kiA Q /A = ki v = ki q = rate of flow A = Total c/s area of soil mass ..i = hyrualic gradiant = h/ L K = coef of permiability V = velocity of flow	8	8	15

V.b	In soils, the permeant or pore fluid is mostly water whose variation in property is generally very less. Permeability of all soils is strongly influenced by the density of packing of the soil particles, which can be represented by void ratio (**e**) or porosity (**n**). In sands, permeability can be empirically related to the square of some representative grain size from its grain-size distribution. For filter sands, Allen Hazen in 1911 found that **k » 100 (D_{10})^2** cm/s where **D_{10}**= effective grain size in cm. Different relationships have been attempted relating void ratio and permeability, such as **k μ $e^3/(1+e)$**, and **k μ e^2**. They have been obtained from the Kozeny-Carman equation for laminar flow in saturated soils. $k = \frac{1}{k_0 k_T S_S^2} \cdot \frac{e^3}{1+e} \cdot \frac{\gamma_w}{\eta}$ where **k_o** and **k_T** are factors depending on the shape and tortuosity of the pores respectively, **Ss** is the surface area of the solid particles per unit volume of solid material, and **g_w** and **h** are unit weight and viscosity of the pore water. The equation can be reduced to a simpler form as $k = C.\frac{e^3}{1+e} \approx C.e^2$ For silts and clays, the Kozeny-Carman equation does not work well, and **log k** versus **e** plot has been found to indicate a linear relationship. For clays, it is typically found that $\log_{10} k = \frac{e - e_k}{C_k}$ where **C_k**is the permeability change index and **e_k** is a reference void ratio	7			
VI.a	Soil is compacted into a 1000 cm^3 mould in 3 equal layers, each layer receiving 25 blows of a 2.6 kg rammer dropped from a height of 310 mm above the soil. The compaction is repeated at various moisture content To assess the degree of compaction, it is necessary to use the dry unit weight, which is an indicator of compactness of solid soil particles in a given volume. The laboratory testing is meant to establish the maximum dry density that can be attained for a given soil with a	6 Fig =2		7	15

standard amount of compactive effort.
In the test, the dry density cannot be determined directly, and as such the bulk density and the moisture content are obtained first to calculate the dry density as $\gamma_d = \frac{\gamma_t}{1+w}$, where γ_t = bulk density, and w = water content.

A series of samples of the soil are compacted at different water contents, and a curve is drawn with axes of dry density and water content. The resulting plot usually has a distinct peak as shown. Such inverted "V" curves are obtained for **cohesive soils** (or soils with fines), and are known as compaction curves.



Dry density can be related to water content and degree of saturation (S) as

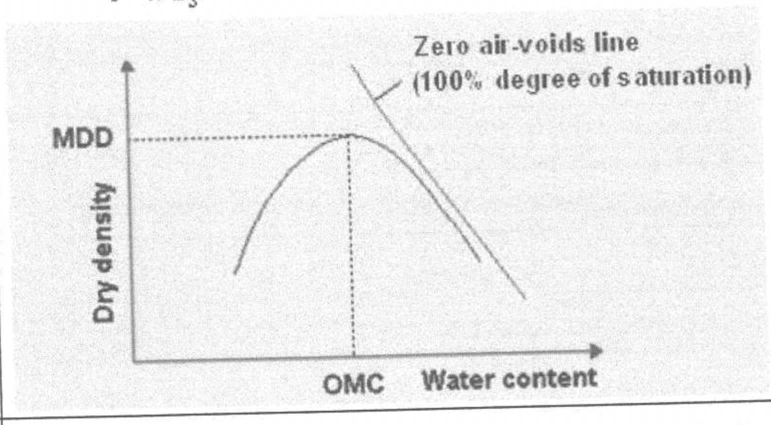
$$\gamma_d = \frac{G_s \cdot \gamma_w}{1+e} = \frac{G_s \cdot \gamma_w}{1+\frac{w.G_s}{S}}$$

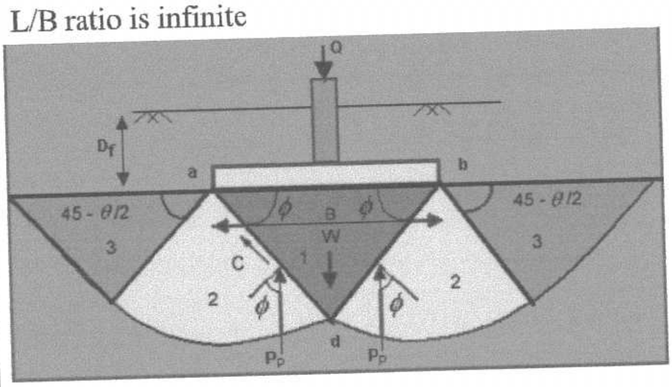
Thus, it can be visualized that an increase of dry density means a decrease of voids ratio and a more compact soil.

Similarly, dry density can be related to percentage air voids (n_a) as

$$\gamma_d = \frac{(1-n_a)G_s \cdot \gamma_w}{1+wG_s}$$

VI.b	The relation between moisture content and dry unit weight for a saturated soil is the **zero air-voids line.** It is not feasible to expel air completely by compaction, no matter how much compactive effort is used and in whatever manner	3 2	7	

	$\gamma_d = \frac{(1-n_a)G_S \cdot \gamma_w}{1+wG_s}$ 	2		
VII. a	The test is conducted in a bore hole by means of a standard split spoon sampler.Once the drilling is done to the desired depth, the drilling tool is removed and the sampler is placed inside the bore hole. By means of a drop hammer of 63.5kg mass falling through a height of 750mm at the rate of 30 blows per minute, the sampler is driven into the soil. This is as per IS -2131:1963. The number of blows of hammer required to drive a depth of 150mm is counted. Further it is driven by 150 mm and the blows are counted. Similarly, the sampler is once again further driven by 150mm and the number of blows recorded. The number of blows recorded for the first 150mm not taken into consideration.. The number of blows recorded for last two 150mm intervals are added to give the **standard penetration number (N)**. In other words, **N = No: of blows required for 150mm penetration beyond seating drive of 150mm.** If the number of blows for 150mm drive exceeds 50, it is taken as refusal and the test is discontinued. The standard penetration number is corrected for dilatancy correction and overburden correction Dilatancy Correction $N_C = 15 + 0.5 (N_R - 15)$ Where N_R is the recorded value and N_C is the corrected value.	7	7	15

	If N_R less than or equal to 15, then $N_c = N_R$ Overburden Pressure Correction $N_c = C_N N$ Here C_N is the correction factor for the overburden pressure			
VII. b	To access the general suitability of the site. To achieve safe and economical design of foundations and temporary works. To know the nature of each stratum and engineering properties of the soil and rock, which may affect the design and mode of construction of proposed structure and foundation. To foresee and provide against difficulties that may arise during construction due to ground and other local conditions. To find out the sources of construction material and selection of sites for disposal of water or surplus material. To investigate the occurrence or causes of all natural and man-made changes in conditions and the results arising from such changes. To ensure the safety of surrounding existing structures. To design for the failed structures or remedial measures for the structures deemed to be unsafe. To locate the ground water level and possible corrosive effect of soil and water on foundation material.	8	8	
VIII. a	Terzaghi's Bearing Capacity Theory: Assumptions in Terzaghi's Bearing Capacity Theory. Depth of foundation is less than or equal to its width. Base of the footing is rough. Soil above bottom of foundation has no shear strength; It is only a surcharge load against the overturning load Surcharge upto the base of footing is considered. Load applied is vertical and non-eccentric. The soil is homogenous and isotropic. L/B ratio is infinite 	8	8	15

we obtain, $Q_d = B(cN_c + \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma)$ ----------

the quantities N_c, N_q, N_γ are called bearing capacity factors.

$$N_q = \frac{a^2}{2\cos^2(45+\phi/2)}$$

$$N_c = \cot\phi\left[\frac{a^2}{2\cos^2(45+\phi/2)} - 1\right]$$

$$N_\gamma = \frac{1}{2}\tan\phi\left[\frac{K_p}{\cos^2\phi} - 1\right]$$

where K_p = passive earth pressure coefficient

$$a = \exp\left[\left(\frac{3\pi}{4} - \frac{\phi}{2}\right)\tan\phi\right]$$

VIII. b

The plate load test is a semi-direct method to measure the allowable pressure of soil to induce a give amount of settlement. Plates, round or square, varying in sizes, from 30 to 60 cm and thickness of abo 2.5 cm are employed for the test. 7*1 7

The load on the plate is applied by making use of a hydraulic jack. The reaction of the jack load is take by a cross beam or a steel truss anchored suitably at both the ends. The settlement of the plate measured by a set of three dial gauges of sensitivity 0.02mm placed at 120° apart. The dial gauges a fixed to independent supports which do not get disturbed during the test. Fig shows the arrangeme for the plate load test.

Procedure:

The method of performing the test is essentially as follows:

Excavate a pit of size not less than 5 times the size of the plate. The bottom of the pit coincides wi level of the foundation.

If water table is above the level of foundation, pump out the water carefully and it should be kept ju at the level of the foundation.

A suitable size of the plate is selected for the test. Normally a plate of size 30cm is used in sandy soi and bigger size in clay soils. The ground should be leveled and the plate is seated over the ground.

A seating load of about $70\,gm/cm^2$ is first placed and released after sometime. A higher load is ne placed on the plate and settlements are recorded by means of the dial gauges.

	Observations on every load increment shall be taken until the rate of settlement is less than 0.25m per hour. Load increments shall be approximately one-fifth of the estimated safe bearing capacity (the soil. The average of the settlements recorded by 2 or 3 dial gauges taken as the settlements of th plate for each of the load increment. The test should continue until a total settlement of 2.5cm or the settlement at which the soil fail whichever is earlier, is obtained. After the load is increased, the elastic rebound of the soil should b recorded.			
IX.a	1.Laying the Cutting Edge 2. Alignment Control 3. Construction of Well Curb 4. Construction of Well Steining 5. Well Sinking.	2 1 1 2 2	8	15
IX.b	1. **Shallow foundation** o Individual footing or isolated footing o Combined footing o Strip foundation o Raft or mat foundation	1	7	
	2. **Deep Foundation** o Pile foundation o Drilled Shafts or caissons	1		
	For most of the fine grained soils (containing silt and clays) it might be sufficient to use simple spread footings, it is largely depending on the magnitude of the load. The location of the foundations in relation to the soil (need to be aware of foundation walls and hydrostatic pressure as moisture is present in the soil). If the soil is poor and structure loads are relatively heavy, then alternate methods are required. Pile foundations might be required in some cases where fine cohesive silt and clay soil is present. (CH, OH). Sometimes it might be desirable and economically feasible to over excavate remove such soils that are not of bearing capacity; can remove compact and fill back or import other engineered soil. The geotechnical engineer based on borings will recommend suitable foundations systems or alternative solutions, also beating capacity, minimum depths, and special design or construction procedures might be established. Safe bearing capacity of soil equals to the ultimate bearing capacity divided by a safety factor (usually 2-4). Ultimate bearing capacity is	5		

	defined as the maximum unit pressure a soil can sustain without permitting large amounts of settlements. Bedrock has the highest safe bearing capacity. Well graded gravel and sand that are confined and drained have a safe bearing capacity of 3,000 – 12,000 PSF. Silts and clays have lower safe bearing capacity of 1,000 – 4,000 PSF.			
X.a	1. When the soil is weak and loads irresistible 2.When a clay soil be shrinkage and swelling seasonally 3.when the building above the water surface, such as sidewalks of ports 4. When there were neighboring buildings with foundations to close to new building which prevent the foundations excavations of the other kinds of foundations 5 - When requires balancing tensile forces or side pushing forces , called (anchor piles) when installed with vertical position or(batter piles) when installed with oblique position 6 - In areas where earthquakes occurs frequently , here it distributes in groups touched with each by strengthen ties 7 - when the groundwater level is high	7		15
X.b	**Foundation failure due to Soil Movement** When water present between soil particles is removed, the soil tend to move closer together. When water is absorbed by soil, the soil starts to swell. This movement of soil is based on the type of soil. Large movement is seen with clayey soils than sandy soils. These kind of movement of soil due to change in water content affects the foundation settlement. Foundation tends to settle to and excessive settlement of foundation may lead to differential settlement and damage to the structure. **Soil movement can occur due to following:** 1. Presence of vegetation or remains of old cut tree 2. Presence of mining areas 3. Shrinkable soils	4	8	

	Remedies for foundation failure due to soil movement: 1. Use of pile foundations where the soil is shrinkable, so that forces are transferred to the hard strata or rock. 2. Taking the foundation levels down to avoid foundation on shrinkable soils. 3. The vegetation is removed from the construction site and its roots are removed. Any cavity due to roots of vegetation shall be compacted and filled with concrete. 4. Presence of any mining areas needs to be inspected and professional help shall be taken while construction new buildings in such areas. **Foundation failure due Settlement of Soil Fill** If the building is constructed on a newly developed land by soil filling, the foundation on such soils tend to settle more with time as long time is needed for such soil to settle and become compact to resist the loads from the building foundation. **Remedies:** It shall be ensured that such soils are adequately compacted before construction begins on them. The foundation depth shall be increased to the hard strata or rock below the filled soil or pile foundations shall be used to prevent subsidence of foundation.	4		