SOLUTION / ANSWER KEY

GEOTECHNICAL ENGINEERING – I

T.E. CIVIL SEM-V (CBSGS) DEC 2017

Q. P. CODE: 18505

Date of Exam: 08/12/2017

1. a) Role of Geotechnical Engineer in Civil Engineering Practices:

i) Supervision of Site Investigation and Sub-Soil Exploration

ii) Performance and Analysis of Field and Laboratory Tests of Index Properties and Shear Strength of Soil

iii) Determination of Bearing Capacity of Soil

iv) Analysis and Design of Foundations (Shallow Foundations, Pile Foundations, Well Foundation, Machine Foundations etc.)

v) Analysis and Design of Flexible and Rigid Pavements

vi) Analysis of Stabilization of Slopes and Soil in General

vii) Analysis of Stability of Retaining Wall

viii) Analysis and Design of Sheet Piles

ix) Seepage Analysis of Hydraulic Stuctures

x) Analysis and Design of miscellaneous structures like bulkheads, cofferdams, shafts, tunnels, conduits etc.

1. b)

	Compaction	**Consolidation**
i)	Dynamic process	Static process
ii)	Almost instantaneous	Long-term process
iii)	Expulsion of air from voids	Expulsion of water from voids
iv)	Soil is always unsaturated	Soil is considered to be completely saturated.
v)	It is done before the construction of structures.	It starts as soon as the construction work begins.

1. c) Merits of Direct Shear Test :

i) It is easy to test sands and gravels.

ii) Large samples can be tested in large shear boxes, as small samples can give misleading results due to imperfections such as fractures and fissures, or may not be truly representative.

iii) Samples can be sheared along predetermined planes, when the shear strength along fissures or other selected planes are needed.

Demerits of Direct Shear Test:

i) The failure plane is always horizontal in the test, and this may not be the weakest plane in the sample. Failure of the soil occurs progressively from the edges towards the centre of the sample.

ii) There is no provision for measuring pore water pressure in the shear box and so it is not possible to determine effective stresses from undrained tests.

iii) The shear box apparatus cannot give reliable undrained strengths because it is impossible to prevent localised drainage away from the shear plane.

1. d) Quick Sand Condition:

When flow takes place in the upward direction, the seepage pressure also acts in the upward direction and the effective pressure is reduced. If the seepage pressure becomes equal to the pressure due to submerged weight of the soil, the effective pressure is reduced to zero. In such a condition, a cohesionless soil loses all its shear strength and the soil particles have a tendency to move up in the direction of flow. This phenomenon of lifting up of soil particles is called **quick sand** or **quick condition** *or* ***boiling condition*.**

Thus, during quick condition,

$$\sigma' = z*\Upsilon' - p_s = 0$$

or,

$$\sigma' = z*\Upsilon' - i * z * \Upsilon_w = 0$$

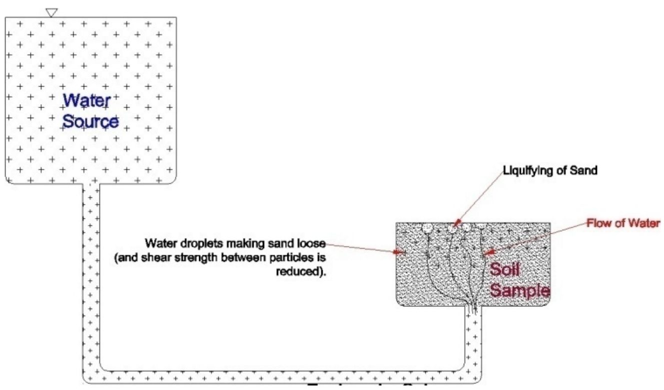
or,

$$\mathbf{i = \Upsilon' / \Upsilon_w}$$

This hydraulic gradient at such a critical state is called critical hydraulic gradient i_c given by

$$\mathbf{i_c = \Upsilon' / \Upsilon_w = (G-1)/(1+e)}$$

For loose deposits of ***sand*** *or silt, if voids ratio* ***e = 0.67*** *and specific gravity* ***G = 2.67****, then* ***i_c = 1****.*



2. a) Water content $w = W_w / W_d \rightarrow 1 + w = (W_w + W_d) / W_d = W / W_d \rightarrow W_d = W / 1 + w$

Now, $\Upsilon_d = W_d / V = W / (1 + w) * V$

$$\boldsymbol{\Upsilon_d = \Upsilon / (1 + w)}$$

2. b) Passing through 75 μ sieve = 10% → coarse grained soil

Passing 4.75 mm sieve = 70% → sand (S)

Uniformity coefficient = 8, coefficient of curvature = 2.8 → well-graded sand (SW)

Plasticity index = 4 → silty sand or clayey sand (SM-SC)

I.S. Classification → well-graded silty or clayey sand

2. c) $w_p = 25$ % and $I_p = 8$ % → $w_L = 25 + 8 = 33\%$

Volume change at liquid limit = 34% → Dry volume $V_d = V_L - 0.34\ V_L = 0.66\ V_L$

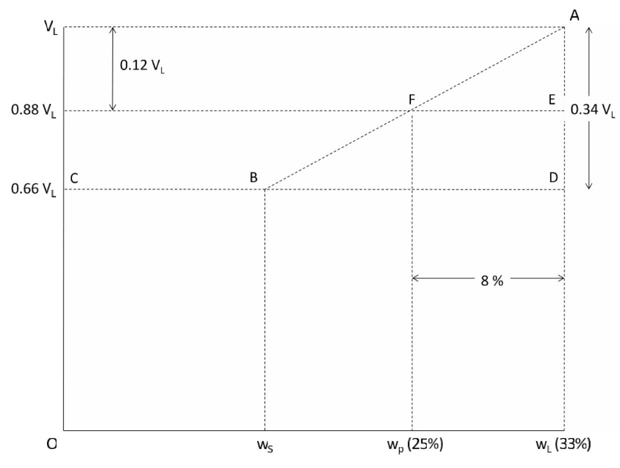
where V_L = volume at liquid limit

Volume change at plastic limit = 25% → Dry volume $V_d = V_P - 0.25\ V_P = 0.75\ V_P$

where V_P = volume at plastic limit

Therefore,

$$V_P = \frac{0.6}{0.7}\ V_L = 0.88\ V_L$$



From the above diagram,

$$\frac{B}{A} = \frac{F}{A} = \frac{8}{0.1\ \ VL}$$

$$BD = \frac{8}{0.1\ \ VL} * 0.34\ V_L = 22.6\ \%$$

$$\mathbf{w_S} = w_L - 22.6\ \% = 33 - 22.6 = \mathbf{10.4\ \%}$$

$$\mathbf{SR} = \{ (V_1 - V_2) / V \} / (w_1 - w_2) = \{ (V_L - V_P) * 100 \} / V_d \text{ x } (w_L - w_P)$$

$$= \{ (V_L - 0.88\ V_L) * 100 \} / 0.86\ V_L \text{ x } (33 - 25) = \mathbf{2.27}$$

3. a) $\Upsilon = 16.5\ kN/m^3$ and $w = 11\ \% = 0.11$

$$\Upsilon_d = \Upsilon / 1 + w = 16.5 / 1 + 0.11 = 14.86\ kN/m^3$$

Case 1: $w = 11\%$, $w = 0.11 = W_w / W_d$ and $V = 1\ m^3$

$$W_d = \Upsilon_d\ * V = 14.86 * 1 = 14.86\ kN$$

$$W_w = 0.11 * W_d = 0.11 * 14.86 = 1.6346\ kN$$

$$V_w = W_w / \Upsilon_w = 1.6346 / 9.81 = 0.1666\ m^3$$

Case 2: $w = 15\%$, $w = 0.15 = W_w / W_d$

$$W_w = w * W_d = 0.15 * 14.86 = 2.229\ Kn$$

$$V_w = W_w / \Upsilon_w = 2.229 / 9.81 = 0.2272\ m^3$$

- **Additional water required = (0.2272 – 0.1666) = 0.0606 m^3 = 60.6 litres**

3. b) **Porosity** or percentage voids (n) of a given soil sample is the ratio of the volume of voids to the total volume of a given sample.

$$\mathbf{n = V_V / V}$$

where V_V = volume of voids and V = total volume of a given sample

Seepage pressure (p_s) is the pressure exerted by the water on the soil through which it percolates.

$$\mathbf{p_s = h * \Upsilon_w}$$

where h = hydraulic head under which the water is flowing

Pore pressure or neutral pressure or pore water pressure (u) is the is the pressure transmitted through the pore fluid.

$$\mathbf{u = h_w * \Upsilon_w}$$

where h = piezometric height

Liquidity index or the water plasticity ration (I_L) is the ratio, expressed as a percentage, of the natural water content (w) of a soil minus its plastic limit (w_p), to its plasticity index (I_p)

$$\mathbf{I_L = (w - w_p) / I_p}$$

Relative density or density index (I_D) is defined as the ratio of the difference between the voids ratio of the soil in its loosest state (e_{max}) and its natural voids ratio (e) to the difference between the voids ratio in its loosest state and densest state (e_{min}).

$$\mathbf{I_D = (e_{max} - e) / (e_{max} - e_{min})}$$

3. c) Horizontal permeability $k_x = (k_1Z_1 + k_2Z_2 + k_3Z_3) / Z$

$k_x = (1.5 * 10^{-5}) + (1.8 * 10^{-7}) + (2 * 10^{-9}) / (1.5 + 1.8 + 2) = 2.86 \times 10^{-6}$ m/s

Vertical permeability $k_z = Z / (Z_1/k_1 + Z_2/k_2 + Z_3/k_3)$

$k_z = (1.5 + 1.8 + 2) / (1.5/10^{-5} + 1.8/10^{-7} + 2/10^{-9}) = 2.63 \times 10^{-9}$ m/s

4. a) Factors affecting compaction:

1. Water content - Proper control of moisture content in soil is necessary for achieving desired density. Maximum density with minimum compacting effort can be achieved by compaction of soil near its OMC (optimum moisture content). In the field the natural moisture content (NMC) of soil is either less than OMC or above OMC. If NMC of the soil is less than OMC, calculated amount of water should be added to soil with sprinkler attached to water tanker and mixed with soil by motor grader for uniform moisture content. When NMC of the soil is more than OMC, it is required to be dried by aeration to reach up to OMC.

2. Amount of Compaction – The amount of compaction greatly affects the maximum dry density and optimum water content of a given soil. The effect of increasing the compactive energy results in the increase of maximum dry density and reduction in OMC. However, the increase in maximum dry density doesn't have a linear relationship with increase of compactive effort.

3. Method of Compaction – The density obtained during compaction, for a given soil, greatly depends on the type of compaction or the manner in which the compactive effort is applied. The various variables in this aspect are: i) weight of the compactive equipment, ii) the manner or operation such as dynamic or impact, static, kneading or rolling and iii) time and area of contact between the compacting element and soil.

4. Type of Soil - Normally, heavy clays, clays & silts offer higher resistance to compaction where as sandy soils and coarse grained or gravelly soils are amenable for easy compaction. The coarse grained soils yield higher densities in comparison to clays. A well graded soil can be compacted to higher density.

5. Addition of Admixtures – The compaction properties / characteristics can be modified by a number of admixtures other than soil material. These admixtures have special application in stabilised soil construction.

4. b) Case 1: Before fill

At z = 3 m,

$$\sigma = 18x3 = 54 \text{ kN/m}^2$$

$$u = 9.81 \text{ x } 3 = 29.43 \text{ kN/m}^2$$

$$\sigma^{/} = \sigma - u = 24.57 \text{ kN/m}^2$$

At z = 5 m,

$$\sigma = 18x3 + 20x2 = 94 \text{ kN/m}^2$$

$$u = 9.81 \text{ x } 5 = 49.43 \text{ kN/m}^2$$

$$\sigma^{/} = \sigma - u = 44.57 \text{ kN/m}^2$$

Case 2: After fill

At z = 2.5 m,

$$\sigma = 22x2.5 = 55 \text{ kN/m}^2$$

$$u = 0 \text{ kN/m}^2$$

$$\sigma^{/} = \sigma - u = 55 \text{ kN/m}^2$$

At z = 5.5 m

$$\sigma = 22x2.5 + 18x3 = 109 \text{ kN/m}^2$$

$$u = 9.81x3 = 29.43 \text{ kN/m}^2$$

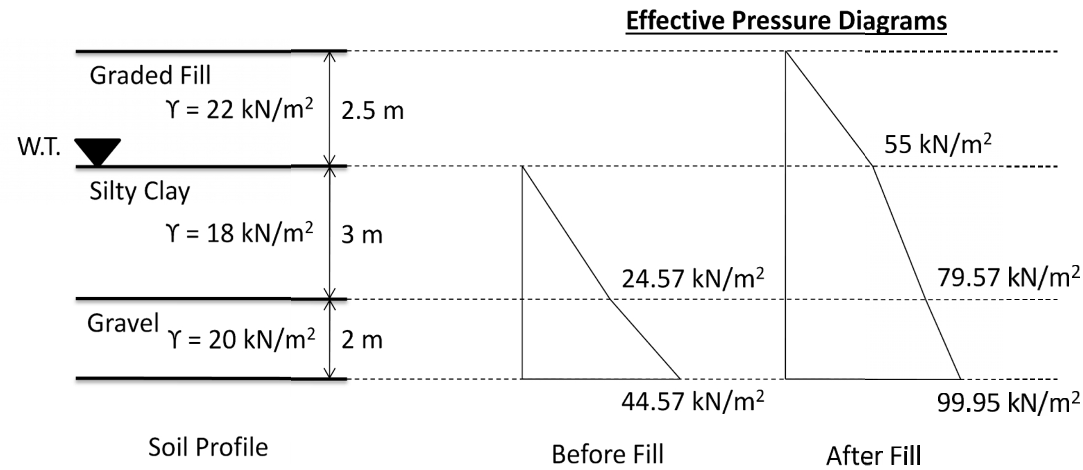
$$\sigma^{/} = \sigma - u = 79.57 \text{ kN/m}^2$$

At z = 7.5 m

$$\sigma = 22x2.5 + 18x3 + 20x4 = 149 \text{ kN/m}^2$$

$$u = 9.81x5 = 49.05 \text{ kN/m}^2$$

$$\sigma^{/} = \sigma - u = 99.95 \text{ kN/m}^2$$



4. c) Properties of Flow Nets:

1. The flow lines and equipotential lines meet at right angles to each other.
2. The fields are approximately squares, so that a circle can be drawn touching all its 4 sides.
3. The quantity of water flowing through each channel is the same.
4. The same potential drop occurs between two successive equipotential lines.
5. Smaller the dimensions of the field, greater will be the hydraulic gradient and velocity of flow through it.
6. In a homogeneous soil, every transition in the shape of the curves is smooth, either being elliptical or parabolic in shape.

Applications Uses of Flow Nets:

1. Determination of Seepage: $q = k\,H\,\frac{N}{N}\,\frac{b}{l}$
2. Determination of Hydrostatic Pressure: $h_w = h - Z$
3. Determination of Seepage Pressure: $p_s = (H - n\Delta h)\,\Upsilon_w$
4. Determination of Exit Gradient: $i_e = \frac{\Delta h}{l}$

5. a)

σ_3	σ_1	u	$\sigma_3' = \sigma_3 - u$	$\sigma_1' = \sigma_1 - u$
17	157	12	5	145
44	204	20	24	184
56	225	22	34	203

$$\sigma_1' = \sigma_3' \tan^2\alpha + 2c' \tan\alpha$$

$$145 = 5 \tan^2\alpha + 2c' \tan\alpha \ldots \text{(i)}$$

$$184 = 24 \tan^2\alpha + 2c' \tan\alpha \ldots \text{(ii)}$$

Subtracting (i) from (ii), we get

$$39 = 19 \tan^2\alpha$$

$$\text{or, } \alpha = 55.08^0$$

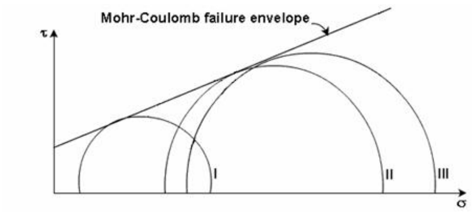
$$\therefore \mathbf{c' = 47.02\ kN/m^2}$$

$$\alpha = 45^0 + \frac{\varphi}{2}$$

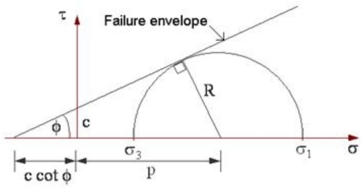
$$\therefore \boldsymbol{\varphi' = 20.16^0}$$

5. b) Mohr – Coulomb Failure Theory

When the soil sample has failed, the shear stress on the failure plane defines the shear strength of the soil. The Mohr circle of stress at failure for the sample can be drawn using the known values of the principal stresses.If data from several tests, carried out on different samples upto failure is available, a series of Mohr circles can be plotted. It is convenient to show only the upper half of the Mohr circle. A line tangential to the Mohr circles can be drawn, and is called the Mohr-Coulomb failure envelope.



If the stress condition for any other soil sample is represented by a Mohr circle that lies below the failure envelope, every plane within the sample experiences a shear stress which is smaller than the shear strength of the sample. Thus, the point of tangency of the envelope to the Mohr circle at failure gives a clue to the determination of the inclination of the failure plane. The orientation of the failure plane can be finally determined by the pole method.



The Mohr-Coulomb failure criterion can be written as the equation for the line that represents the failure envelope. The general equation is

$$\tau_f = c + \sigma_f \tan\varphi$$

Where τ_f = shear stress on the failure plane, *c* = apparent cohesion, σ_f = normal stress on the failure plane, *f* = angle of internal friction

The failure criterion can be expressed in terms of the relationship between the principal stresses. From the geometry of the Mohr circle, Rearranging,

$$\frac{1+\sin\phi}{1-\sin\phi} = \tan^2\left[\frac{\pi}{4}+\frac{\phi}{2}\right]$$

5. c) Assumptions in Terzaghi's One Dimensional Consolidation Theory:

i) The soil is homogenous (uniform in composition throughout) and isotropic (show same physical property in each direction).

ii) The soil is fully saturated (zero air voids due to water content being so high).

iii) The solid particles and water are incompressible.

iv) Compression and flow are one-dimensional (vertical axis being the one of interest).

v) Strains in the soil are relatively small.

vi) Darcy's Law is valid for all hydraulic gradients.

vii) The coefficient of permeability and the coefficient of volume compressibility remain constant throughout the process.

viii) There is a unique relationship, independent of time, between the void ratio and effective stress

ix) The boundary is a free surface offering no resistance to the flow of water from soil.

x) Secondary consolidation is disregarded.

6. a) Auger boring: Drilling is made using a device called Soil Auger.

Power driven (upto 3 to 5m) and Hand operated.

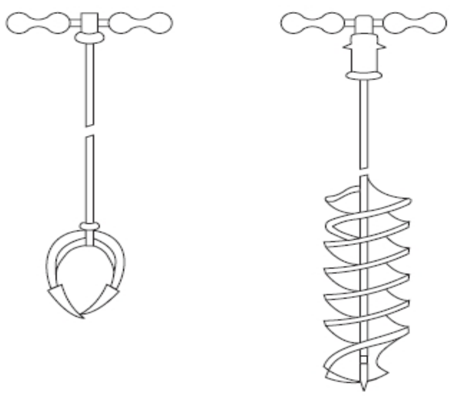
Advancement is made by drilling the auger by simultaneous rotating and pressing it into the soil

Dry and unsupported bore holes

When the auger gets filled with soil same, it is taken out and the soil sample collected.

It is particularly useful for subsurface investigations of highway, railway and airfield where the depth of exploration is small.

Auger boring cannot be used when there are large boulders present in the soil profile.



(a) Post-hole auger (b) Helical auger

Students may also explain core boring, wash boring, percussion & rotary drilling.

6. b) $T_V = \frac{\pi}{4}(\frac{U}{1})^2 = \frac{\pi}{4}(\frac{5}{1})^2 = 0.196$

$t = (T_V \times d^2)/c_v = (0.196 \times 800^2) / 1 \times 10^{-4}$

$= 1.256 \times 10^9$ minutes

$= 20.94 \times 10^6$ hours

$= 872.66 \times 10^3$ days

$= 2389.22$ years

6. c) **Geosynthetics** are synthetic products used to stabilize soil. They are generally polymeric products used to solve civil engineering problems. The common nomenclature of geosynthetics are i) geotextiles and geostrips, ii) geogrids, iii) geomembranes, iv) geonets, v) geocells and geoweb members, vi) geofoam, vii) geosynthetic clay liners, geocomposites. The polymeric nature of the products makes them suitable for use in the ground where high levels of durability are required. They can also be used in exposed applications. Geosynthetics are available in a wide range of forms and materials. These products have a wide range of applications and are currently used in many civil, geotechnical, transportation, geoenvironmental, hydraulic and private development applications including bridges, highway & roads, railway structures, industrial & mining, waterways & dams, protective structures etc.