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Stress-Strain & Shear Combination Resistance Against an Impact Load

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Abstract: For this STRESS – STRAIN – STRENGTH PROPERTIES, which relates to the three most common tests we performed and in line with the previous experiments, it is deduced that by applying them practically, it helps to explain the conditions of the loaded areas on the field. Therefore, in the field of operations, I have selected the three most common tests; I – D Consolidation (Compression) Test, where the loaded area is large with soil thickness of m & n ~5; Direct (box) Shear Test, using the Oedometer of Non-uniform Strains and Poor version of simple Shear; and Triaxial Test. However, the Std. TC tests at varying stresses of normal for Mohr-Coulomb Failure criteria figures into the practice very well. Hence the envelope from fig. 1.2 represents the limiting condition of the state of stress, for it is the conditions for which Mohr Circle lies above the envelope. It is also known that when Mohr Circle Tangent to envelope, then the point of tangency represents conditions on the failure plane, which results to rupture surface, where shear stress = Shear Strength; which leads to large deformations.

Keywords: Stress-Strain Relation, Mohr Test and Load Applications.

1. INTRODUCTION

In introducing these STRESS – STRAIN – STRENGTH PROPERTIES, with the formation of soil, is the settling of formation particles that we experience and talks about in terms of minerals for the formations with its gradual deposition of fine soil grains. To explain this further, the following were added for more clarification.

(1) If soil were linear-elastic-isotropic with infinite strength, then one simple test → 2 elastic constants to completely define σ - ϵ characteristics. By Lopez (2008)

(2) But soil is a "particulate" system of finite strength wherein the plastic-strains result from:

a) Deformation of particles – elastic & crushing (granular at high σ^1)

b) Sliding & rolling amongst particles

(3) Soil mechanics, therefore, developed several types of tests that attempt to simulate typical conditions encountered in the field (Long before the existence of reliable SOIL MOISTURE Tarzrgi,(2014).

2. METHODOLOGY PROCEDURE

PART 1. COMMON STRESS-STRAIN TEST

1. Three Most Common Tests

1.1 1-D Consolidation (Compression) = Odometer Test (For Stress-Strain)

(1) Field situation.

The loaded area is a large wet soil thickness; m & n → 5.

P = Load Pressure.

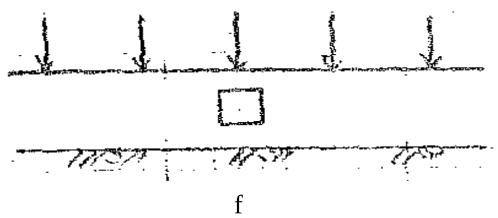

Fig. 1.1a. Loading Condition

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2) Lab Test

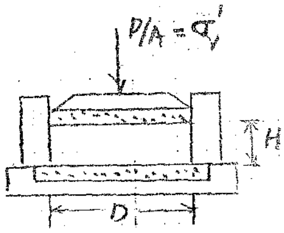


Fig. 1.1b. Profile Pressure.

D = 2,5 – 3”, D/H = 3-4
(60 – 15mm)

3) Typical σ - ϵ

- Lab Test Strain hardening
- Plastic deformations
- Constrained modulus

$D = \Delta\sigma^1_v/\Delta\epsilon_v = 1/m_v$
(coef. Of volume change)

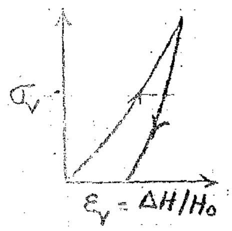


Fig. 1.1c; Plastic Deformation

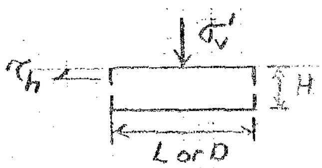
(4) “Elastic relationships

$$D = EI\,(I - u^I)/(I + u^I)\,(I - 2u^I)$$

$$K_o = u^I/(I - u^I) = 0.50 \text{ for } u^I = 1/3$$

1.2 Direct (Box) Shear Test (one at 1st strength tests)

(1) Field situation



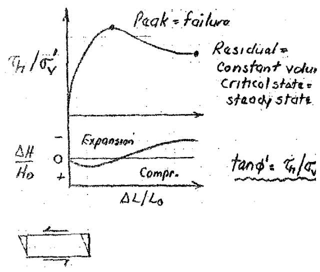


Fig. 1.2a Applied Load
Fig. 1.2b

(2) Lab test

- Dimensions like odometer
- Non–uniform strains
- Poor version of simple shea

(3) Typical t_n – “strain” medium – dense sand

This is the application of applied load on loose sand that is damp with the possibility of stressing to its limit for an absolute value. Thereby demonstrating the strength of the material, and showing an interface fractional forces of $F_{ss} = A_{ss} \cdot F_{ss} = A_{ss} \cdot \partial n.\, t\varphi_{ss}$. for soil material interactions, known as soil geosynthetic interaction, Tarzaghi, (2013).

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1.3. Triaxial Test (1st used soils 1930's; both $\sigma - \epsilon$ & strength)

1) Field Situation

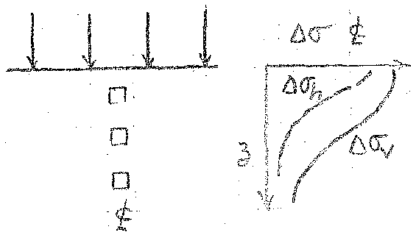


Fig. 1.3b. Sec. of Applied Load

(2) Lab test (cylindrical sample)

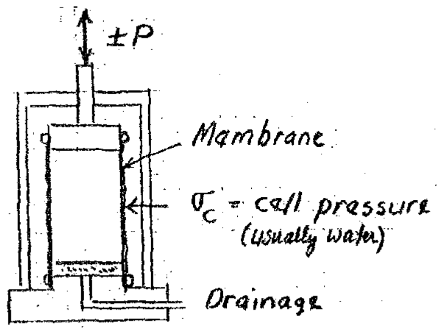
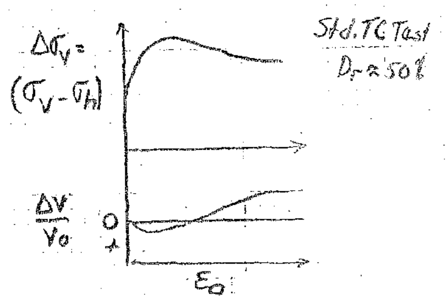


Fig. 1.3a. Triaxial Test

3) Typical $\sigma - \epsilon$



- D = 1.5 – 3", H/D = 2 – 2 ½
- $\sigma_y = \sigma_c + P/A$ (P + or-) $\sigma_h = \sigma_c$
- Controlled stress or strain (constant & = de/dt most common)
- $\epsilon_a = \Delta H/H_6$

PART II – 2, STRESS-STRAIN – STR. PROP

2.1 TRIAXIAL TEST (Cont.)

(3) Four basic types of tests (drained starting from isotropic σ^1_c)

(4) Triaxial Compression $\sigma_2 = \sigma_3 = \sigma_h$ **.** $\sigma_v > \sigma_h$ (K < 1)

. Axial (vertical) compression of sample

. b = Triaxial Extension ($\sigma_2 = \sigma_1 = \sigma_h$)**.** $\sigma_h > \sigma_v$ (K > I)

. Axial (vertical) extension of the sample**.** b = _

Since the triaxial test has been determined to be the conclusive approach for the new development and the new compression processes for improvements from old process

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Table; 2.1

Letter	$\Delta\sigma$	Description	Field case
A	$+\Delta\sigma_v$ $\Delta\sigma_h = 0$	Triaxial Compression	Shallow Footings.
B	$\Delta\sigma_v = 0$ $-\Delta\sigma_h$	Axial Tension.	Lateral wind Load
C	$\Delta\sigma_v = 0$ $+\Delta\sigma_h$	Axial Compression	Lateral Wind Load
D	$-\Delta\sigma_v$ $\Delta\sigma_h = 0$	Triaxial Extension	Wind Uplift

. Can, of course, vary both σ_v & σ_h during testing
. σ_2 condition defined by $b = \dfrac{(\sigma_2 - \sigma_3)}{(\sigma_1 - \sigma_3)}$ = TC = TE

a. Strength of Cohesionless soils (At "Low" Confinement)

3.0 INTRODUCTION OF MOHR CIRCLES

3.1 Mohr-Coulomb Failure Criteria

1) Std. TC tests at varying σ^1_c on med-dense sand (Drained, $\sigma = \sigma^1$)

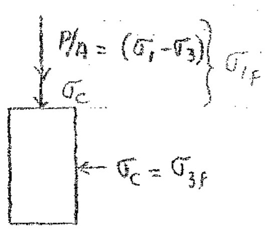

Fig. 3.1a; Std. Stress Block

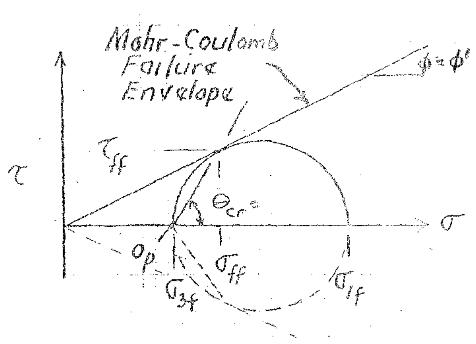

Fig. 3.1b; Mohr-Coulomb Failure.

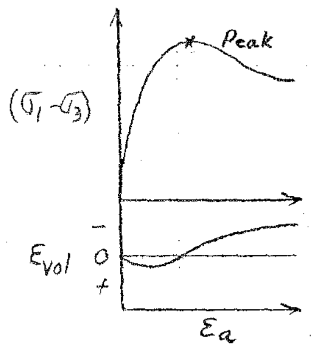

Fig. 3.1; Stress-Strain Graph.

The clay silt material grain size demo. refers to the above Strength of Cohesionless Soils at "Low" Confinement for the increase in capacity by remediation for the shear envelope looking at fig. 3.1b, when T_{ff} as the shear stress is being reduced. Dimitrive, (1993), Braja, (1999). Refer to below friction angle for straight line.

- $Ø^1$ = friction angle for straight line thru origin
- Bottom half is symmetrical

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2) Mohr-Coulomb failure criteria state.

No. 1 Envelop represents limiting condition of state of stress, i.e cannot have SOS for which Mohr circle lies above envelop

No. 2 When Mohr circle tangent to envelop, then point of tangency represents conditions on the failure plane = rupture surface

Where shear stress = shear strength,
leading to large deformations

. $\tau_{ff} = \sigma^{I}_{ff} \tan Ø^{1}$
. ff = on failure plane at failure
. O_{cr} = < between failure plane & σ_{If} plane. Das and Braja, (2001)

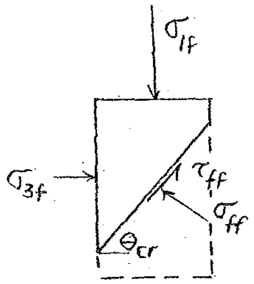

Fig.3.2; Friction Angle

4.0 EQUATION OF MOHR CIRCLE IN PARTS. IN TABLER FORM

PART III; 3 DEVELOPMENT OF MORE CIRCLES.

4.1; STRESS IMPACT

From Stress-Strain relationship, Mohr Circles' definition of grain material shows the measure of relativities. Note: **ø** is the angle between plane and σ_I plane (+)

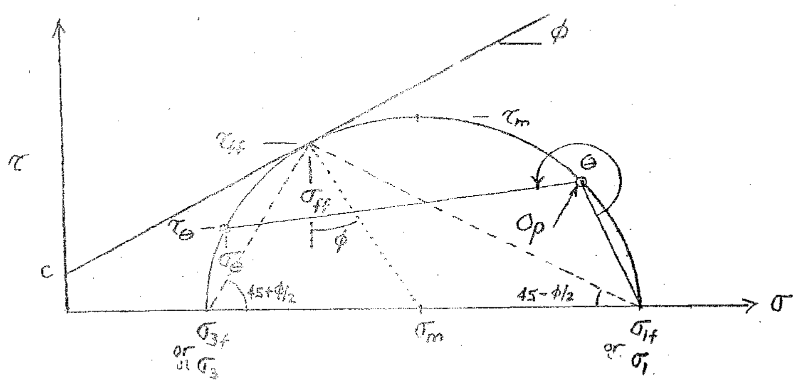

Fig. 4.1; Mohr Circles for stress-Impact.

Table III 4 –1. The equation for computing stresses with Mohr Circle

Definitions & Identities

(1) $N_Ø = 1 + \sin Ø/1 - \sin Ø = \tan^2 (45 + Ø/2)$ (2) $C = O$, $R_f = (\sigma_1/\ \sigma_3)_f = N_Ø$; $\sin Ø = (R_f - 1)/(R_f + 1)$

(3) $\sqrt{NØ} = \tan (45 + Ø/2) = \cos Ø/1 - \sin Ø$; (4) $I/\sqrt{NØ} = \tan (45 - Ø/2) = \cos Ø/1 + \sin Ø$

For any state of stress

(5) $\tau_{max} = \tau_m = 0.5 (\sigma_1 - \sigma_3)$; (6) $\tau_o = \tau_M \sin 2 = 2\tau_m \sin O \cos o$

(7) $\sigma_{mean} = \sigma_m = 0.5 (\sigma_1 + \sigma_3)$;

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For states of stress at failure

(8) $\sigma_O = \sigma_m + \tau_m \cos{}^2$

(9) ${}^2O = \sigma_1, \cos{}^{2O} + \sigma_3 \sin{}^2O$

(10) $\tau_{ff} = \tau_m \cos \varnothing$

(11) $2\varnothing = c + \sigma_{ff} \tan \varnothing$

(12) $\sigma_{ff} = \tau_{ff}/\tan \varnothing = \sigma_m - \tau_m \sin \varnothing$

(13) $\sigma_{ff} = \sigma_{ff} + \sqrt{N\varnothing}\ \tau t_{ff} = \sigma^{3f}\ N\varnothing + 2c\sqrt{N\varnothing}$

(14) $\sigma_{3f} = \sigma_{ff} - \tau_{ff}/\sqrt{N\varnothing} = \sigma_{If}/N\varnothing - 2C/\sqrt{N\varnothing}$

4.2 Presentation of Triaxial Test Data ($\sigma = \sigma^1$)

$Q_f/p_f = \tan \propto^1 = \sin \varnothing_1$

For ease of presentation, Not different failure criteria for grain Soil materials.

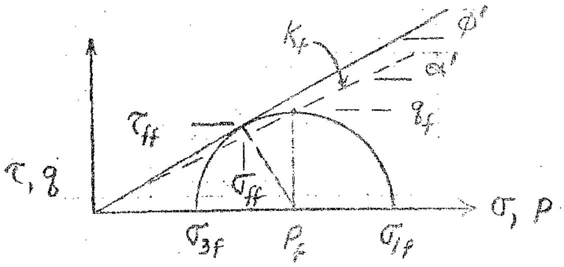

Fig. 4.2; Mohr Circle on Triaxial Test

4.3 Interpretation of Direct Shear Test (Really Indeterminate)

1) Usual assumption of horizontal failure plane, i.e, $t_{hmax} = t_{ff}$ ($\sigma = \sigma^1$)

. For NC sand starting from K_o condition

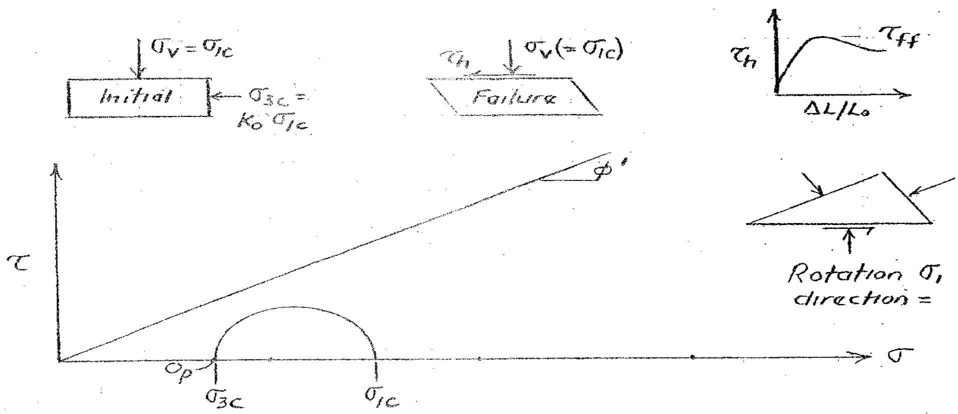

Fig. 4.3; Direct Shear Test Result.

2) Alternative assumption that $t_h(\max) = t_{\max}$ ($\sigma = \sigma^1$)

$\varnothing^1{}_1 = \arctan t_h/\sigma_v$ {common conservative

$\varnothing^1{}_2 = \arcsin t_h/\sigma_v$

($t_h/\sigma_v = 0.6 \sim \varnothing^1 = 31^0$ vs.37^0)

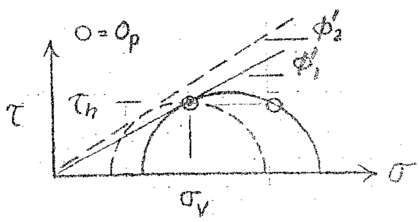

Fig. 4.4. More coulomb diagram.

The relative illustration of more circles in the effect of stress credibility of combination on soil conditions.

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5.0 EFFECTIVE STRESS RELATIONSHIP FOR STRAIN ELONGATION

5.1 Effect of Relative Density (Illustrated via Std. TC tests)

1) **Stress-Strain data ($\sigma = \sigma^1$)------------ Dense } $\sigma^1_c = \sigma^1_{3f} = 1$ atom**
----------- Loose } $\sigma^1_c = \sigma^1_{3f} = 1$ atom

Dense
. Small E_f
. Significant strain softening
. Initial small contraction then large expansion (dilation)

Loose
. Large E_f
. Little strain softening

For both at Critical = Steady State
* Unique e – q – p condition
* With continued shearing
[called critical state line]

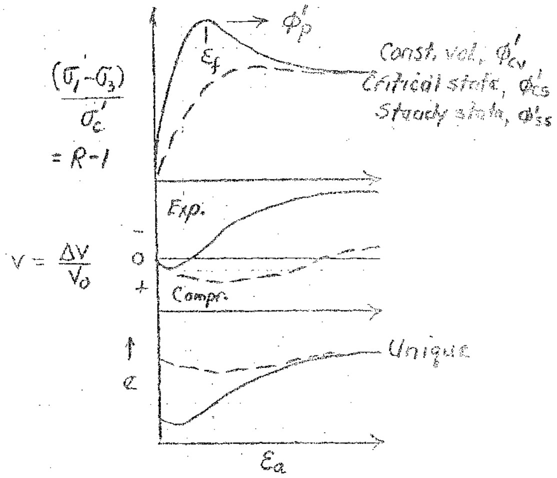

Fig.5.1; Dense, Loose and Critical State.

2. **Variation in Ø^1**
R = (σ_1/σ_3) = tan^2 (45 + Ø^1/2)} L & W
(1 + sin Ø^1)/(1 – sin Ø^1) }
. Also sin Ø^1 = (R – 1) / (R + 1)

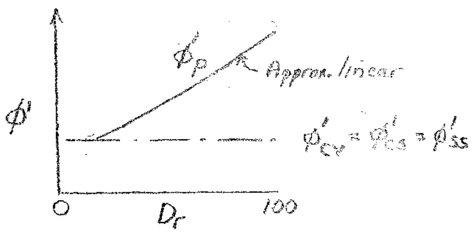

Fig. 5.2; Pure Critical State Graph.

5.2 Three components of strength (Rowe 2002), differs from (L & W, 2004)

1) Frictional resistance
. Coef. Of friction u = T/N = tan Ø^1_u
. Rowe (2002) states that Ø^1_u due to sliding only
. But more recent research indicates that also rolling at high
Ø^1_u (Skinner 1969, Geate)

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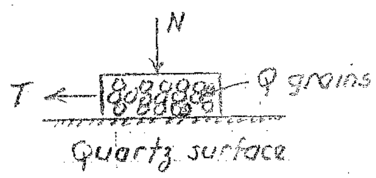


Fig. 5.3; Material Grain

2) Resistance due to Dilation

. Component due to expansion at soil during shear against the confining stresses (expansion from "interlocking")
. Magnitude is proportional to
rate of volume change

$E_{vol} = \Delta V/Vo = v$

$R_p = (\sigma^1{}_1/\sigma^1{}_3)_{max} = (1 + RD) \tan^2 (45 + Ø^1f/2)$

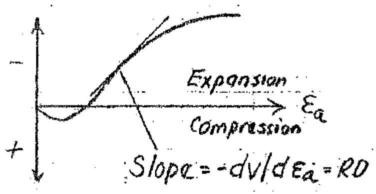


Fig. 5.2; Resistance Dilation.

3) Resistance due to interference

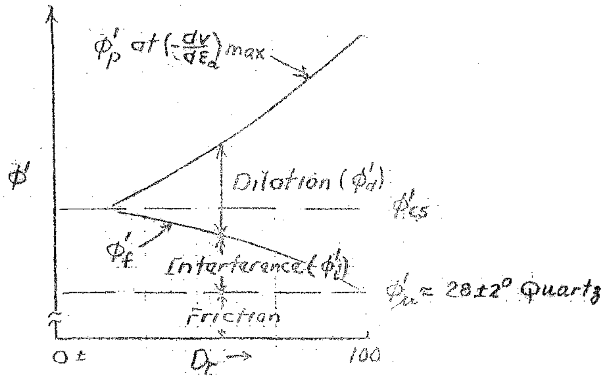
- "Interlocking" also results in which shows that sand particles cannot move in a straight line, but must go around each other
- At const. vol. $(dv/dE_a = o)$ $\tan Ø^1{}_{cs} = \pi/2 \tan Ø^1{}_u$

($\pi/2$ = ½ circumference/diameter) (really not that simple)

SUMMARY/CONCLUSION

From the variation of (Part 2) of the Common Stress-Strain Test which is tied to the Methodology Procedure and the proposed Stress-Strain conditions on its relativity with (Part 3) of the Application of Stress-Strain cohesionless angle of grain soils individual that gives the development of material for remediation. This remediation practice is to help in soil improvement and consequently introduced a new era for soil improvement both laboratory and field practices.

1. To this end, the Mohr-Coulomb failure help in defining the limiting conditions for the stress-strain condition of the soil. Very dense: $Ø^IP = Ø^I{}_u + Ø^I{}_d$
2.At critical state and Very loose: $Ø^I{}_P = Ø^I{}_{cs} = Ø^I{}_u + Ø^I{}_i$
3. Intermediate: $Ø^I{}_P = Ø^I{}_u + Ø^I{}_i + Ø^I{}_d$ L & W
"Interlocking"



Note: $Ø^I$ calculated from measured $Ø^I{}_p$ (I.e R_{max}) & max.$(-dv/dE_a)$

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