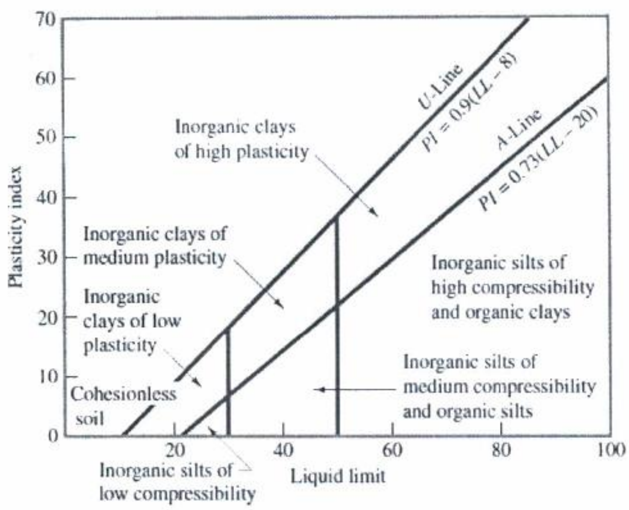
B-43

MATERY OF SOIL MECHANIS -1

(MODUL-1)

MEKANIKA TANAH 1



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1 st Edition April 2012

MATERI :

Sejarah Tanah

Sifat – Sifat Indeks

Klasifikasi Tanah

Tegangan Dalam Tanah

Proses Pembentukan Tanah

BATUAN: bagian dari kerak bumi yang mengandung satu macam atau lebih mineral yang terikat sangat kuat...
Berdasarkan proses pembentukannya batuan dapat dikategorikan sebagai:

- Batuan Beku (Igneous Rock)
  Contoh: granite, andesite, basalt
- Batuan Endapan (Sedimentary Rock)
  Contoh: claystone, siltstone, sandstone, shales, limestone, coal
- Batuan Metamorf (Metamorphic Rock)
  Contoh: gneiss, quartzite, slate, marble

Tanah: hasil pelapukan batuan berupa kumpulan butiran-butiran partikel dengan ikatan antar butir yang lemah...

Pembagian Kelompok Tanah

Berdasarkan Proses Transportasi:

- Tanah **Residual**
- Tanah **Colluvial**
- Tanah **Endapan Air** (Alluvial Soils)
- Tanah **Endapan Angin** (Eolian Soils)
- Tanah **Endapan Sungai Es** (Glacial Soils)

Tanah Residual: hasil pelapukan batuan dasar dan masih berada di tempat asalnya.

Contoh: **Tanah merah/tanah laterit hasil dekomposisi batuan di daerah tropis. Tanah merah lebih banyak mengandung lempung kaolinite, tidak begitu aktif, dan non-swelling.**

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Tanah Colluvial: terbentuk dari tanah yang berpindah dari tempat asalnya akibat gaya gravitasi pada saat kejadian keruntuhan lereng

Tanah Alluvial (endapan air): terbentuk dari tanah yang berpindah dari tempat asalnya akibat terbawa air yang mengalir

- Fluvial: tanah deposit endapan sungai
- Lacustrine: tanah deposit endapan danau
- Coastal: tanah deposit endapan di tepi pantai
- Marine deposits: offshore deposits

Tanah Eolian (endapan angin): tanah deposit yang ditransportasikan oleh angin

- Sand dunes
- Loess (silty)
- Volcanic dust

Tanah Glacial: tanah yang terbentuk karena terbawa oleh perpindahan/gerakan massa es dan oleh air dari lelehan massa es tersebut

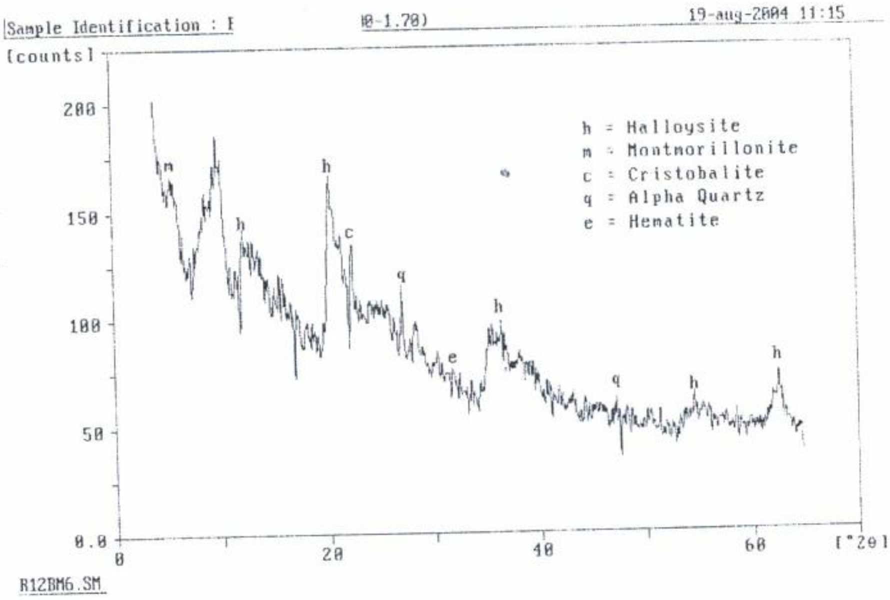
- TILL: **tanah endapan yang terbawa langsung oleh massa es**
- OUTWASH: **tanah yang diendapkan oleh aliran air lelehan massa es**
-

Tanah Khusus:

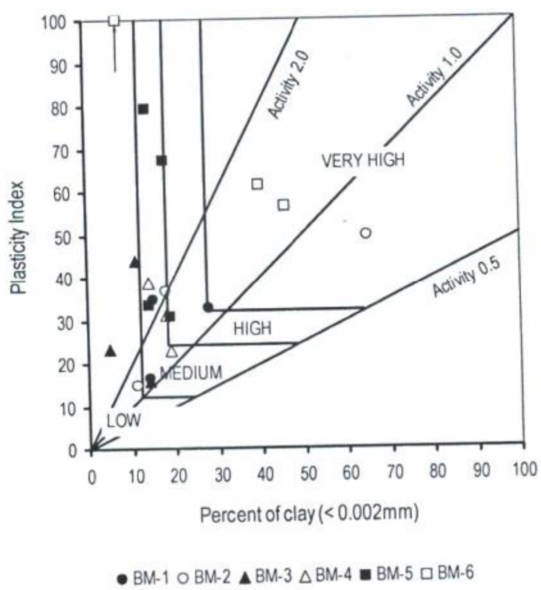
- Tanah Expansive
  Tanah yang berpotensi mengembang (peningkatan volume) akibat terjadi peningkatan kadar air dan menyusut bila kadar air berkurang. Clay shales dan tanah lempung montmorillonite

- Tanah Colapsible

  Tanah yang berpotensi mengalami pengurangan kuat geser yang besar bila terjadi peningkatan kadar air tanpa adanya perubahan beban luar.



Grafik Hasil X-Ray Diffraction



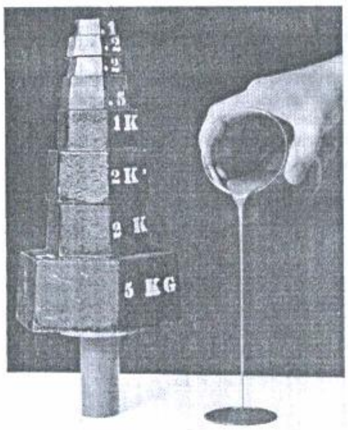
Department of the Navy, 1982, Soil Mechanic, Navfac DM-7.1

Pembagian Kelompok Tanah

Tanah Khusus:

- Quick Clay:

Tanah yang sangat peka terhadap gangguan. Apabila terganggu kekuatannya berkurang drastis. Kadar kepekaan adalah perbandingan antara kuat geser tanah asli dengan kuat geser tanah terganggu



Sensitifitas: $S_t = \dfrac{q_{u,\text{tidak terganggu}}}{q_{u,\text{terganggu}}}$

St	Derajat Kepekaan	
<4	Tidak sensitif	Kebanyakan lempung pada umumnya
4 < St < 8	Sensitif	
> 8	Sangat sensitif	

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Pembagian Kelompok Tanah

- **Tanah Organik**:

Tanah yang banyak mengandung komponen organik. Kuat geser rendah dan memiliki kompresibilitas yang besar

Mengandung massa kayu berserat, berwarna gelap hitam, berbau tumbuhan yang membusuk

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Karakteristik Tanah

Sistem Particulate:

Massa tanah terdiri dari partikel-partikel yang umumnya tidak terikat kuat satu dan lainnya. Pergeseran antar partikel menjadi tidak linear dan tidak dapat kembali ke bentuk asal

Sistem Multi Fase:

- **Zat padat**
- **Zat cair atau gas di dalam pori antar partikel (biasanya air dan udara)**

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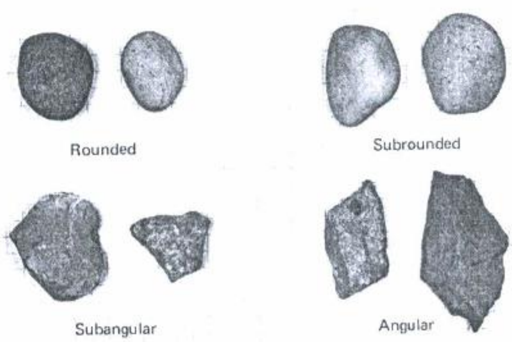
Bentuk, ukuran, tekstur, susunan, dan struktur partikel tanah:

Tanah Berbutir Kasar (ukuran > 0.06 mm):



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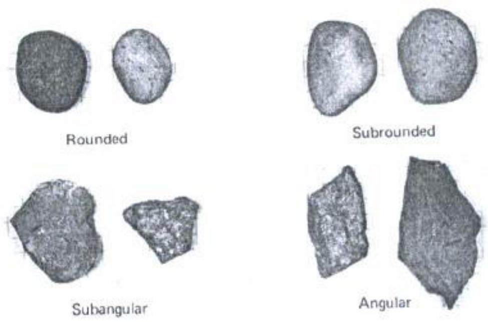

Bentuk partikel	Penyebab
Angular	**Pecahan batuan akibat pengaruh lingkungan atau pelapukan**
Subangular	**Pecahan batuan dengan bagian permukaan yang halus akibat transportasi**
Subrounded	**Permukaan umumnya halus karena sudah ditransportasikan cukup jauh**
Rounded	**Permukaan halus dan bulat karena sudah bertahun-tahun ditransportasikan**

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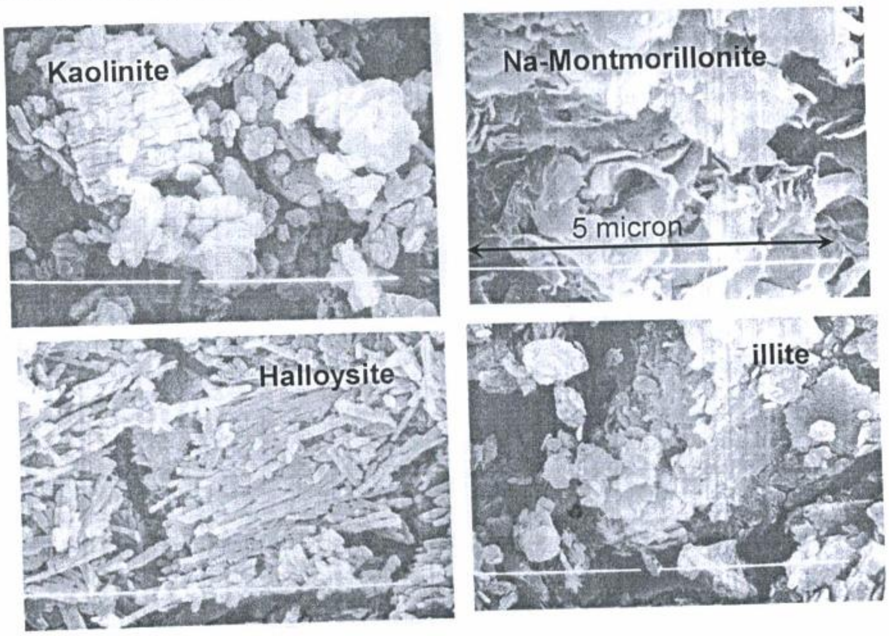
Karakteristik Tanah

Tanah Berbutir Halus (ukuran < 0.06 mm):



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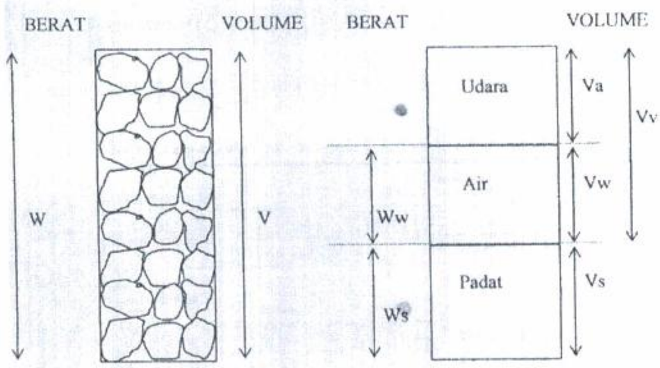
II. SIFAT-SIFAT INDEKS

INDEX PROPERTIES
Tanah dalam 3 Fase



W = Ws + Ww
V = Vs + Vw + Va
Vv = Va + Vw

Dimana : W = Berat total tanah
Ws = berat butiran tanah ; Ww= Berat air
Vs = vol.butir ; Vw = vol. air ; Va = vol.udara

DIFINISI :

Angka pori (e) = Vv / Vs

Porositas (n) = Vv / V

Berat isi tanah (γ) = W / V

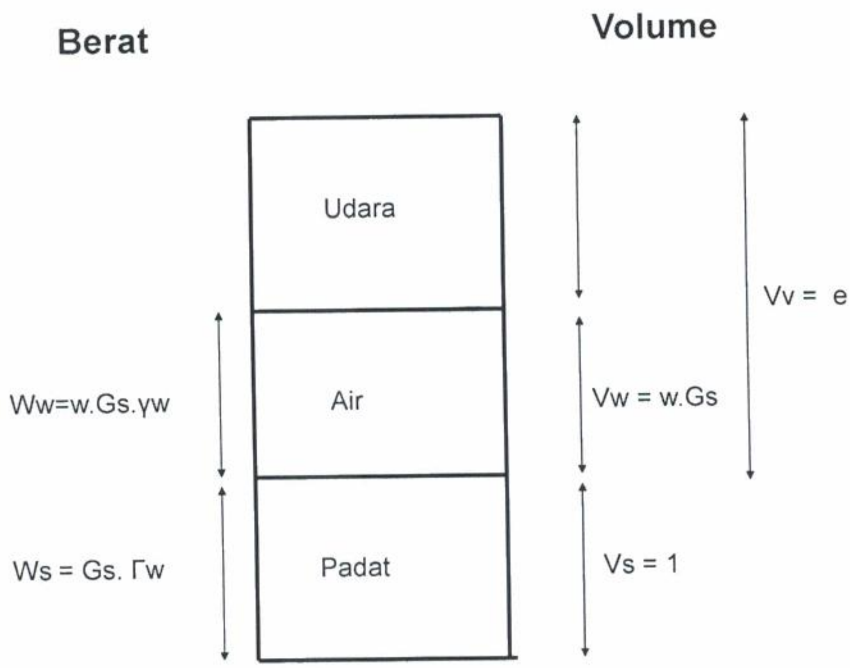
Berat isi kering (γd) = Ws/V

Berat jenis tanah /Specific Gravity (Gs) :

Ws /Vs. γw

Drajat kejenuhan (Sr) :

Vw/Vv x 100 %



Relationships among Unit Weight, Void Ratio, Moisture Content, and Specific Gravity

To obtain a relationship among unit weight (or density), void ratio, and moisture content, consider a volume of soil in which the volume of the soil solids is 1, as shown in Figure 3.2. If the volume of the soil solids is 1, then the volume of voids is numerically equal to the void ratio, e [from Eq. (3.3)]. The weights of soil solids and water can be given as

$$W_s = G_s\gamma_w$$

$$W_w = wW_s = wG_s\gamma_w$$

where

G_s = specific gravity of soil solids
w = moisture content
γ_w = unit weight of water

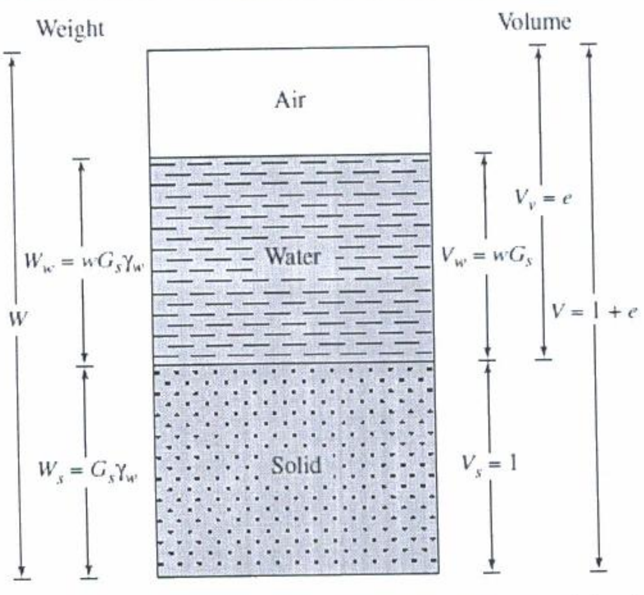


Figure 3.2 Three separate phases of a soil element with volume of soil solids equal to 1

The unit weight of water is 9.81 kN/m^3. Now, using the definitions of unit weight and dry unit weight [Eqs. (3.9) and (3.11)], we can write

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{G_s\gamma_w + wG_s\gamma_w}{1+e} = \frac{(1+w)G_s\gamma_w}{1+e} \tag{3.17}$$

and

$$\gamma_d = \frac{W_s}{V} = \frac{G_s\gamma_w}{1+e} \tag{3.18}$$

Since the weight of water in the soil element under consideration is $wG_s\gamma_w$, the volume occupied by it is

$$V_w = \frac{W_w}{\gamma_w} = \frac{wG_s\gamma_w}{\gamma_w} = wG_s$$

Hence, from the definition of degree of saturation [Eq. (3.5)], we have

$$S = \frac{V_w}{V_v} = \frac{wG_s}{e}$$

or

$$Se = wG_s \tag{3.19}$$

This is a very useful equation for solving problems involving three-phase relationships.

If the soil sample is *saturated*—that is, the void spaces are completely filled with water (Figure 3.3)—the relationship for saturated unit weight can be derived in a similar manner:

$$\gamma_{sat} = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{G_s\gamma_w + e\gamma_w}{1 + e} = \frac{(G_s + e)\gamma_w}{1 + e} \tag{3.20}$$

where γ_{sat} = saturated unit weight of soil.

As mentioned before, because it is convenient to work with densities, the following equations [similar to the unit-weight relationships given in Eqs. (3.17), (3.18), and (3.20)] are useful:

$$\text{Density} = \rho = \frac{(1 + w)G_s\rho_w}{1 + e} \tag{3.21}$$

$$\text{Dry density} = \rho_d = \frac{G_s\rho_w}{1 + e} \tag{3.22}$$

$$\text{Saturated density} = \rho_{sat} = \frac{(G_s + e)\rho_w}{1 + e} \tag{3.23}$$

where ρ_w = density of water = 1000 kg/m^3.

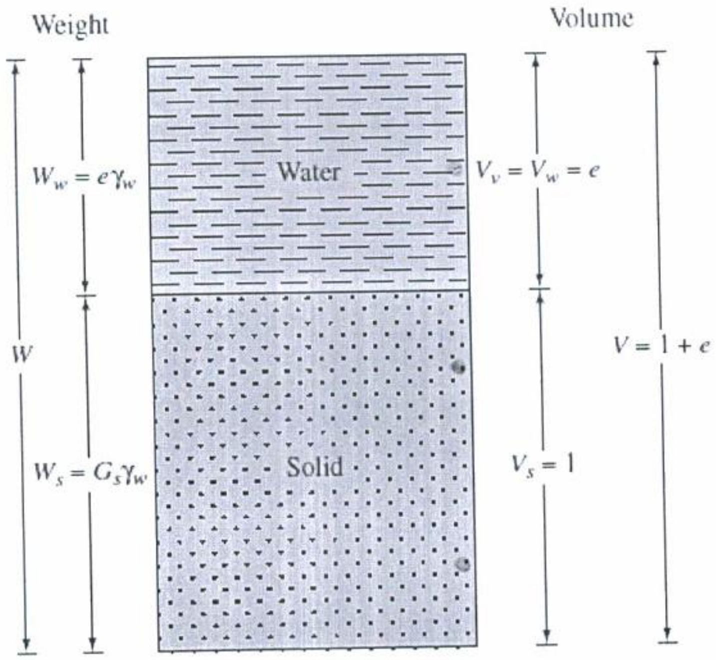


Table 3.1 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

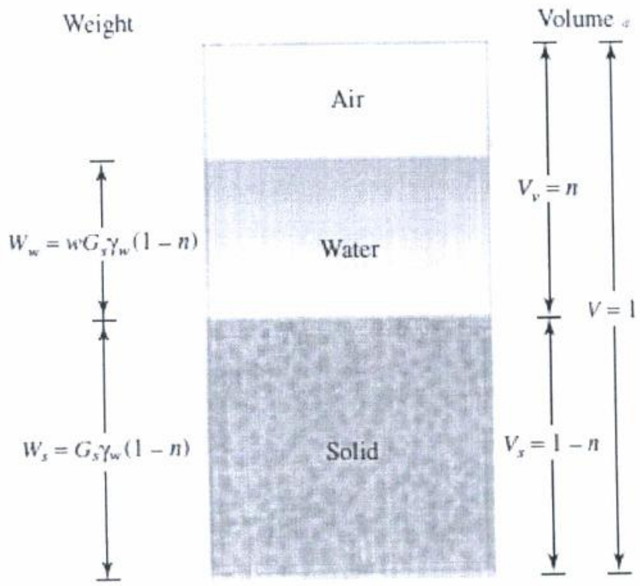
Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d (kN/m^3)
Loose uniform sand	0.8	30	14.5
Dense uniform sand	0.45	16	18
Loose angular-grained silty sand	0.65	25	16
Dense angular-grained silty sand	0.4	15	19
Stiff clay	0.6	21	17
Soft clay	0.9–1.4	30–50	11.5–14.5
Loess	0.9	25	13.5
Soft organic clay	2.5–3.2	90–120	6–8
Glacial till	0.3	10	21

Some typical values of void ratio, moisture content in a saturated condition, and dry unit weight for soils in a natural state are given in Table 3.1.

Relationships among Unit Weight, Porosity, and Moisture Content

The relationships among *unit weight, porosity*, and *moisture content* can be developed in a manner similar to that presented in the preceding section. Consider a soil that has a total volume equal to one, as shown in Figure 3.4. From Eq. (3.4),

$$n = \frac{V_v}{V}$$



If V is equal to 1, then V_v is equal to n, so $V_s = 1 - n$. The weight of soil solids (W_s) and the weight of water (W_w) can then be expressed as follows:

$$W_s = G_s\gamma_w(1 - n) \tag{3.24}$$

$$W_w = wW_s = wG_s\gamma_w(1 - n) \tag{3.25}$$

So, the dry unit weight equals

$$\gamma_d = \frac{W_s}{V} = \frac{G_s\gamma_w(1 - n)}{1} = G_s\gamma_w(1 - n) \tag{3.26}$$

The moist unit weight equals

$$\gamma = \frac{W_s + W_w}{V} = G_s\gamma_w(1 - n)(1 + w) \tag{3.27}$$

Figure 3.5 shows a soil sample that is saturated and has $V = 1$. According to this figure,

$$\gamma_{sat} = \frac{W_s + W_w}{V} = \frac{(1 - n)G_s\gamma_w + n\gamma_w}{1} = [(1 - n)G_s + n]\gamma_w \tag{3.28}$$

The moisture content of a saturated soil sample can be expressed as

$$w = \frac{W_w}{W_s} = \frac{n\gamma_w}{(1 - n)\gamma_w G_s} = \frac{n}{(1 - n)G_s} \tag{3.29}$$

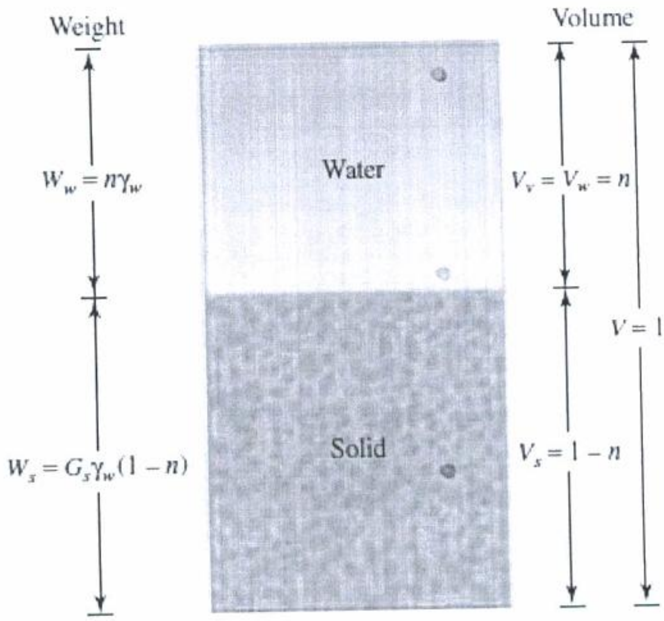


Figure 3.5 Saturated soil element with total volume equal to 1

Relative Density

The term *relative density* is commonly used to indicate the *in situ* denseness or looseness of granular soil. It is defined as

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \tag{3.30}$$

where

D_r = relative density, usually given as a percentage
e = *in situ* void ratio of the soil
e_{max} = void ratio of the soil in the loosest condition
e_{min} = void ratio of the soil in the densest condition

The values of D_r may vary from a minimum of 0 for very loose soil, to a maximum of 1 for very dense soil. Soils engineers qualitatively describe the granular soil deposits according to their relative densities, as shown in Table 3.2.

By using the definition of dry unit weight given in Eq. (3.18), we can also express relative density in terms of maximum and minimum possible dry unit weights. Thus,

$$D_r = \frac{\left[\dfrac{1}{\gamma_{d(min)}}\right] - \left[\dfrac{1}{\gamma_d}\right]}{\left[\dfrac{1}{\gamma_{d(min)}}\right] - \left[\dfrac{1}{\gamma_{d(max)}}\right]} = \left[\frac{\gamma_d - \gamma_{d(min)}}{\gamma_{d(max)} - \gamma_{d(min)}}\right]\left[\frac{\gamma_{d(max)}}{\gamma_d}\right] \tag{3.31}$$

Table 3.2 Qualitative description of granular soil deposits

Relative density (%)	Description of soil deposit
0–15	Very loose
15–50	Loose
50–70	Medium
70–85	Dense
85–100	Very dense

where

$\gamma_{d(\min)}$ = dry unit weight in the loosest condition (at a void ratio of $e_{\max}$)

γ_d = *in situ* dry unit weight (at a void ratio of e)

$\gamma_{d(\max)}$ = dry unit weight in the densest condition (at a void ratio of $e_{\min}$)

Cubrinovski and Ishihara (2002) studied the variation of $e_{\max}$ and $e_{\min}$ for a very large number of soils. Based on the best-fit linear-regression lines, they provided the following relationships.

- Clean sand ($F_c = 0$ to 5%)

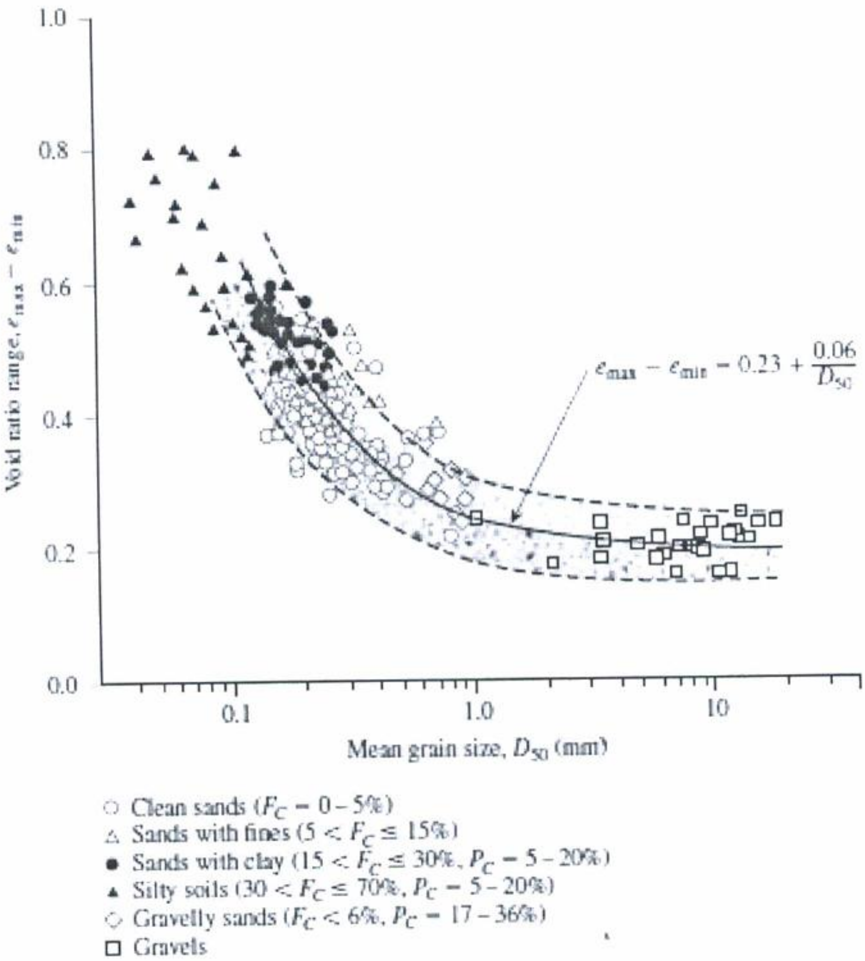
$$e_{\max} = 0.072 + 1.53\, e_{\min} \tag{3.32}$$

- Sand with fines ($5 < F_c \leq 15\%$)

$$e_{\max} = 0.25 + 1.37\, e_{\min} \tag{3.33}$$

- Sand with fines and clay ($15 < F_c \leq 30\%$; $P_c = 5$ to 20%)

$$e_{\max} = 0.44 + 1.21\, e_{\min} \tag{3.34}$$



- Silty soils ($30 < F_c \leq 70\%$; $P_c = 5$ to 20%)

$$e_{max} = 0.44 + 1.32\, e_{min} \tag{3.35}$$

where

F_c = fine fraction for which grain size is smaller than 0.075 mm
P_c = clay-size fraction (< 0.005 mm)

Figure 3.7 shows a plot of $e_{max} - e_{min}$ versus the mean grain size (D_{50}) for a number of soils (Cubrinovski and Ishihara, 1999 and 2002). From this figure, the average plot for sandy and gravelly soils can be given by the relationship

$$e_{max} - e_{min} = 0.23 + \frac{0.06}{D_{50}\,(\text{mm})} \tag{3.36}$$

KONSISTENSI TANAH

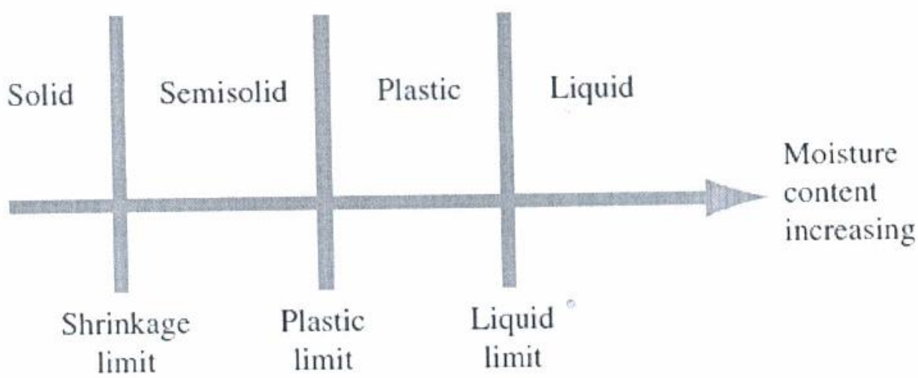
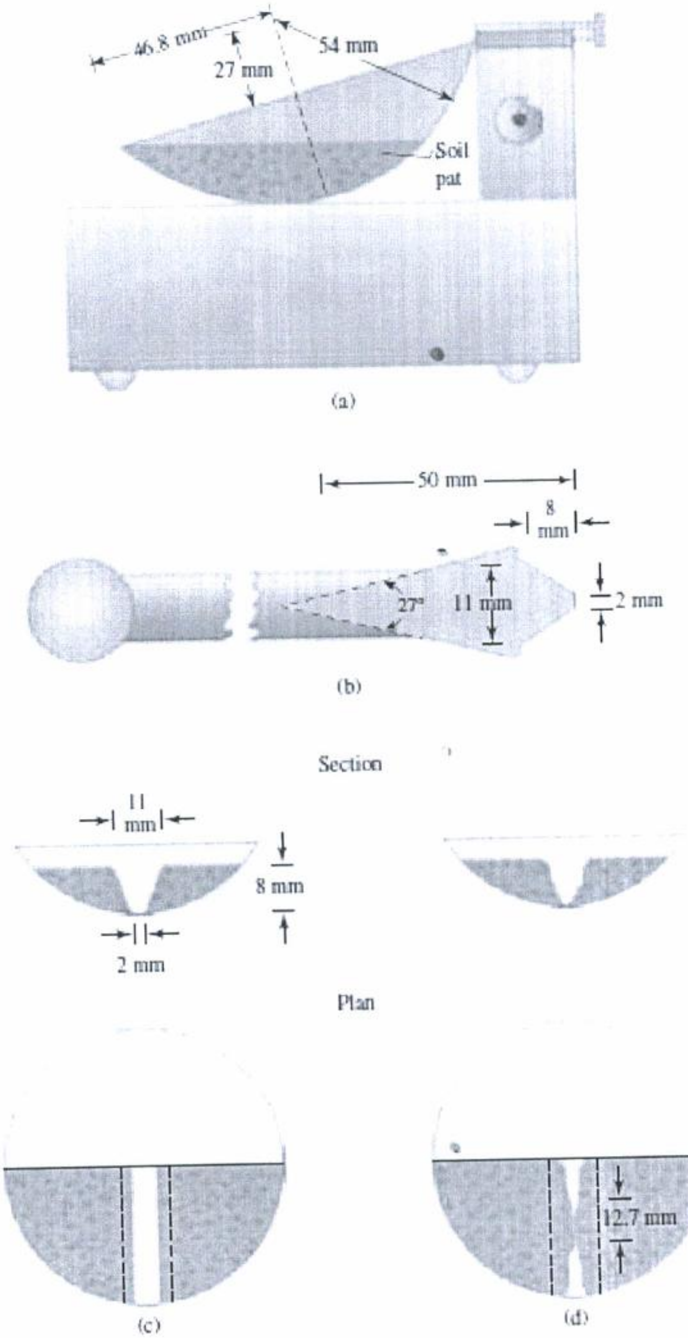


Figure 3.8 Atterberg limits

LIQUID LIMITS



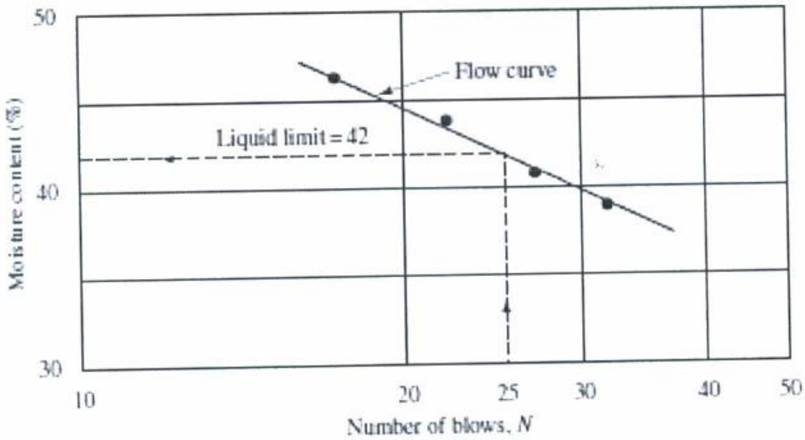


Figure 3.10 Flow curve for liquid limit determination of a silty clay

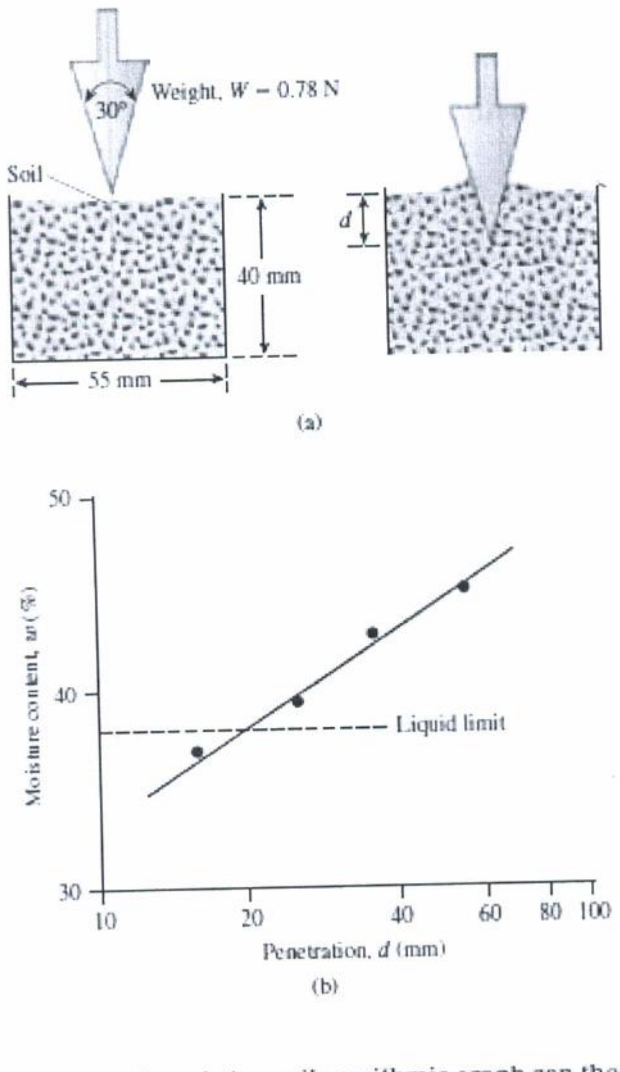


Figure 3.11
(a) Fall cone test (b) plot of moisture content vs. cone penetration for determination of liquid limit

penetration, d. A semilogarithmic graph can then be plotted with moisture content (w) versus cone penetration d. The plot results in a straight line. The moisture content corresponding to $d = 20$ mm is the liquid limit (Figure 3.11b).

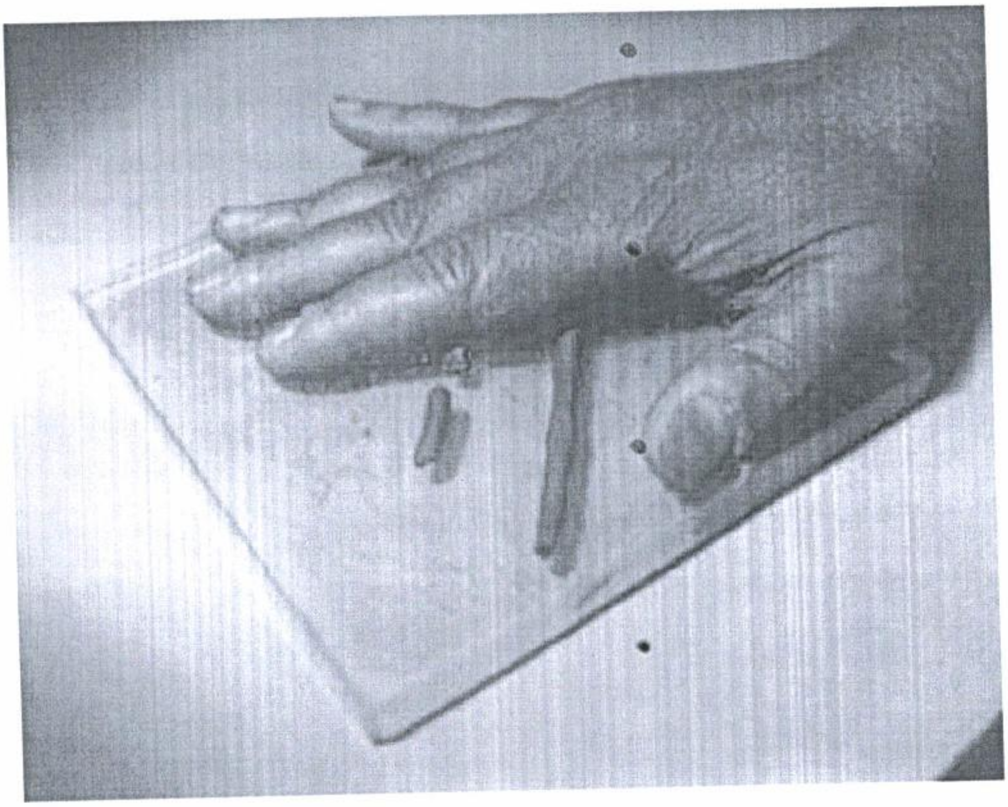
PLASTIC LIMITS

The *plastic limit* is defined as the moisture content, in percent, at which the soil when rolled into threads of 3.2 mm in diameter, crumbles. The plastic limit is the lower limit of the plastic stage of soil. The test is simple and is performed by repeated rollings by hand of an ellipsoidal size soil mass on a ground glass plate (Figure 3.12).

The *plasticity index* (*PI*) is the difference between the liquid limit and plastic limit of a soil, or

$$PI = LL - PL \tag{3.37}$$

The procedure for the plastic limit test is given in ASTM Test Designation D-4318.



SHRINKAGE LIMITS

Soil mass shrinks as moisture is gradually lost from the soil. With continuous loss of moisture, a stage of equilibrium is reached at which point more loss of moisture will result in no further volume change (Figure 3.14). The moisture content, in percent, at which the volume change of the soil mass ceases is defined as the *shrinkage limit*.

Shrinkage limit tests (ASTM Test Designation D-427) are performed in the laboratory with a porcelain dish about 44 mm in diameter and about 13 mm in height. The inside of the dish is coated with petroleum jelly and is then filled completely with wet soil. Excess soil standing above the edge of the dish is struck off with a straightedge. The mass of the wet soil inside the dish is recorded. The soil pat in the dish is then oven dried. The volume of the oven-dried soil pat is determined by the displacement of mercury. Figure 3.15 shows a photograph of the equipment needed for the shrinkage limit test. Because handling mercury can be hazardous, ASTM Test Designation D-4943 describes a method of dipping the oven-dried soil pat in a pot of melted wax. The wax-coated soil pat is then cooled. Its volume is determined by submerging it in water.

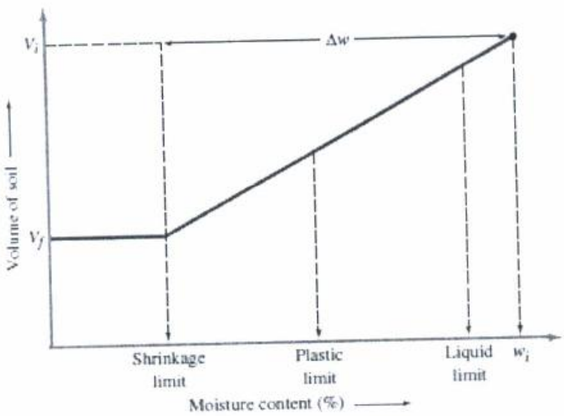


Figure 3.14 Definition of shrinkage limit

Referring to Figure 3.14, we can determine the shrinkage limit in the following manner:

$$SL = w_i\,(\%) - \Delta w\,(\%) \qquad (3.38)$$

Figure 3.15 Equipment for shrinkage limit test (Courtesy of Braja Das)

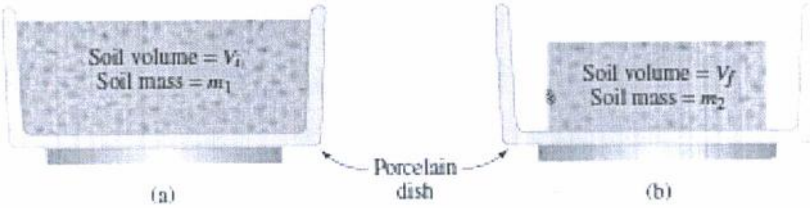


Figure 3.16 Shrinkage limit test: (a) soil pat before drying; (b) soil pat after drying

where

w_i = initial moisture content when the soil is placed in the shrinkage limit dish
Δw = change in moisture content (that is, between the initial moisture content and the moisture content at shrinkage limit)

However,

$$w_i(\%) = \frac{m_1 - m_2}{m_2} \times 100 \tag{3.39}$$

where

m_1 = mass of the wet soil pat in the dish at the beginning of the test (g)
m_2 = mass of the dry soil pat (g) (see Figure 3.16)

Also,

$$\Delta w(\%) = \frac{(V_i - V_f)\rho_w}{m_2} \times 100 \tag{3.40}$$

where

V_i = initial volume of the wet soil pat (that is, inside volume of the dish, cm^3)
V_f = volume of the oven-dried soil pat (cm^3)
ρ_w = density of water (g/cm^3)

Now, combining Eqs. (3.38), (3.39), and (3.40), we have

$$SL = \left(\frac{m_1 - m_2}{m_2}\right)(100) - \left[\frac{(V_i - V_f)\rho_w}{m_2}\right](100) \tag{3.41}$$

Table 3.3 Activity of clay minerals

Mineral	Activity, A
Smectites	1–7
Illite	0.5–1
Kaolinite	0.5
Halloysite ($2H_2O$)	0.5
Holloysite ($4H_2O$)	0.1
Attapulgite	0.5–1.2
Allophane	0.5–1.2

defined a quantity called *activity*, which is the slope of the line correlating PI and percent finer than 2 μ. This activity may be expressed as

$$A = \frac{PI}{\text{percent of clay-size fraction, by weight}} \tag{3.42}$$

where A = activity. Activity is used as an index for identifying the swelling potential of clay soils. Typical values of activities for various clay minerals are listed in Table 3.4 (Mitchell, 1976).

Seed, Woodward, and Lundgren (1964) studied the plastic property of several artificially prepared mixtures of sand and clay. They concluded that although the relationship of the plasticity index to the percent of clay-size fraction is linear, as observed by Skempton, the line may not always pass through the origin. They showed that the relationship of the plasticity index to the percent of clay-size fraction present in a soil can be represented by two straight lines. This relationship is shown qualitatively in Figure 3.17. For clay-size fractions greater than 40%, the straight line passes through the origin when it is projected back.

Liquidity Index

The relative consistency of a cohesive soil in the natural state can be defined by a ratio called the *liquidity index* (LI):

$$LI = \frac{w - PL}{LL - PL} \tag{3.43}$$

where w = *in situ* moisture content of soil.

The *in situ* moisture content of a sensitive clay may be greater than the liquid limit. In that case,

$$LI > 1$$

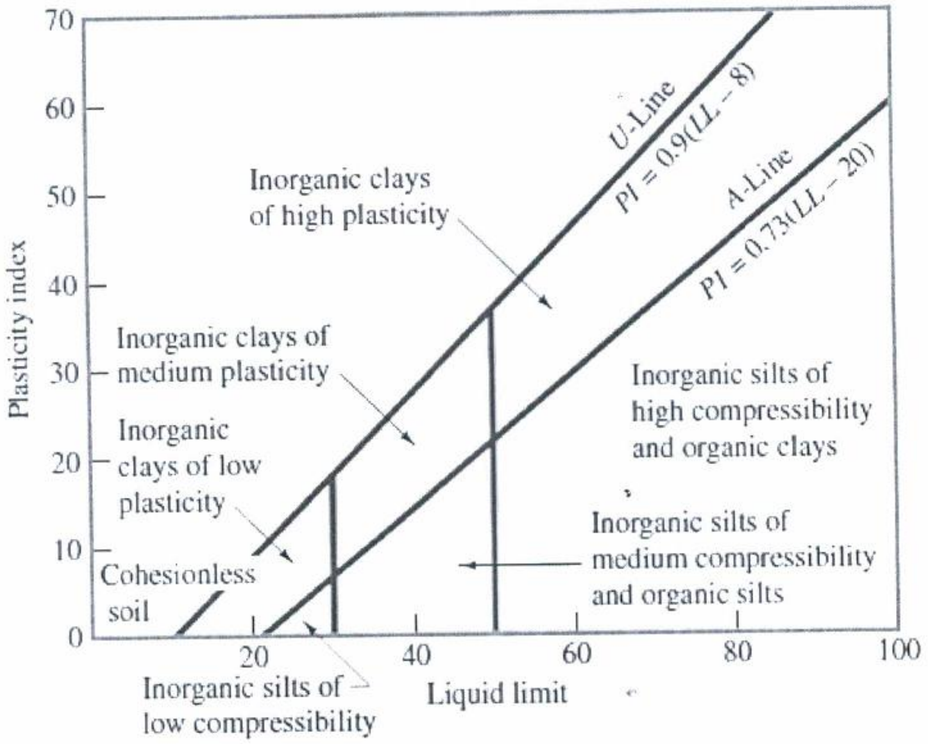
These soils, when remolded, can be transformed into a viscous form to flow like a liquid.

Soil deposits that are heavily overconsolidated may have a natural moisture content less than the plastic limit. In that case,

$$LI < 1$$

The values of the liquidity index for some of these soils may be negative.

Plasticity Chart, untuk tanah berbutir halus



KLASIFIKASI TANAH

KLASIFIKASI TANAH CARA AASHTO (AASHTO Methode)

General classification	**Granular materials (35% or less of total sample passing No. 200)**						
	A-1			*A-2*			
Group classification	*A-1-a*	*A-1-b*	*A-3*	*A-2-4*	*A-2-5*	*A-2-6*	*A-2-7*
Sieve analysis (percent passing)							
No. 10	50 max.						
No. 40	30 max.	50 max.	51 min.				
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.
Characteristics of fraction passing No. 40							
Liquid limit				40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand			
General subgrade rating	Excellent to good						

General classification	**Silt-clay materials (more than 35% of total sample passing No. 200)**			
Group classification	*A-4*	*A-5*	*A-6*	*A-7* *A-7-5** *A-7-6*†
Sieve analysis (percent passing)				
No. 10				
No. 40				
No. 200	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Silty soils		Clayey soils	
General subgrade rating	Fair to poor			

*For A-7-5, $PI \leq LL - 30$

†For A-7-6, $PI > LL - 30$

Plasticity Chart untuk tanah berbutir halus dengan cara AASHTO Methode

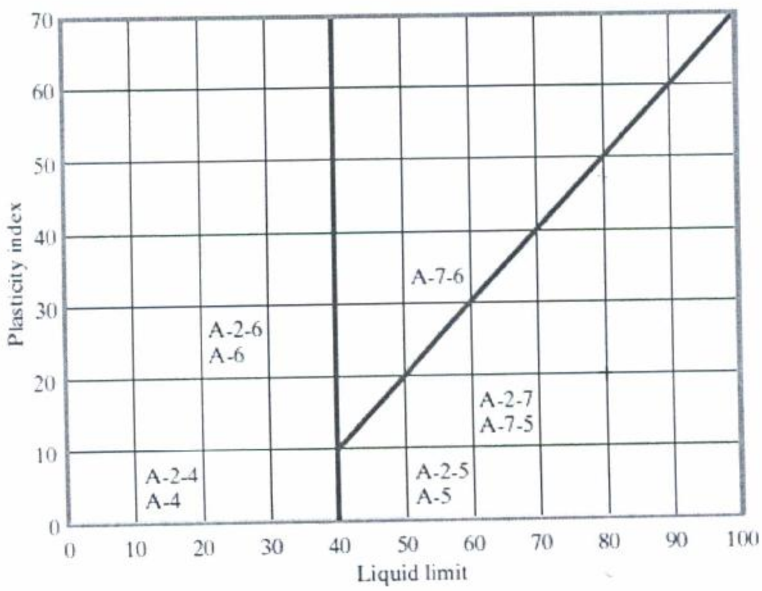


Figure 3.19 Range of liquid limit and plasticity index for soils in groups A-2, A-4, A-5, A-6, and A-7

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

Table 3.5 Unified Soil Classification System (Based on Material Passing 75-mm Sieve)

Criteria for Assigning Group Symbols				Group Symbol
Coarse-Grained Soils More than 50% of retained on No. 200 sieve	**Gravels** More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[c]	GW
			$C_u < 4$ and/or $1 > C_c > 3$[c]	GP
		Gravels with Fines More than 12% fines[a,d]	$PI < 4$ or plots below "A" line (Figure 3.20)	GM
			$PI > 7$ and plots on or above "A" line (Figure 3.20)	GC
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[b]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[c]	SW
			$C_u < 6$ and/or $1 > C_c > 3$[c]	SP
		Sands with Fines More than 12% fines[b,d]	$PI < 4$ or plots below "A" line (Figure 3.20)	SM
			$PI > 7$ and plots on or above "A" line (Figure 3.20)	SC
Fine-Grained Soils 50% or more passes No. 200 sieve	**Silts and Clays** Liquid limit less than 50	Inorganic	$PI > 7$ and plots on or above "A" line (Figure 3.20)[e]	CL
			$PI < 4$ or plots below "A" line (Figure 3.20)[e]	ML
		Organic	$\dfrac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$; see Figure 3.20; OL zone	OL
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line (Figure 3.20)	CH
			PI plots below "A" line (Figure 3.20)	MH
		Organic	$\dfrac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$; see Figure 3.20; OH zone	OH
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor			Pt

[a] Gravels with 5 to 12% fine require dual symbols: GW-GM, GW-GC, GP-GM, GP-GC.
[b] Sands with 5 to 12% fines require dual symbols: SW-SM, SW-SC, SP-SM, SP-SC.
[c] $C_u = \dfrac{D_{60}}{D_{10}}$; $C_c = \dfrac{(D_{30})^2}{D_{60} \times D_{10}}$
[d] If $4 \leq PI \leq 7$ and plots in the hatched area in Figure 3.16, use dual symbol GC-GM or SC-SM.
[e] If $4 \leq PI \leq 7$ and plots in the hatched area in Figure 3.16, use dual symbol CL-ML.

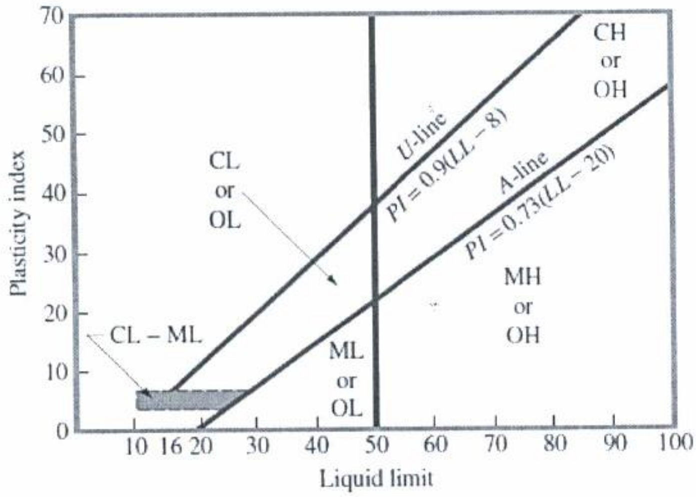


Figure 3.20 Plasticity chart

Group Symbol		Group Name
GW	<15% sand	Well-graded gravel
	≥15% sand	Well-graded gravel with sand
GP	<15% sand	Poorly graded gravel
	≥15% sand	Poorly graded gravel with sand
GW-GM	<15% sand	Well-graded gravel with silt
	≥15% sand	Well-graded gravel with silt and sand
GW-GC	<15% sand	Well-graded gravel with clay (or silty clay)
	≥15% sand	Well-graded gravel with clay and sand (or silty clay and sand)
GP-GM	<15% sand	Poorly graded gravel with silt
	≥15% sand	Poorly graded gravel with silt and sand
GP-GC	<15% sand	Poorly graded gravel with clay (or silty clay)
	≥15% sand	Poorly graded gravel with clay and sand (or silty clay and sand)
GM	<15% sand	Silty gravel
	≥15% sand	Silty gravel with sand
GC	<15% sand	Clayey gravel
	≥15% sand	Clayey gravel with sand
GC-GM	<15% sand	Silty clayey gravel
	≥15% sand	Silty clayey gravel with sand
SW	<15% gravel	Well-graded sand
	≥15% gravel	Well-graded sand with gravel
SP	<15% gravel	Poorly graded sand
	≥15% gravel	Poorly graded sand with gravel
SW-SM	<15% gravel	Well-graded sand with silt
	≥15% gravel	Well-graded sand with silt and gravel
SW-SC	<15% gravel	Well-graded sand with clay (or silty clay)
	≥15% gravel	Well-graded sand with clay and gavel (or silty clay and gravel)
SP-SM	<15% gravel	Poorly graded sand with silt
	≥15% gravel	Poorly graded sand with silt and gravel
SP-SC	<15% gravel	Poorly graded sand with clay (or silty clay)
	≥15% gravel	Poorly graded sand with clay and gravel (or silty clay and gravel)
SM	<15% gravel	Silty sand
	≥15% gravel	Silty sand with gravel
SC	<15% gravel	Clayey sand
	≥15% gravel	Clayey sand with gravel
SC-SM	<15% gravel	Silty clayey sand
	≥15% gravel	Silty clayey sand with gravel

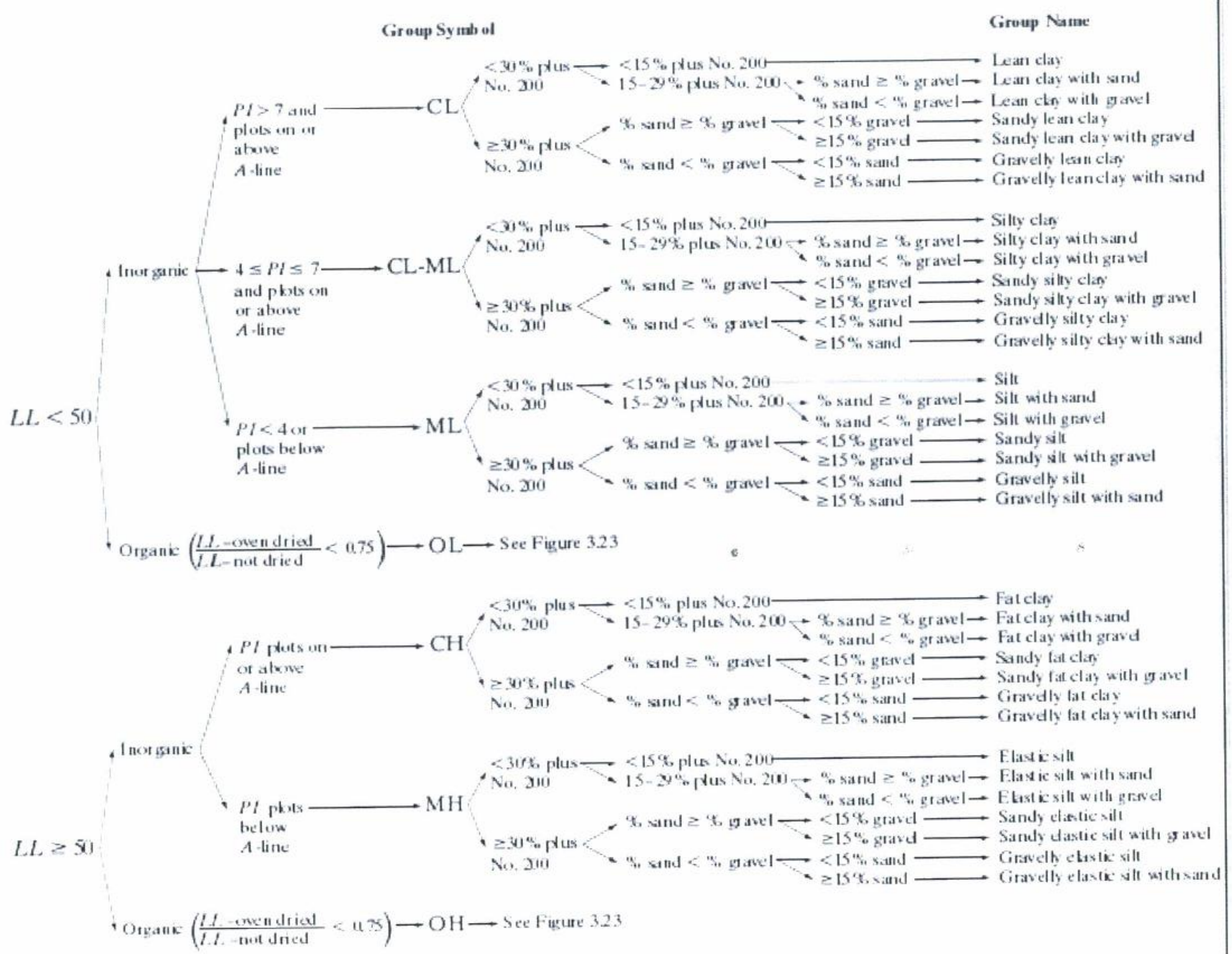


Figure 3.22 Flowchart group names for inorganic silty and clayey soils (After ASTM, 2006)

SOIL CLASSIFICATION BY U.S.D.A

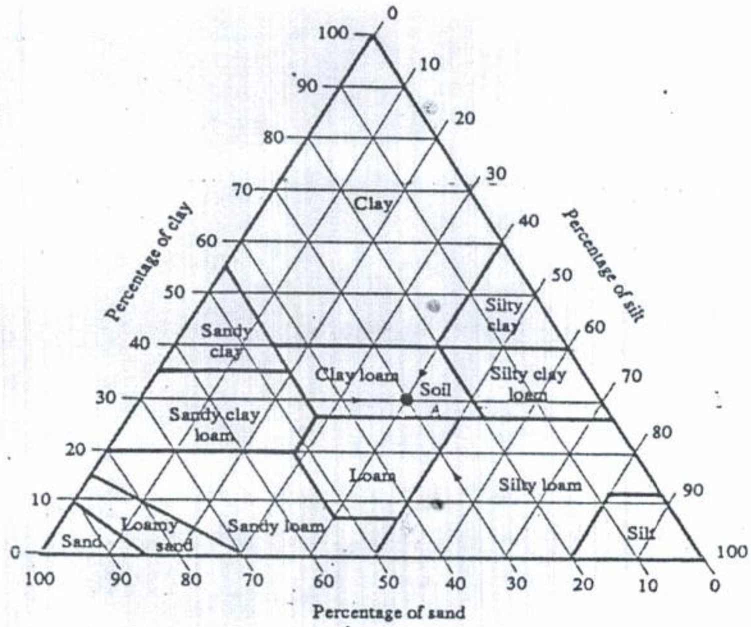


Figure 3.1 U.S. Department of Agriculture textural classification

TEGANGAN DALAM TANAH

Beban Terpusat (P Load)

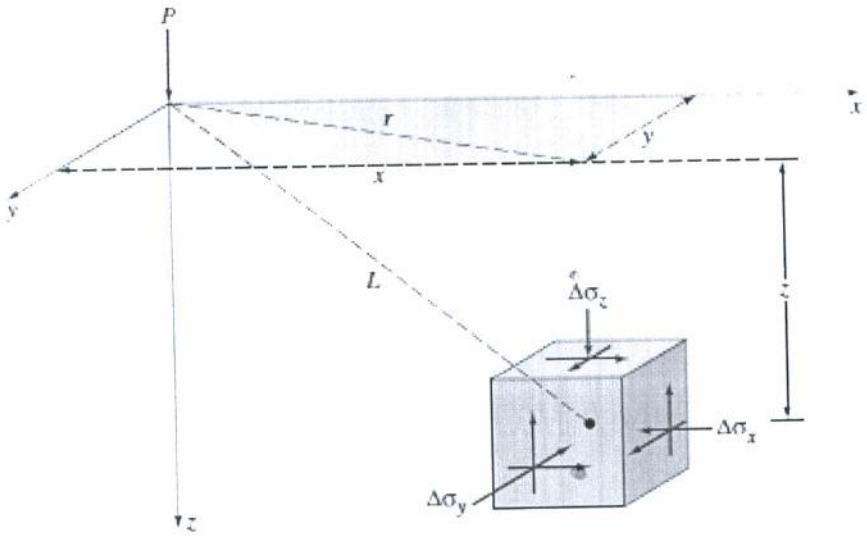


Figure 6.11 Stresses in an elastic medium caused by a point load

and

$$\Delta\sigma_z = \frac{3P}{2\pi}\frac{z^3}{L^5} = \frac{3P}{2\pi}\frac{z^3}{(r^2 + z^2)^{5/2}} \tag{6.19}$$

where

$r = \sqrt{x^2 + y^2}$

$L = \sqrt{x^2 + y^2 + z^2} = \sqrt{r^2 + z^2}$

μ_s = Poisson's ratio

Note that Eqs. (6.17) and (6.18), which are the expressions for horizontal normal stresses, are dependent on Poisson's ratio of the medium. However, the relationship for the vertical normal stress, $\Delta\sigma_z$, as given by Eq. (6.19), is independent of Poisson's ratio. The relationship for $\Delta\sigma_z$ can be rewritten in the following form:

$$\Delta\sigma_z = \frac{P}{z^2}\left\{\frac{3}{2\pi}\frac{1}{[(r/z)^2 + 1]^{5/2}}\right\} = \frac{P}{z^2}I_1 \tag{6.20}$$

$$\text{where } I_1 = \frac{3}{2\pi}\frac{1}{[(r/z)^2 + 1]^{5/2}}. \tag{6.21}$$

The variation of I_1 for various values of r/z is given in Table 6.1.

Typical values of Poisson's ratio for various soils are listed in Table 6.2.

Table 6.1 Variation of I_1 [Eq. (6.20)]

r/z	I_1	r/z	I_1
0	0.4775	0.9	0.1083
0.1	0.4657	1.0	0.0844
0.2	0.4329	1.5	0.0251
0.3	0.3849	1.75	0.0144
0.4	0.3295	2.0	0.0085
0.5	0.2733	2.5	0.0034
0.6	0.2214	3.0	0.0015
0.7	0.1762	4.0	0.0004
0.8	0.1386	5.0	0.00014

Table 6.2 Representative values of Poisson's ratio

Type of soil	Poisson's ratio, μ_s
Loose sand	0.2–0.4
Medium sand	0.25–0.4
Dense sand	0.3–0.45
Silty sand	0.2–0.4
Soft clay	0.15–0.25
Medium clay	0.2–0.5

Westergaard's Solution for Vertical Stress Due to a Point Load

Westergaard (1938) has proposed a solution for the determination of the vertical stress due to a point load P in an elastic solid medium in which there exist alternating layers with thin rigid reinforcements (Figure 6.12a). This type of assumption may be an idealization of a clay layer with thin seams of sand. For such an assumption the vertical stress increase at a point A (Figure 6.12) can be given as

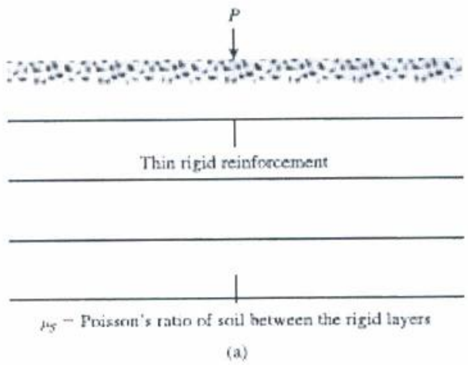
$$\Delta\sigma_z = \frac{P\eta}{2\pi z^2}\left[\frac{1}{\eta^2 + (r/z)^2}\right]^{3/2} \tag{6.22}$$

where

$$\eta = \sqrt{\frac{1 - 2\mu_s}{2 - 2\mu_s}} \tag{6.23}$$

μ_s = Poisson's ratio of the solid between the rigid reinforcements

$r = \sqrt{x^2 + y^2}$



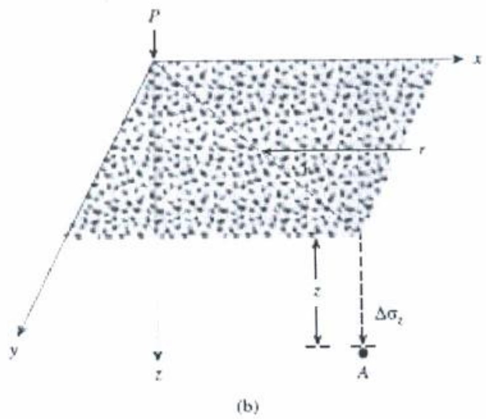


Figure 6.12 Westergaard's solution for vertical stress due to a point load

Equation (6.22) can be rewritten as

$$\Delta\sigma_z = \left(\frac{P}{z^2}\right) I_2 \tag{6.24}$$

where

$$I_2 = \frac{1}{2\pi\eta^2}\left[\left(\frac{r}{\eta z}\right)^2 + 1\right]^{-3/2} \tag{6.25}$$

Table 6.3 gives the variation of I_2 with μ_s.

In most practical problems of geotechnical engineering, Boussinesq's solution (Section 6.6) is preferred over Westergaard's solution. For that reason, *further development of stress calculation under various types of loading will use Boussinesq's solution in this chapter.*

Table 6.3 Variation of I_2 [Eq. (6.25)].

r/z	I_2		
	$\mu_s = 0$	$\mu_s = 0.2$	$\mu_s = 0.4$
0	0.3183	0.4244	0.9550
0.1	0.3090	0.4080	0.8750
0.2	0.2836	0.3646	0.6916
0.3	0.2483	0.3074	0.4997
0.4	0.2099	0.2491	0.3480
0.5	0.1733	0.1973	0.2416
0.6	0.1411	0.1547	0.1700
0.7	0.1143	0.1212	0.1221
0.8	0.0925	0.0953	0.0897
0.9	0.0751	0.0756	0.0673
1.0	0.0613	0.0605	0.0516
1.5	0.0247	0.0229	0.0173
2.0	0.0118	0.0107	0.0076
2.5	0.0064	0.0057	0.0040
3.0	0.0038	0.0034	0.0023
4.0	0.0017	0.0015	0.0010
5.0	0.0009	0.0008	0.0005

Beban Garis (Line Load)

Figure 6.13 shows a flexible line load of infinite length that has an intensity q per unit length on the surface of a semiinfinite soil mass. The vertical stress increase, $\Delta\sigma$, inside the soil mass can be determined by using the principles of the theory of elasticity, or

$$\Delta\sigma = \frac{2qz^3}{\pi(x^2 + z^2)^2} \tag{6.26}$$

The preceding equation can be rewritten as

$$\Delta\sigma = \frac{2q}{\pi z[(x/z)^2 + 1]^2}$$

or

$$\frac{\Delta\sigma}{(q/z)} = \frac{2}{\pi\left[\left(\frac{x}{z}\right)^2 + 1\right]^2} \tag{6.27}$$

Note that Eq. (6.27) is in a nondimensional form. Using this equation, we can calculate the variation of $\Delta\sigma/(q/z)$ with x/z. The variation is given in Table 6.4. The value of $\Delta\sigma$

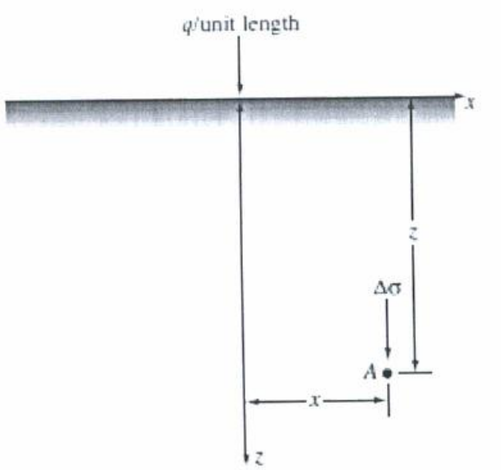


Figure 6.13 Line load over the surface of a semiinfinite soil mass

Table 6.4 Variation of $\Delta\sigma/(q/z)$ with x/z [Eq. (6.27)]

x/z	$\frac{\Delta\sigma}{q/z}$	x/z	$\frac{\Delta\sigma}{q/z}$
0	0.637	0.7	0.287
0.1	0.624	0.8	0.237
0.2	0.589	0.9	0.194
0.3	0.536	1.0	0.159
0.4	0.473	1.5	0.060
0.5	0.407	2.0	0.025
0.6	0.344	3.0	0.006

Beban Garis dengan panjang tidak terbatas

Figure 6.14 shows a line load having a length L located on the surface of a semiinfinite soil mass. The intensity of the load per unit length is q. The vertical stress increase ($\Delta\sigma$) at a point $A(0, 0, z)$ can be obtained by integration of Boussinesq's solution [Eq. (6.19)] as

$$\Delta\sigma = \frac{q}{z} I_3 \tag{6.28}$$

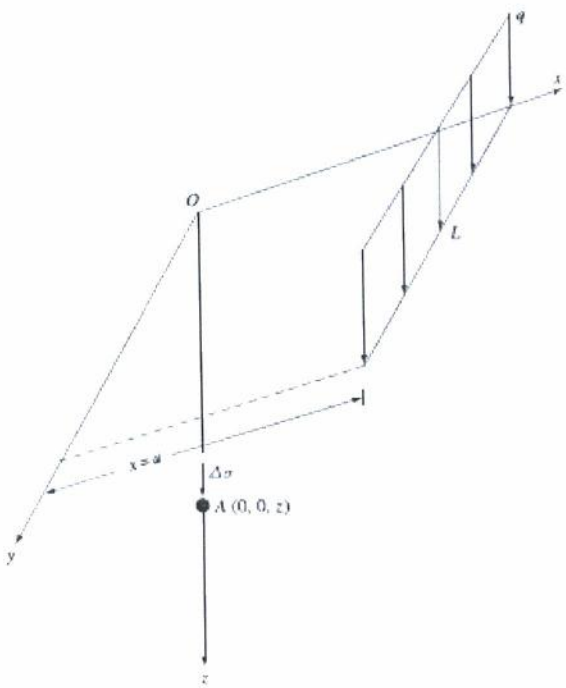


Figure 6.14 Line load of length L on the surface of a semiinfinite soil mass

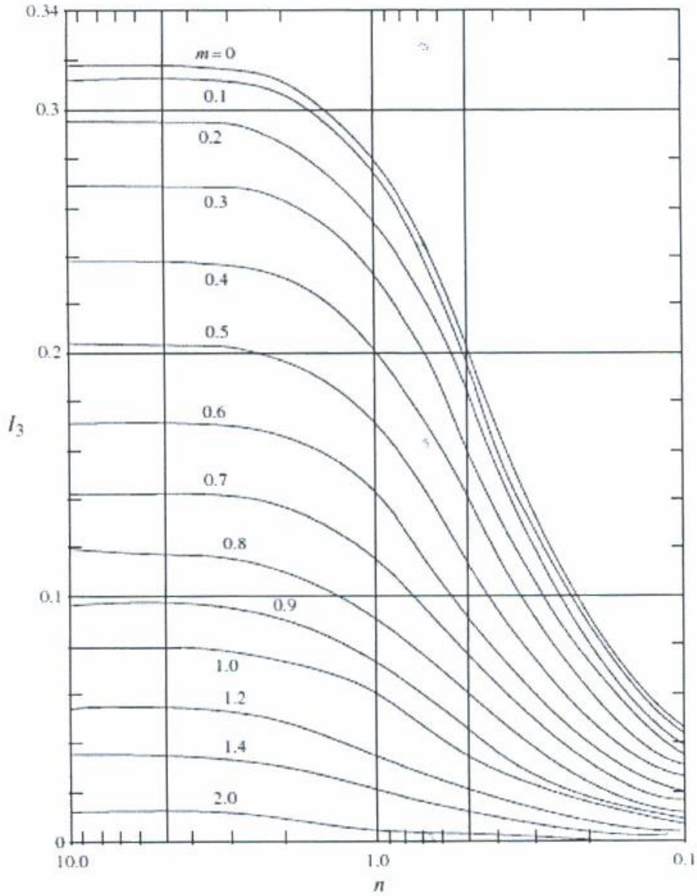
where

$$I_3 = \frac{1}{2\pi(m^2+1)^2}\left[\frac{3n}{\sqrt{m^2+n^2+1}} - \left(\frac{n}{\sqrt{(m^2+n^2+1)}}\right)^3\right] \tag{6.29}$$

$$m = \frac{a}{z} \tag{6.30}$$

$$n = \frac{L}{z} \tag{6.31}$$

Figure 6.15 shows the plot of I_3 for various values of m and n.



0.34
0.3
0.2
0.1
0
I_3
m = 0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.2
1.4
2.0
10.0
1.0
0.1
n

Beban Terbagi Rata dengan lebar terbatas dan panjang tidak terbatas.

The fundamental equation for the vertical stress increase at a point in a soil mass as the result of a line load (see Section 6.8) can be used to determine the vertical stress at a point caused by a flexible strip load of width B (Figure 6.17). Let the load per unit area of the strip shown in Figure 6.17 be equal to q. If we consider an elemental strip of width dr, the load per unit length of this strip will be equal to $q\,dr$. This elemental strip can be treated as a line load. Equation (6.26) gives the vertical stress increase, $d\sigma$, at point A inside the soil mass caused by this elemental strip load. To calculate the vertical stress increase, we need to substitute $q\,dr$ for q and $(x - r)$ for x. So

$$d\sigma = \frac{2(q\,dr)z^3}{\pi[(x-r)^2 + z^2]^2} \tag{6.32}$$

The total increase in the vertical stress ($\Delta\sigma$) at point A caused by the entire strip load of width B can be determined by the integration of Eq. (6.32) with limits of r from $-B/2$ to $+B/2$, or

$$\begin{aligned}\Delta\sigma &= \int d\sigma = \int_{-B/2}^{+B/2}\left(\frac{2q}{\pi}\right)\left\{\frac{z^3}{[(x-r)^2+z^2]^2}\right\}dr \\ &= \frac{q}{\pi}\left\{\tan^{-1}\left[\frac{z}{x-(B/2)}\right] - \tan^{-1}\left[\frac{z}{x+(B/2)}\right] \right. \\ &\quad \left. - \frac{Bz[x^2 - z^2 - (B^2/4)]}{[x^2+z^2-(B^2/4)]^2 + B^2z^2}\right\}\end{aligned} \tag{6.33}$$

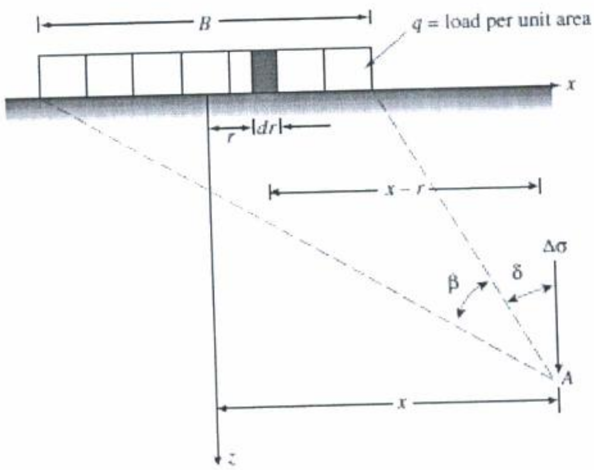


Table 6.5 Variation of $\Delta\sigma/q$ with $2z/B$ and $2x/B$

	2x/B				
2z/B	**0**	**0.5**	**1.0**	**1.5**	**2.0**
0	1.000	1.000	0.500	—	—
0.5	0.959	0.903	0.497	0.089	0.019
1.0	0.818	0.735	0.480	0.249	0.078
1.5	0.668	0.607	0.448	0.270	0.146
2.0	0.550	0.510	0.409	0.288	0.185
2.5	0.462	0.437	0.370	0.285	0.205
3.0	0.396	0.379	0.334	0.273	0.211
3.5	0.345	0.334	0.302	0.258	0.216
4.0	0.306	0.298	0.275	0.242	0.205
4.5	0.274	0.268	0.251	0.226	0.197
5.0	0.248	0.244	0.231	0.212	0.188

Equation (6.33) can be simplified to the form

$$\Delta\sigma = \frac{q}{\pi}[\beta + \sin\beta\cos(\beta + 2\delta)] \tag{6.34}$$

Beban Cyrcular terbagi rata.

Using Boussinesq's solution for vertical stress $\Delta\sigma$ caused by a point load [Eq. (6.19)], we can also develop an expression for the vertical stress below the center of a uniformly loaded flexible circular area.

From Figure 6.19, let the intensity of pressure on the circular area of radius R be equal to q. The total load on the elemental area (shaded in the figure) = $qr\,dr\,d\alpha$. The vertical stress, $d\sigma$, at point A caused by the load on the elemental area (which may be assumed to be a concentrated load) can be obtained from Eq. (6.19):

$$d\sigma = \frac{3(qr\,dr\,d\alpha)}{2\pi}\frac{z^3}{(r^2+z^2)^{5/2}} \tag{6.35}$$

The increase in the stress at point A caused by the entire loaded area can be found by integrating Eq. (6.35), or

$$\Delta\sigma = \int d\sigma = \int_{\alpha=0}^{\alpha=2\pi}\int_{r=0}^{r=R}\frac{3q}{2\pi}\frac{z^3 r}{(r^2+z^2)^{5/2}}\,dr\,d\alpha$$

So

$$\Delta\sigma = q\left\{1-\frac{1}{[(R/z)^2+1]^{3/2}}\right\} \tag{6.36}$$

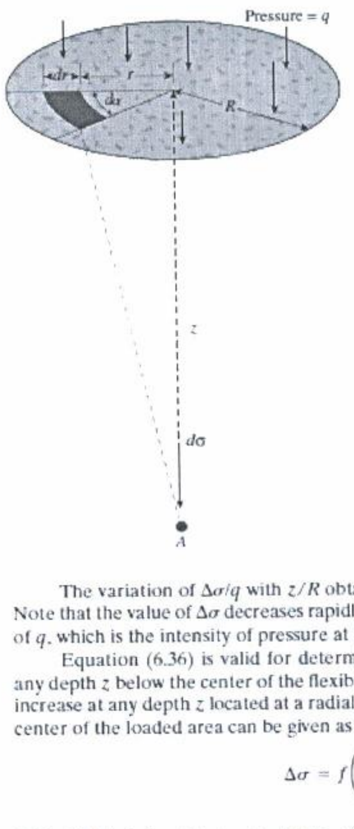


Figure 6.19 Vertical stress below the center of a uniformly loaded flexible circular area

The variation of $\Delta\sigma/q$ with z/R obtained from Eq. (6.36) is given in Table 6.6. Note that the value of $\Delta\sigma$ decreases rapidly with depth, and, at $z = 5R$, it is about 6% of q, which is the intensity of pressure at the ground surface.

Equation (6.36) is valid for determination of vertical stress increase ($\Delta\sigma$) at any depth z below the center of the flexible loaded circular area. Similarly, the stress increase at any depth z located at a radial distance r measured horizontally from the center of the loaded area can be given as

$$\Delta\sigma = f\left(q, \frac{r}{R}, \frac{z}{R}\right)$$

Table 6.6 Variation of $\Delta\sigma/q$ with z/R [Eq. (6.36)]

z/R	$\Delta\sigma/q$	z/R	$\Delta\sigma/q$
0	1	1.0	0.6465
0.02	0.9999	1.5	0.4240
0.05	0.9998	2.0	0.2845
0.10	0.9990	2.5	0.1996
0.2	0.9925	3.0	0.1436
0.4	0.9488	4.0	0.0869
0.5	0.9106	5.0	0.0571
0.8	0.7562		

Table 6.7 Variation of I_4 [Eq. (6.37)]

z/R	r/R 0	0.2	0.4	0.6	0.8	1.0
0	1.000	1.000	1.000	1.000	1.000	1.000
0.1	0.999	0.999	0.998	0.996	0.976	0.484
0.2	0.992	0.991	0.987	0.970	0.890	0.468
0.3	0.976	0.973	0.963	0.922	0.793	0.451
0.4	0.949	0.943	0.920	0.860	0.712	0.435
0.5	0.911	0.902	0.869	0.796	0.646	0.417
0.6	0.864	0.852	0.814	0.732	0.591	0.400
0.7	0.811	0.798	0.756	0.674	0.545	0.367
0.8	0.756	0.743	0.699	0.619	0.504	0.366
0.9	0.701	0.688	0.644	0.570	0.467	0.348
1.0	0.646	0.633	0.591	0.525	0.434	0.332
1.2	0.546	0.535	0.501	0.447	0.377	0.300
1.5	0.424	0.416	0.392	0.355	0.308	0.256
2.0	0.286	0.286	0.268	0.248	0.224	0.196
2.5	0.200	0.197	0.191	0.180	0.167	0.151
3.0	0.146	0.145	0.141	0.135	0.127	0.118
4.0	0.087	0.086	0.085	0.082	0.080	0.075

or

$$\frac{\Delta\sigma}{q} = I_4$$

The variation of I_4 with r/R and z/R is given in Table 6.7.

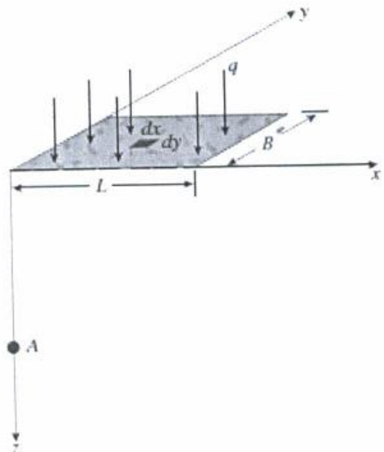


Figure 6.20 Vertical stress below the corner of a uniformly loaded flexible rectangular area

The increase in the stress $\Delta\sigma$ at point A caused by the entire loaded area can now be determined by integrating the preceding equation:

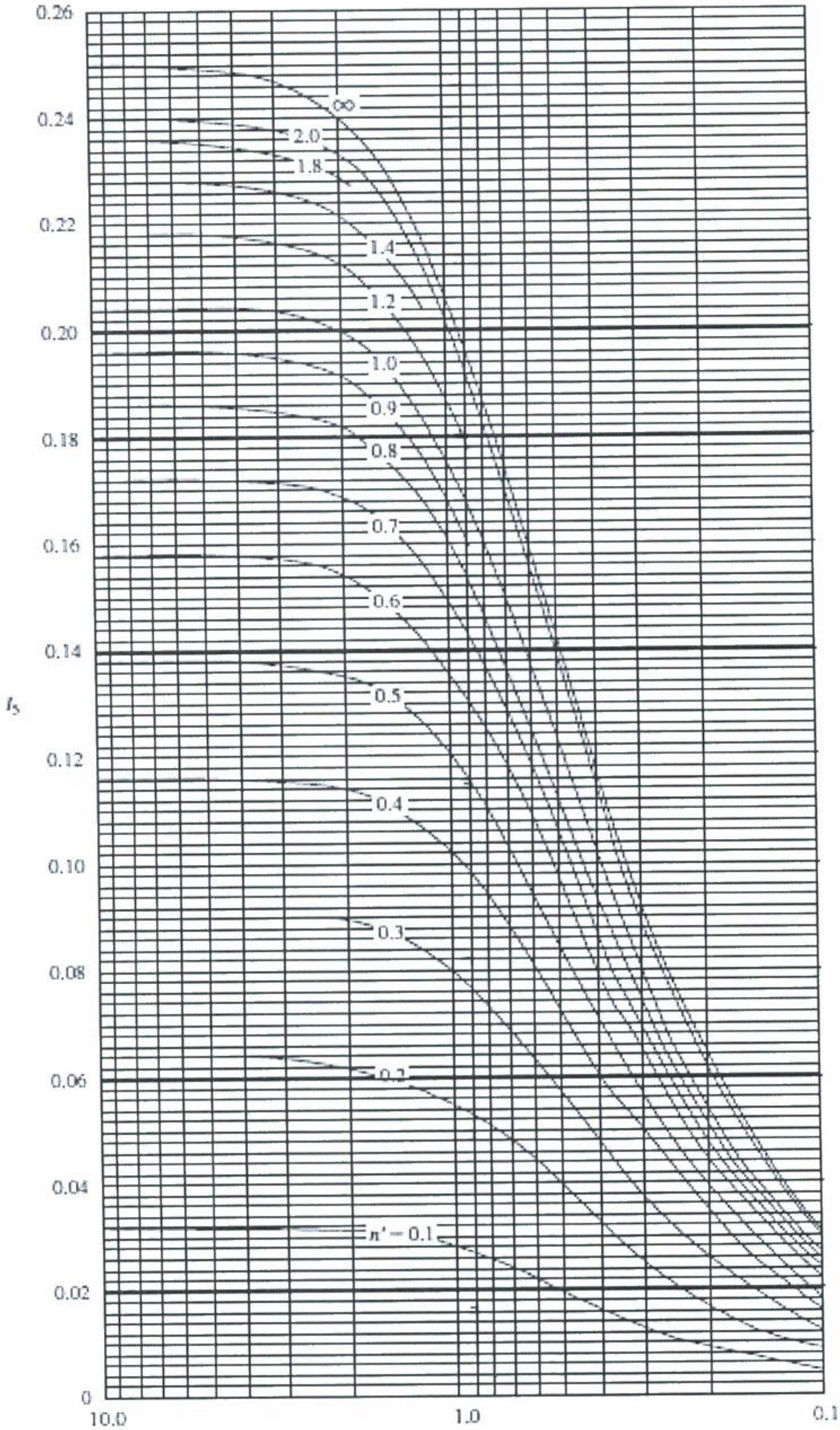
$$\Delta\sigma = \int d\sigma = \int_{y=0}^{B}\int_{x=0}^{L} \frac{3qz^3(dx\,dy)}{2\pi(x^2 + y^2 + z^2)^{5/2}} = qI_5 \tag{6.40}$$

where

$$I_5 = \frac{1}{4\pi}\left[\frac{2m'n'\sqrt{m'^2 + n'^2 + 1}}{m'^2 + n'^2 + m'^2n'^2 + 1}\left(\frac{m'^2 + n'^2 + 2}{m'^2 + n'^2 + 1}\right) + \tan^{-1}\left(\frac{2m'n'\sqrt{m'^2 + n'^2 + 1}}{m'^2 + n'^2 - m'^2n'^2 + 1}\right)\right] \tag{6.41}$$

$$m' = \frac{B}{z} \tag{6.42}$$

$$n' = \frac{L}{z} \tag{6.43}$$


0.26
0.24
0.22
0.20
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
0
I_5
10.0
1.0
0.1
∞
2.0
1.8
1.4
1.2
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
n' = 0.1

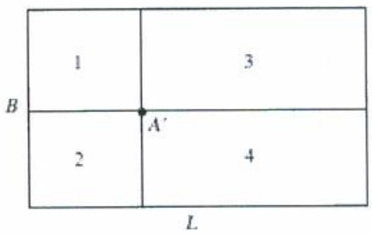


Figure 6.22 Increase of stress at any point below a rectangularly loaded flexible area

due to each rectangular area can now be calculated by using Eq. (6.40). The total stress increase caused by the entire loaded area can be given by

$$\Delta\sigma = q[I_{S(1)} + I_{S(2)} + I_{S(3)} + I_{S(4)}] \tag{6.44}$$

In many circumstances it may be necessary to calculate the stress increase below the center of a uniformly loaded rectangular. For convenience the stress increase may be expressed as

$$\Delta\sigma_c = qI_c \tag{6.45}$$

where

$$I_c = f(m_1, n_1) \tag{6.46}$$

$$m_1 = \frac{L}{B} \tag{6.47}$$

and

$$n_1 = \frac{z}{\frac{B}{2}} \tag{6.48}$$

Table 6.8 gives the variation of I_c with m_1 and n_1.

Table 6.8 Variation of I_c with m_1 and n_1 [Eq. (6.45)]

	m_1									
n_1	1	2	3	4	5	6	7	8	9	10
0.20	0.994	0.997	0.997	0.997	0.997	0.997	0.997	0.997	0.997	0.997
0.40	0.960	0.976	0.977	0.977	0.977	0.977	0.977	0.977	0.977	0.977
0.60	0.892	0.932	0.936	0.936	0.937	0.937	0.937	0.937	0.937	0.937
0.80	0.800	0.870	0.878	0.880	0.881	0.881	0.881	0.881	0.881	0.881
1.00	0.701	0.800	0.814	0.817	0.818	0.818	0.818	0.818	0.818	0.818
1.20	0.606	0.727	0.748	0.753	0.754	0.755	0.755	0.755	0.755	0.755
1.40	0.522	0.658	0.685	0.692	0.694	0.695	0.695	0.696	0.696	0.696
1.60	0.449	0.593	0.627	0.636	0.639	0.640	0.641	0.641	0.641	0.642
1.80	0.388	0.534	0.573	0.585	0.590	0.591	0.592	0.592	0.593	0.593
2.00	0.336	0.481	0.525	0.540	0.545	0.547	0.548	0.549	0.549	0.549
3.00	0.179	0.293	0.348	0.373	0.384	0.389	0.392	0.393	0.394	0.395
4.00	0.108	0.190	0.241	0.269	0.285	0.293	0.298	0.301	0.302	0.303
5.00	0.072	0.131	0.174	0.202	0.219	0.229	0.236	0.240	0.242	0.244
6.00	0.051	0.095	0.130	0.155	0.172	0.184	0.192	0.197	0.200	0.202
7.00	0.038	0.072	0.100	0.122	0.139	0.150	0.158	0.164	0.168	0.171
8.00	0.029	0.056	0.079	0.098	0.113	0.125	0.133	0.139	0.144	0.147
9.00	0.023	0.045	0.064	0.081	0.094	0.105	0.113	0.119	0.124	0.128
10.00	0.019	0.037	0.053	0.067	0.079	0.089	0.097	0.103	0.108	0.112