Lecture # 17

Seepage and Flow-nets

- Poisson and Laplace’s equations

- Forcheimer’s algebraic version of Laplace

- Flow nets

- Drainage (de-watering, etc)

CONTINUITY OF FLOW THROUGH SOILS AND ROCKS.

During seepage, there are similar conditions between the solids and the fluid.

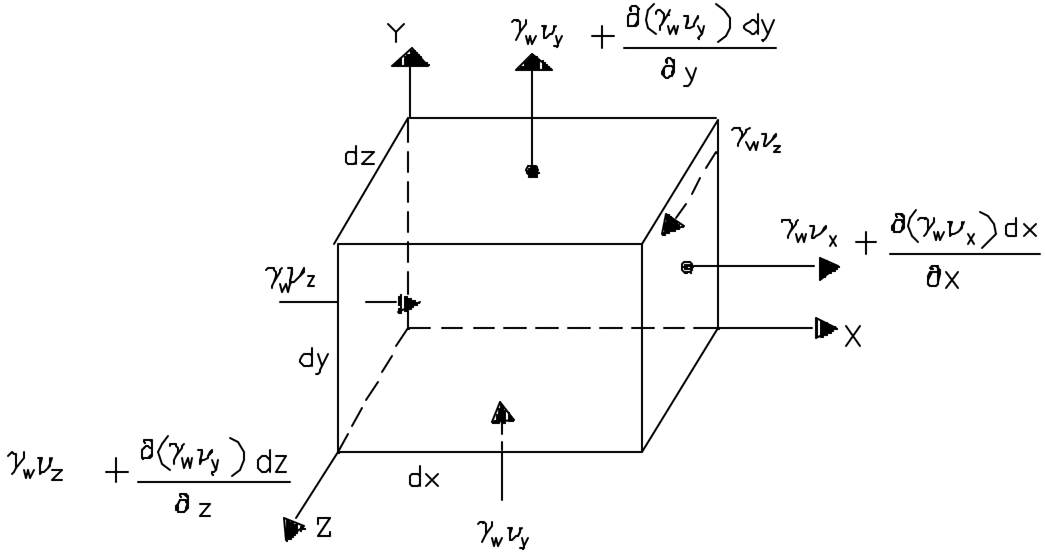
a) fluids,
- i) compatibility, which means, flow in = flow out + storage
- ii) material conditions, such as, pressure, temperature, volume relations,
- iii) force relations, $F = m_f\, a$.

b) solids,
- i) geometric compatibility,
- ii) material comp. (stress-strain relationship),
- iii) force equations.

c) interaction relationships, such that on any surface the total pressure $\sigma = \sigma' + u$.

Assume a simplifying assumption that there is no storage. Considering fluid conditions only.

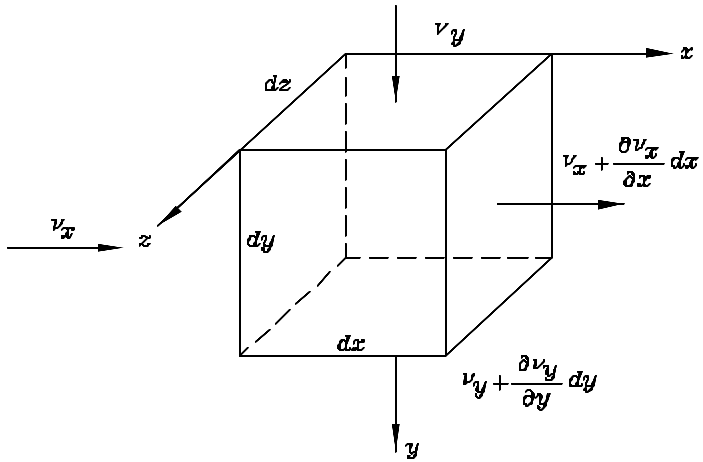
$$\text{Mass flow} = \frac{\text{Flow}}{\text{Area}}$$



statement at bottom: "integrate for solution with boundary conditions"

assume:
- a) soil is saturated or
- b) volume of water in voids is constant
- c) k is constant.

consider an element of soil: $(dx)(dy)(1)$



$$q_x = k_x i_x A_x$$
$$q_y = k_y i_y A_y$$

but $i = \frac{dh}{dl}$ *and* $v = \frac{q}{A}$

$$v_x = k_x i_x = k_x \frac{\partial h}{\partial x}$$
$$v_y = k_y i_y = k_y \frac{\partial h}{\partial y}$$

but $q_{in} = q_{out}$ $\therefore$ $(vA)_i = (vA)_o$

flow in / *flow out*

$$v_x\, dy\,(1) + v_y\, dx\,(1) = \left(v_x + \frac{\partial v_x}{\partial x} dx\right) dy + \left(v_y + \frac{\partial v_y}{\partial y}\right) dx$$

$$\therefore \boxed{\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} = 0}$$ *Poisson's equation*

but $v_x = k_x \frac{\partial h}{\partial x}$ *,etc...*

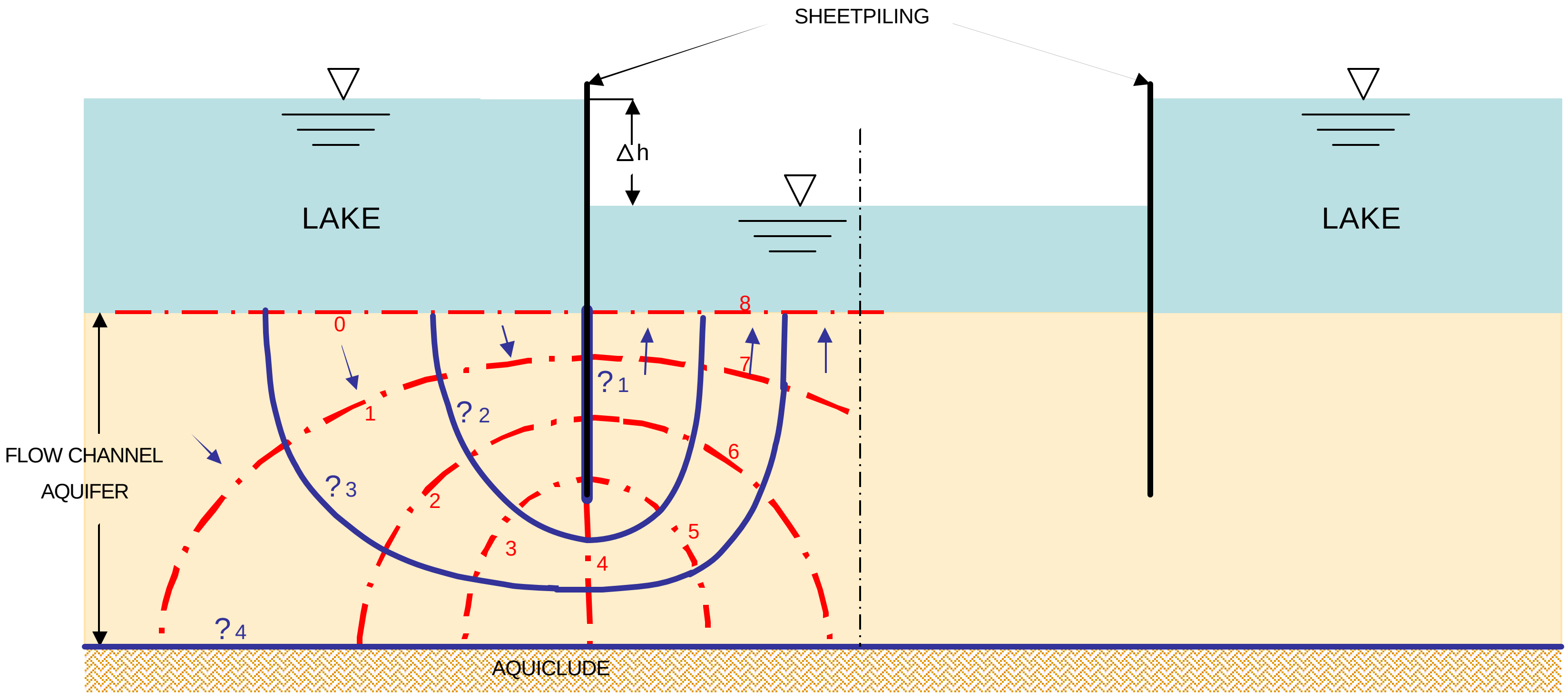
$$\therefore k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial y^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0$$ ← *means no volume change otherwise* $= \frac{dv}{dt}$

if isotropic permeability $k_x = k_y = k_z = k$

$$\therefore \boxed{\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0}$$ *Laplace's equation.*

EQUATION OF CONTINUITY.

If flow is not uniform or unidirectional, we will need to use flow nets, which are graphical representations of Laplace's equation of continuity of flow. For example, consider the cofferdam:



? FLOW LINES (TRAJECTORIES OF WATER PARTICLES, STREAM LINES)

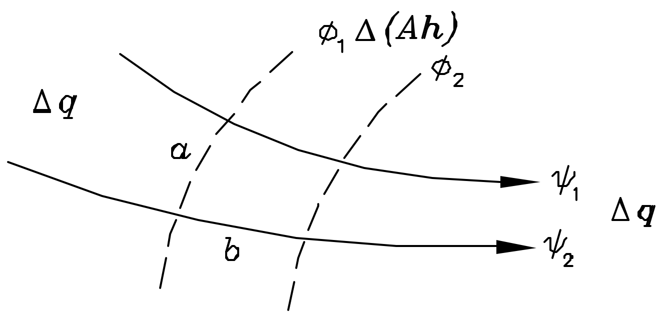
Ø EQUIPOTENTIAL LINES (LOCI OF POINT WITH EQUAL TOTAL HEADS OR CONTOURS OF EQUAL ENERGY)

Forcheimer's graphical solution

rules of flow nets with: $\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$

1. ψ & ϕ are othogonal families of curves (canonical).
2. upstream + downstream surfaces of perm. layers are ϕ's
3. boundaries of imperm. surfaces are ψ's

consider a flow channel between ψ_1 and ψ_2



number of flow channels $q = N_f \Delta q$

number of equipotentials $\Delta h = N_p \Delta(Ah)$

$\therefore \quad q = N_f \Delta q = N_f \Delta(k i A) = N_f\, k \left(\frac{\Delta(Ah)}{b}\right) a(1)$

$\therefore \quad \boxed{q = k \Delta h \left(\frac{a}{b}\right) \frac{N_f}{N_p}}$

if you make $\frac{a}{b} \cong 1$ for a square net, then

$$q = k \Delta h \frac{N_f}{N_p}$$

for inisotropic conditions $kx \neq ky$ in Laplace's

$$kx \frac{\partial^2 h}{\partial x^2} + ky \frac{\partial^2 h}{\partial y^2} = 0 \qquad (\therefore \bar{\Psi} * \bar{\Phi})$$

write as $\left(\frac{ky}{kx}\right) \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$ set $x' = \sqrt{\frac{ky}{kx}}$

$$\therefore \frac{\partial^2 h}{\partial x'^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

x' is a new coordinate system where y scale remains the same, but x-scale is drawn to $\sqrt{\frac{ky}{kx}}$ scale.

Then the rate of seepage,

$$q = \sqrt{kx\ ky}\ \Delta H \frac{N_f}{N_{eq}}$$

Drainage.

There are several methods commonly used to drain construction site:

1) ***Gravity.*** This is the cheapest method. The site is drained through channels placed at intervals, that permit the water to flow away from the high points. Miami was built by cutting channels and using the fill to raise the land above the flood levels. This method has been used for thousands of years. It has the disadvantage of requiring a long time to drain the land.

2) ***Evaporation.*** A slightly faster method than gravity is evaporation, wherein the land is plowed to open the soil to the sun and accelerate the evaporation of its moisture. Although cheap, it is slow because rain and flooding quickly cancel the progress.

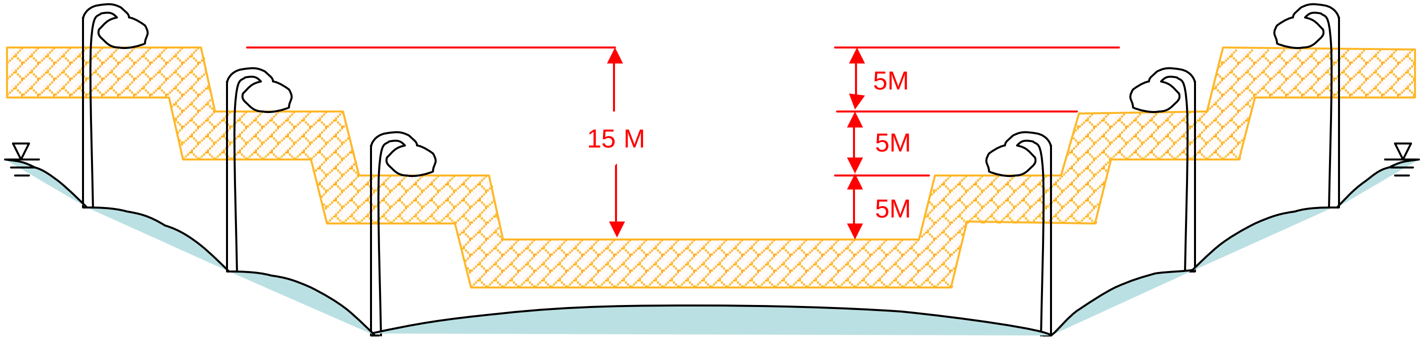
3) ***Vacuum.*** This method is more expensive than gravity, but is faster in results. It requires pumps that suck the water out of the soil and remove it to a distant river or lake.

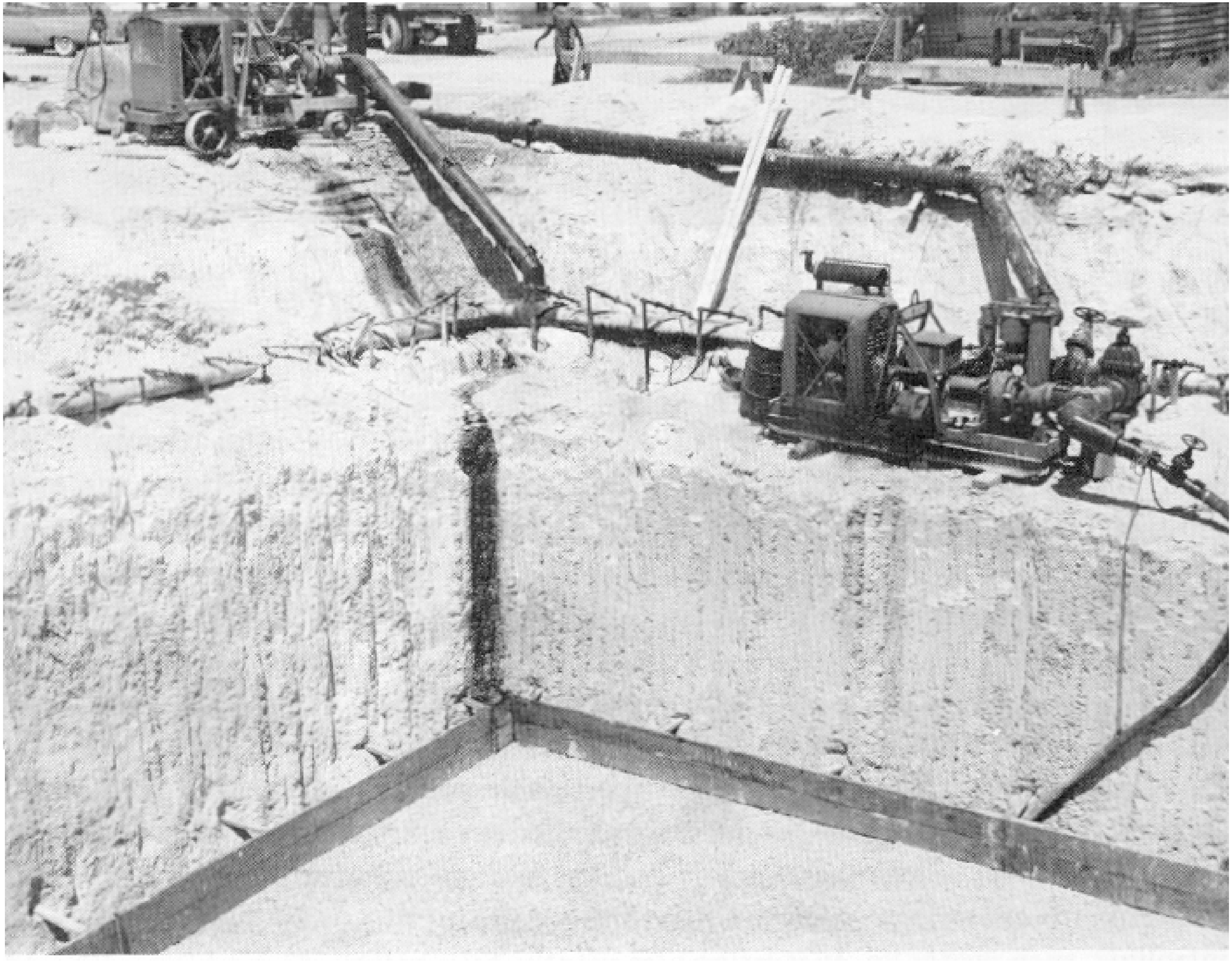
4) ***Electro-osmosis.*** The use of low voltage to force the migration of the water molecules to the cathodes of the mesh is probably the most expensive of the alternatives, but may be the only effective method in deep clay soils.

In the above methods, the drainage path for the water must be made as short as possible. The methods used include ***open and closed drains***, ***ditches***, ***sumps***, ***french drains***, ***sand*** or ***auger holes***, ***drainge wicks***, ***blanket drains***, ***deep wells***, ***well-points*** (next slide) and ***soil freezing*** (through brine), etc. All these methods should lead to collecting points where the water is filtered and maybe even treated before it is disposed into an existing river or lake.

Staged dewatering through well-points.

The diagram below shows the lowering of the GWT by 7 m of a construction site, in order to build a building's foundations in a dry environment. It must be done with great caution, because it could cause large settlements of the adjacent structures, and the collapse of the perimeter embankments due to the loss of the capillary moisture.





Dewatering an excavation in downtown Miami.

Lecture #18

The Concepts of

Effective Stress and Porewater Pressure

- Archimedes’ principle
- Effective stress s ’
- Pore water pressure u
- Liquefaction of soils (“quicksand”)
- The “critical” gradient

Archimedes' Principle.

The important concept of buoyancy was developed by ***Archimedes (287-212 BC)*** and published in his book "***Floating Bodies***", which is probably the first known text on hydrostatics. The roman historian Vitruvius brought to us the now famous story of how he discovery the principle of buoyant weights. Syracuse's king Hieron had commissioned some jewelers to fashion a new crown from a large amount of gold he provided to them. The crown was finished in time, but King Hieron suspected that some of the gold had been replaced with much cheaper silver. Hieron requested Archimedes to find a way to prove or disprove his suspicions. Incidentally, this request was possibly the first recorded contract in history for consulting engineering.

Archimedes pondered on this problem for weeks. Unable to sleep, one night he visited the public baths well past midnight, and he stepped into the quiet and still waters of the soaking pool. As he stepped in, he observed that the waters were spilling out of the pool in the same rate as his body submerged into the pool. He then had a flash of insight! Purportedly he leaped from the bath and ran home naked (having forgotten his toga by the pool) shouting "eureka" (I have found it)! That same day he borrowed from the king, the famous crown, a piece of gold and a piece of silver. He then weighed each (W), and then measured the volume (V) of water displaced by each of the three. In essence, Archimedes had come up with the concept of specific gravity of solids (W/V). Thus, having determined the specific gravity of the gold and the silver separately, he was able to calculate the proportional amounts of gold and silver in the crown. He found the crown to have 25% of its gold volume replaced with silver. Unfortunately Vitruvius did not report on the indubitably cruel fate of the jewelers.

Archimedes' important principle has come to us thusly:

"The loss of weight of a body submerged in water is equal to the weight of water displaced, and that a floating body displaces its own weight of water."

The Principle of Effective Stress

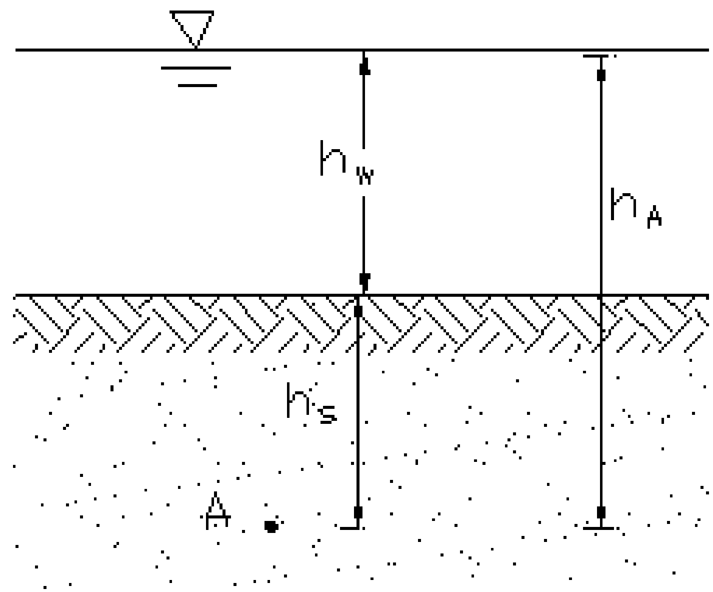
In order to know the state of stress at any point inside a body of soil, it is essential to predict the soil behavior under loads. Consider these two simple general cases:

(A) The state of stress under static hydraulic forces, and

(B) The state of stress of the soil when subjected to seepage (dynamic) forces.

(A) Stresses in Soils under Static Hydraulic Forces.

The stress s at any point A is ,



$$s = h_w\ ?_w + (h_A - h_w)\ ?_{sat}$$

where s is the ***total stress***.

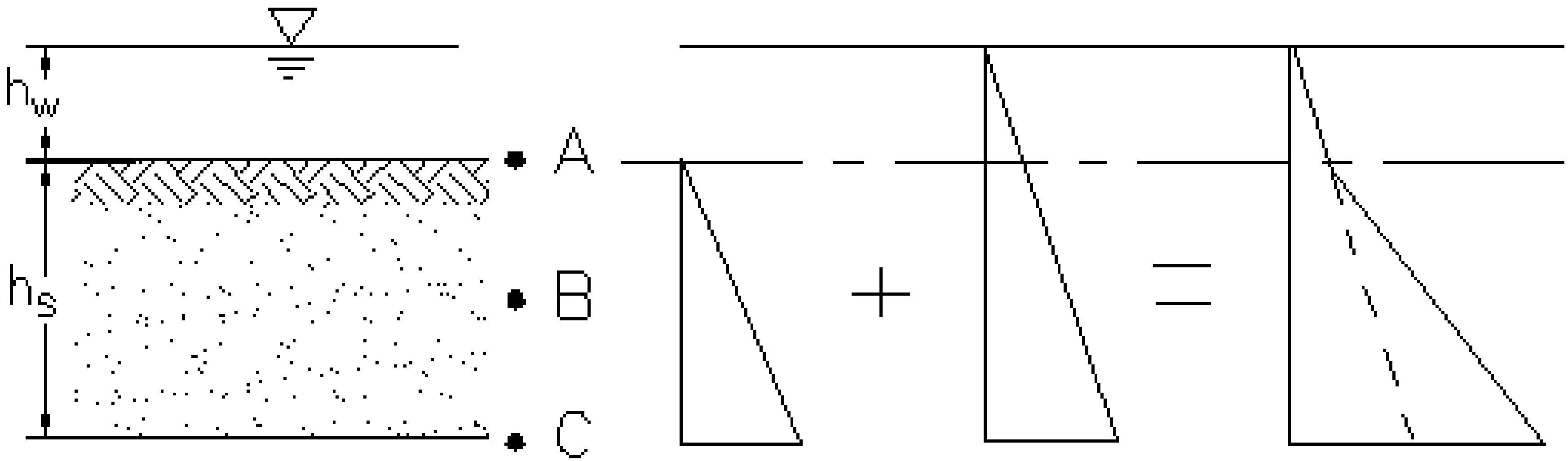
This total stress s can be visualized to consist of TWO components : (1) the pressure upon the soil skeletal component, called the ***effective stress s '*** and, (2) the pressure from the water component in the voids continuum, called the ***pore water pressure u***, also known as the neutral stress.

$$\underline{\boldsymbol{s = s' + u}}$$

or $s' = s - u = [h_w ?_w + (h_A - h_w) ?_{sat}] - h_A ?_w$

$s' = (h_A - h_w)(?_{sat} - ?_w) = \underline{h_{soil} ?'}$

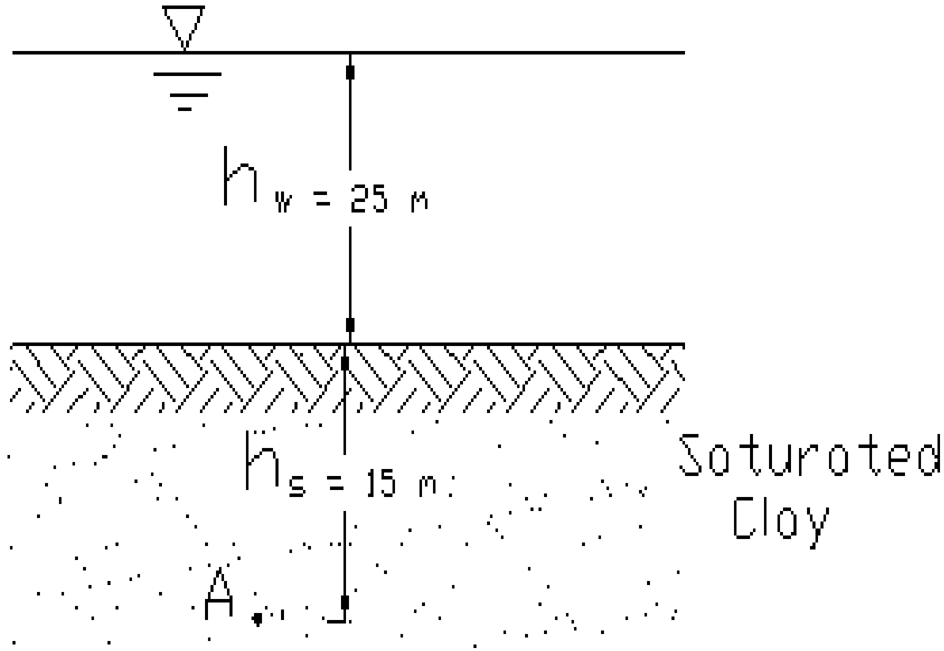
where $?' = ?_b = ?_{sat} - ?_w$ is the ***buoyant*** or ***submerged unit weight*** of soil .Therefore, the s' at A is independent of the depth of the water above that point.



$h_s ?' + (h_w + h_s) ?_w = h_s ?' + (h_w + h_s) ?_w$

$s' + u = s$

For example, a soil sample was obtained from point A in the submerged clay layer shown below, and it had a w = 54 % and a G_s = 2.78. What is the effective vertical stress s ' at A?



$$s' = ?' h_s$$

but $?' = ?_{sat} - ?_w$

$$= \frac{(G_s + e)\, ?_w}{1 + e} - ?_w$$

and since e = w G_s (because S = 1)

$$s' = \left[\frac{(G_s + w G_s)}{[1 + w G_s]} ?_w - ?_w \right] h_s$$

$$s' = \left[\frac{2.78 + 0.54(2.78)}{1 + 0.54(2.78)}\right](9.8) - 9.8] (15 \text{ m}) = \underline{105 \text{ kPa}}$$

Example. The city of Houston (Texas) has been experiencing a gradual lowering of its phreatic surface during the past 43 years due to large well draw-downs from heavy industrial users. (a) What was the effective vertical stress at a depth of 15 m in 1961? (b) What is the effective stress at the same depth in 2004, and what has happened to the ground surface?

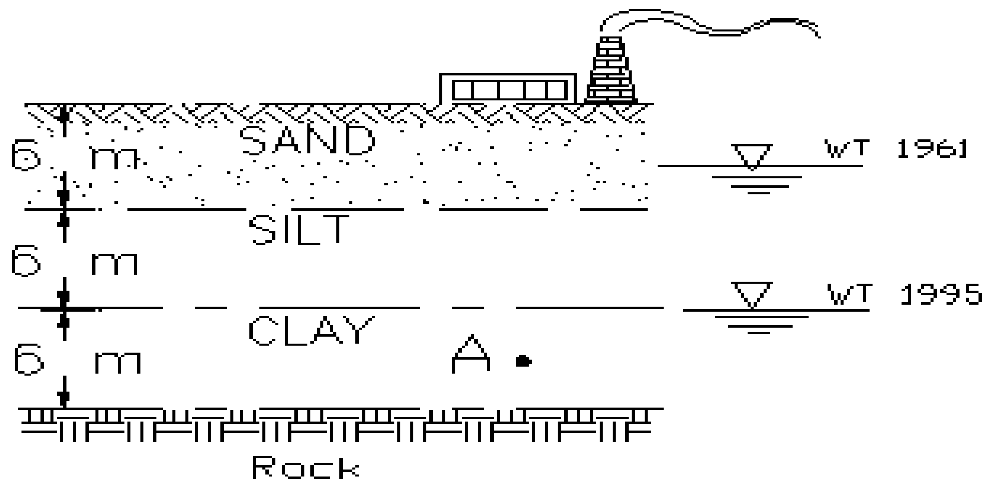
For sand ? = 20.4 kN/m^3 ⇒

?$_{sat}$ = 18.8 kN/m^3

For silt ? = 16.5 kN/m^3 ⇒

?$_{sat}$ = 14.9 kN/m^3

For clay ?$_{sat}$ = 12.6 kN/m^3 ⇒



(a) s_v' = [?h + ?' h']$_{sand}$ + [?' h']$_{silt}$ + [?' h']$_{clay}$

s_v' = [(20.4)(3) + (18.8 - 9.8)(3)] + [(14.9 - 9.8)(6)] + [(12.6 -9.8)(3)] = <u>128 kPa</u>

(b) s_v' = [?h]$_{sand}$ + [? h]$_{silt}$ + [?' h']$_{clay}$

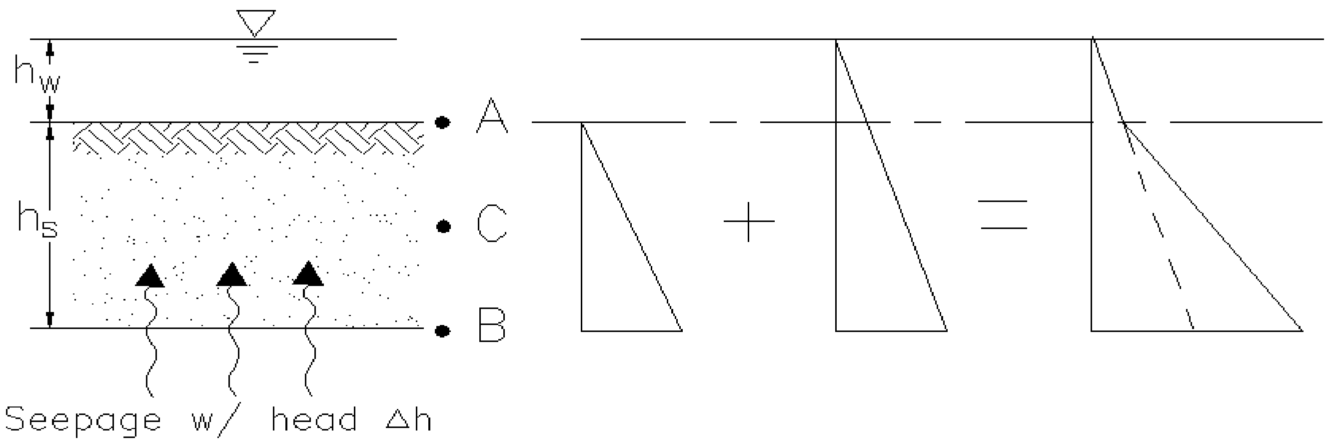
s_v' = [(20.4)(6) + (16.5)(6)] + [(12.6 - 9.8)(3)] = <u>230 kPa, an 80 % increase in stress.</u>

(c) The ground surface has also been lowered, due to the decreasing thickness of the clay and silt strata.

(B) Stresses with (dynamic) Seepage Forces.

Depending on the direction of the seepage flow, the effective stress may increase or decrease in size.

Case 1. Upward Seepage **s ' + u = s**



at point A, s = s ' + u or $h_w\,?_w$ = s ' + $h_w\,?_w$ ∴ <u>s ' = 0</u> (obviously, since there is no soil down to this level)

at point B, s = s ' + u or $h_w\,?_w + h_s\,?_{sat}$ = s ' + $(h_w + h_s + ?\,h)\,?_w$

∴ s ' = s - u = $h_s\,(?_{sat} - ?_w) - ?\,h\,?_w$ = $h_s\,?' - ?\,h\,?_w$

at point C (that is, at any depth z),

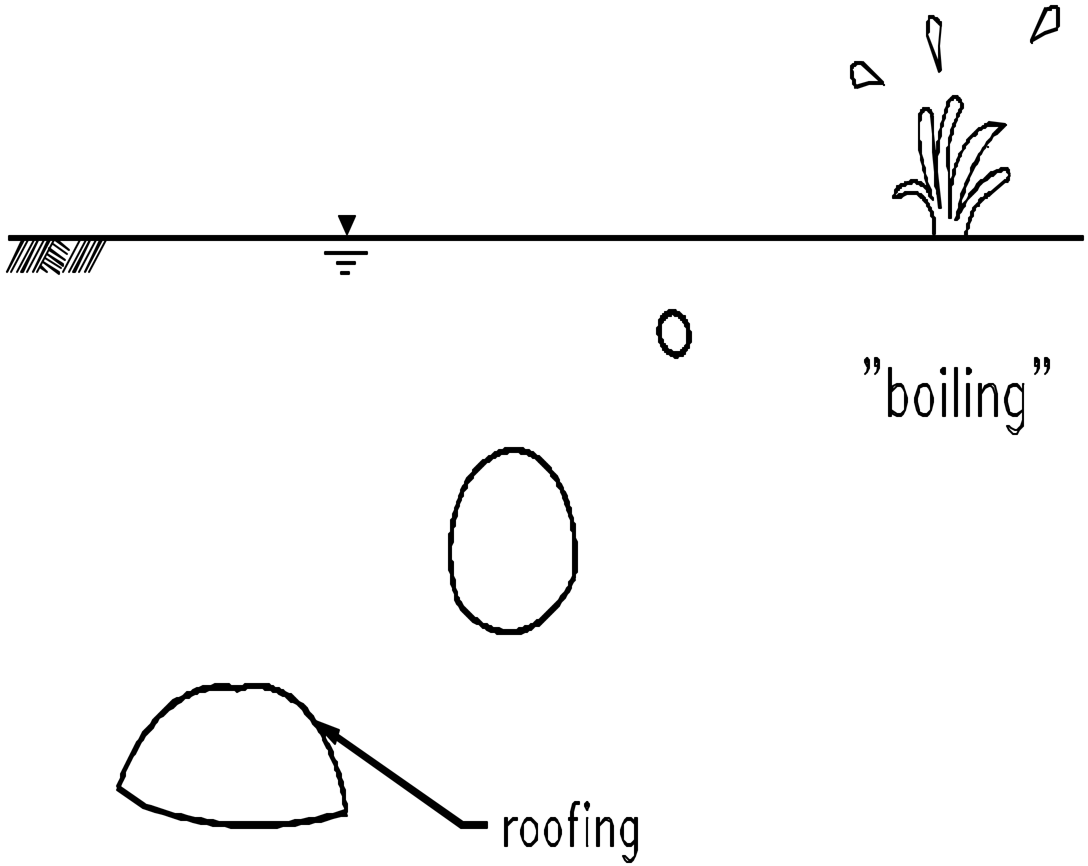
$$s' = s - u = (h_w ?_w + z ?_{sat}) - (h_w + z + \frac{?h\,z}{h_s}) ? = z ?' - \frac{?h}{h_s} z ?_w$$

where $?h/h_s$ is the ***hydraulic gradient i*** created by the flow. At the point where $s_{cr}' = 0$ the soil skeleton is incapable of carrying any load (the stability of the soil is zero), in other words, the hydraulic gradient is critical, and creates a condition referred to as ***boiling*** or a ***quick condition***, although the correct term is ***liquefaction***. At that point $s = u$ and $s' = 0$. From the equation above,

Since $s_{cr}' = 0$ $\therefore$ $s_{cr}' = z ?' - \boldsymbol{i_{cr}} z ?_w = 0$

therefore, $i_{cr} = \dfrac{?'}{?_w}$ (typically varies between 0.9 to 1.1)

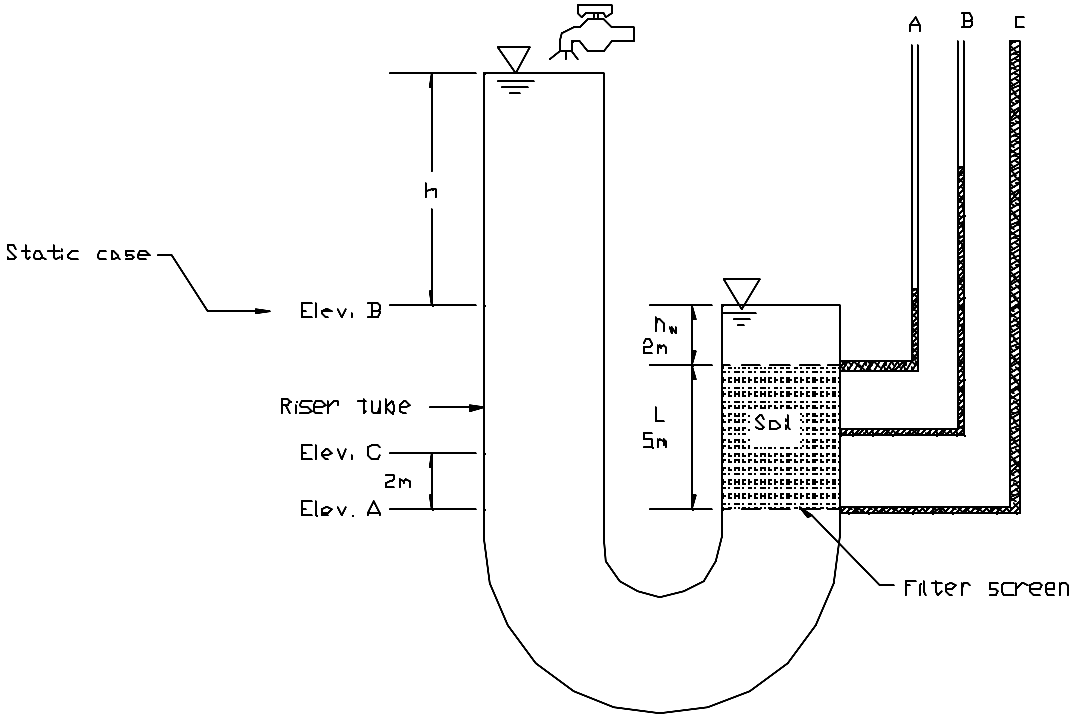
In a sand, the quick condition will be described as ***quicksand***. Note that this is a condition and not a soil type. The soil may be stable in normal circumstances, and then a flood may generate large vertical flows in a limited region. That soil turns quick, and its condition has changed until its cause is eliminated.



The term boiling comes from the rapid rise of pressure in the pore water as the upward seepage forces increase. The bubbles of water burst as they arrive at the soil surface, and produce an effect similar to a boiling pot of water, although the soil phenomenon has nothing to do with temperature. The correct name is ***liquefaction***.

Liquefaction.

The flow of water through a soil produce a ***seepage force*** on the soil grains. In the apparatus shown in the figure below, consider three states : **(1) a** low head seepage, wherein the weight of the soil is much greater than the upward seepage force.



(2) As the upward seepage force is increased by an increasing head, the force gradually overcomes the weight of the soil skeleton. At the point where both forces are equal the soil is said to have attained ***liquefaction***.

(3) When the upward seepage force continues to increase, the soil is said to be in a ***quick*** condition. The word comes from the Old English "***cwic***" which meant ***living*** or ***alive***. It was used to describe the apparent "***boiling***" of the surface of the sand, where it "***seemed to be alive***" whence "***quicksand***".

Also evident from the figure in the previous slide is the height ***h*** at which "boiling" will commence.

The total stress at A, $\sigma = \gamma_{sat} L + \gamma_w h_w$

The hydraulic seepage force $S_f = \boldsymbol{i} * \gamma_w * L$ (1) for a unity cross-section

and the weight of the solids is $W = L * (\gamma - \gamma_w)$ (1) for a unity cross-section.

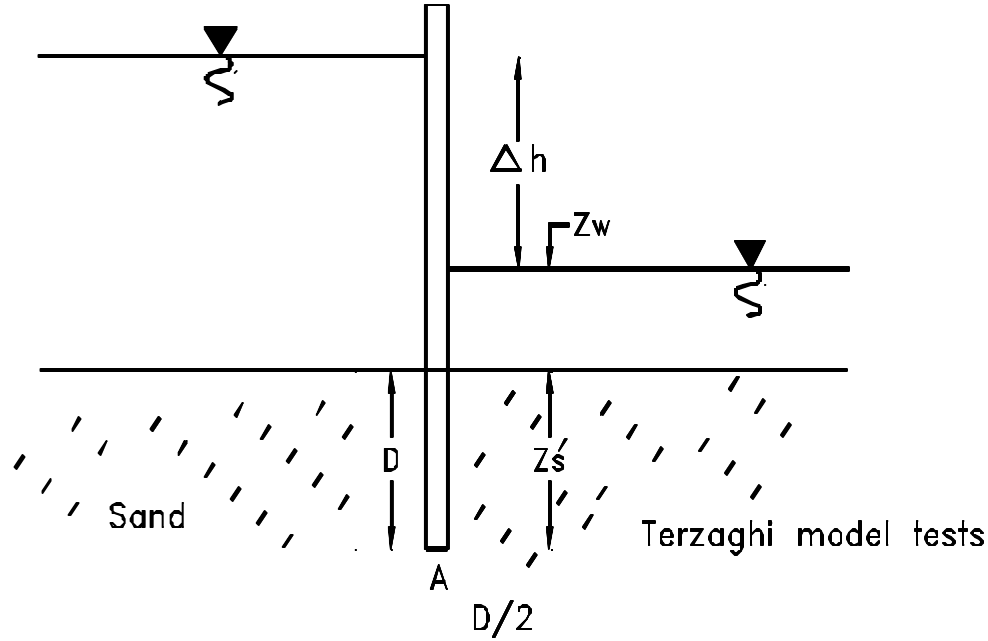
Therefore, ***liquefaction*** will occur when $S_f = W$, and the critical gradient $\boldsymbol{i}_{cr}$ is,

$$i_{cr} = \frac{g - g_w}{g_w}$$

and also $$i_{cr} = \frac{G_s - 1}{e + 1}$$

In summary any $\boldsymbol{i}_{cr}$ that approaches 1.0 will risk the loss of the structure. As a factor of safety, common hydraulic gradients around structures should be less than 0.30 to 0.35.

Consider the condition shown at right, where a sheet-piling cofferdam is being dewatered.



Look closely at the soil volume between point A and the bottom of the excavated cofferdam.

In that region, s ' = 0

$$u = (\mathrm{D}\mathbf{h} + \mathbf{z_w} + \mathbf{z_s})\, g_\mathbf{w} \quad \text{and} \quad s = \mathbf{z_w}\, g_\mathbf{w} + \mathbf{z_s}\, g_\mathbf{sat}$$

$$\therefore \quad \Delta h\, g_\mathbf{w} + \mathbf{z_w}\, g_\mathbf{w} + \mathbf{z_s}\, g_\mathbf{w} = \mathbf{z_w}\, g_\mathbf{w} + \mathbf{z}_\mathrm{s}\, g_\mathbf{sat}$$

$$\therefore \quad \Delta h\, g_\mathbf{w} = \mathbf{z_s}\,(g_\mathbf{sat} - g_\mathbf{w}) = \mathbf{z_s}\ \gamma'$$

$$\text{but} \quad \boldsymbol{i}_\mathrm{cr} = \frac{D\boldsymbol{h}}{\mathbf{z_s}} = \frac{g¢}{g_\mathbf{w}} = \frac{g - g_\mathbf{w}}{g_\mathbf{w}}$$

Note that z_s = D (the depth of embedment of the sheet-piling).

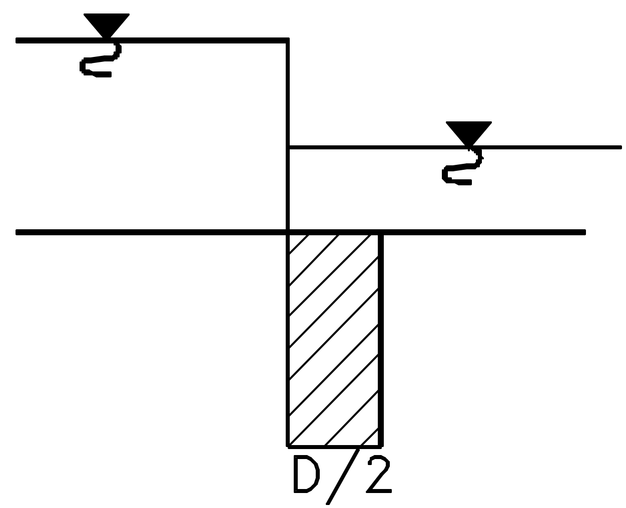
For example, a sand with γ = 125 pcf, Gs = 2.65 and e ranging from 1.0 for loose to 0.25 for dense,

$$\boldsymbol{i_{cr}} = \frac{\Delta \mathbf{h}}{\mathbf{D}} = \frac{\mathbf{G_s - 1}}{\mathbf{1 + e}} \sim \mathbf{1}$$

therefore, look for liquefaction when $\Delta h \approx D$.

Karl Terzaghi performed some tests on models of excavations, and predicted that liquefaction will occur within a distance of D/2 from the inside face of the sheet-piling.

$$\mathbf{FS} = \frac{\mathbf{W'}}{\mathbf{U}} = \frac{\mathbf{D(D/2)}\ (\gamma_{sat} - \gamma_w)}{\mathbf{1/2\ D^2}\ \boldsymbol{i_{avg}}\ \gamma_w} = \frac{\gamma'}{\boldsymbol{i_{avg}}\ \gamma_w}$$



References.

1. “The Works of Archimedes”, T.L. Heath, Cambridge University Press.

2. “Vitruvius: Scriptores metrologici Romani”, Hultsch.

3. “Recherches Historique sur le Principe d’Archimdède”, Thurot, Paris, 1869.

Lecture #21

Stress in Soil Masses

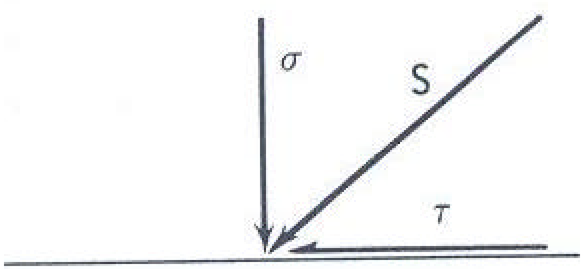
1. ***The general stress equations***
2. ***Mohr's graphical solutions***
3. ***Boussinesq's mathematical solutions***
4. ***Newmark's influence chart solutions***

Introduction.

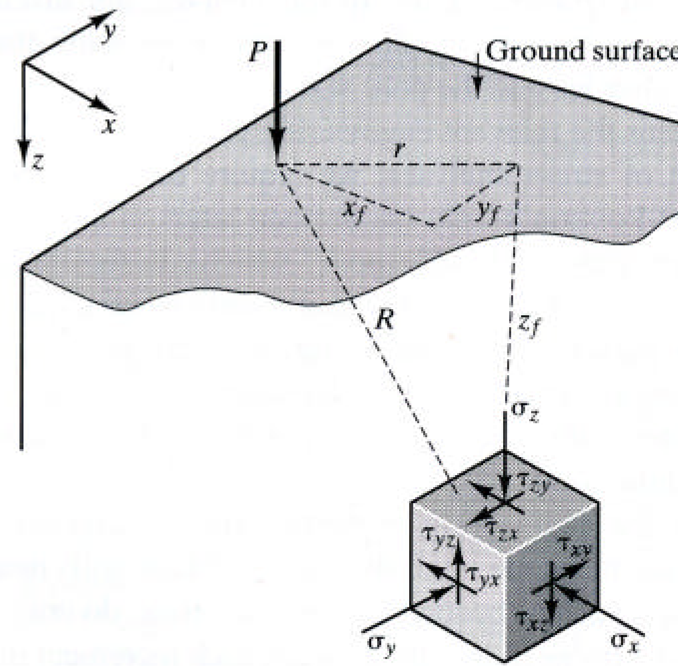
There are two basic types of forces that operate *in* and *on* structures:

1. ***Body forces*** **("inner" forces, such as self weight, nuclear, etc.), and**
2. ***Traction or Surface forces*** **("outer" forces, such as normal and shear forces, etc.).**

These outer forces S are resolved into the two components, the *normal stress* s that is perpendicular to the surface and the *shear stress* t that is along the surface.

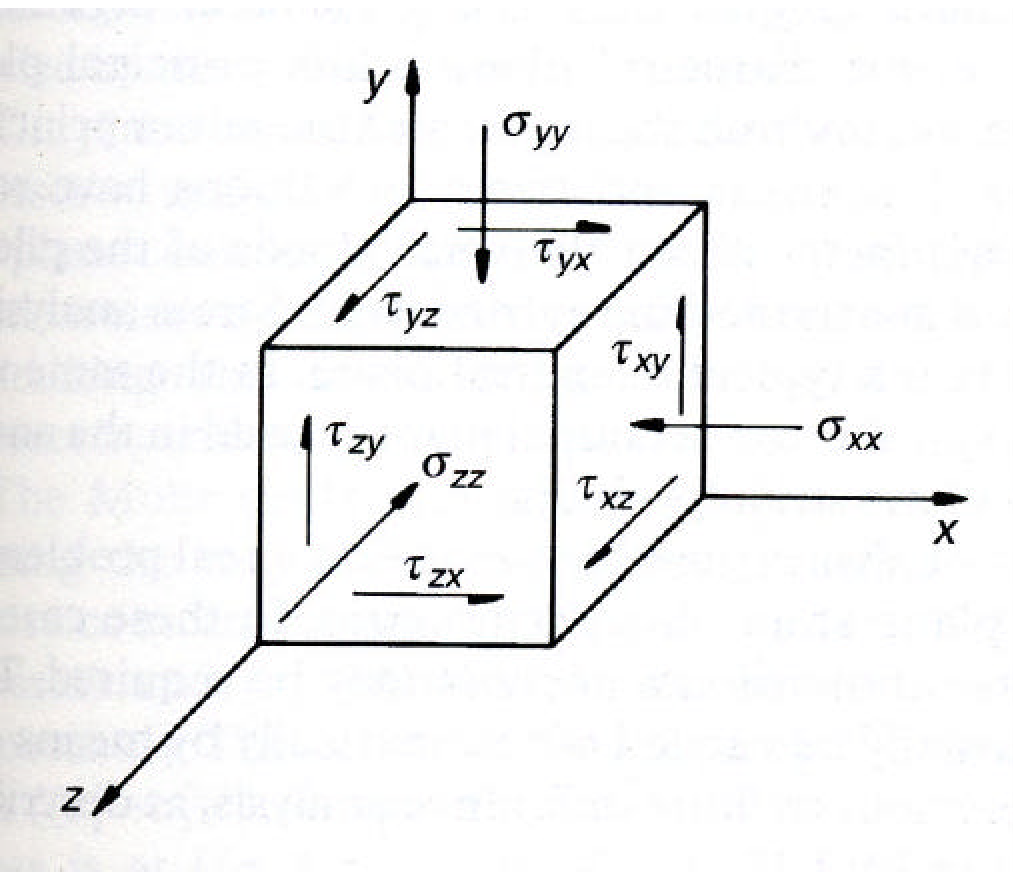


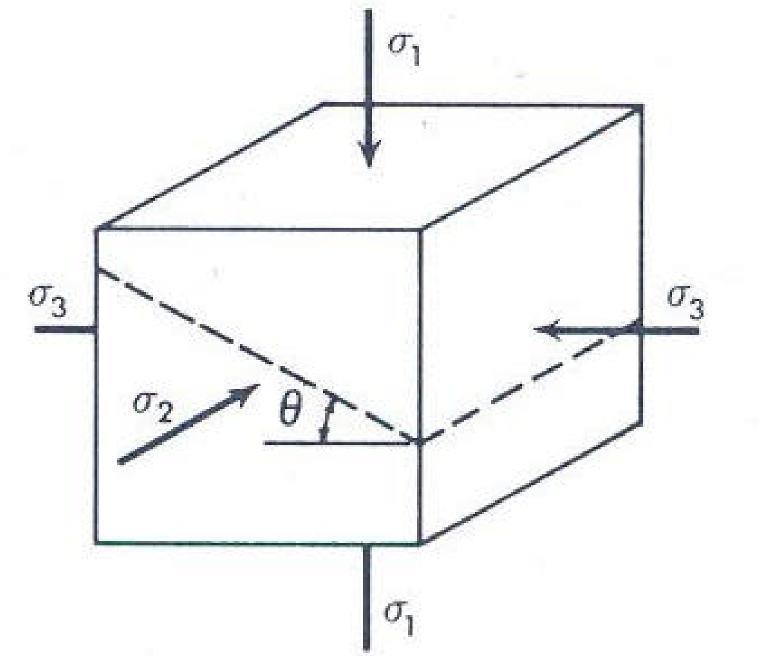
Shear and normal stresses



The normal stress s is called a *principal stress* if it acts upon a principal plane. A principal plane is any surface that does not have a shear t, and only has a normal stress. Consider a small cube of soil oriented in space such that it only has principal stresses on all of the faces. The largest principal stress is s_1, the intermediate is s_2 and the smallest is s_3. In an isostatic condition (which means, *equal loads*) all three have the same magnitude.

For example, in a hydrostatic case, $s_1 = s_2 = s_3$.





Isometric view

It is important to be able to find the relation between the principal stresses and the stresses corresponding to any other plane. In order to find the transformation between these two, consider a unit cube:

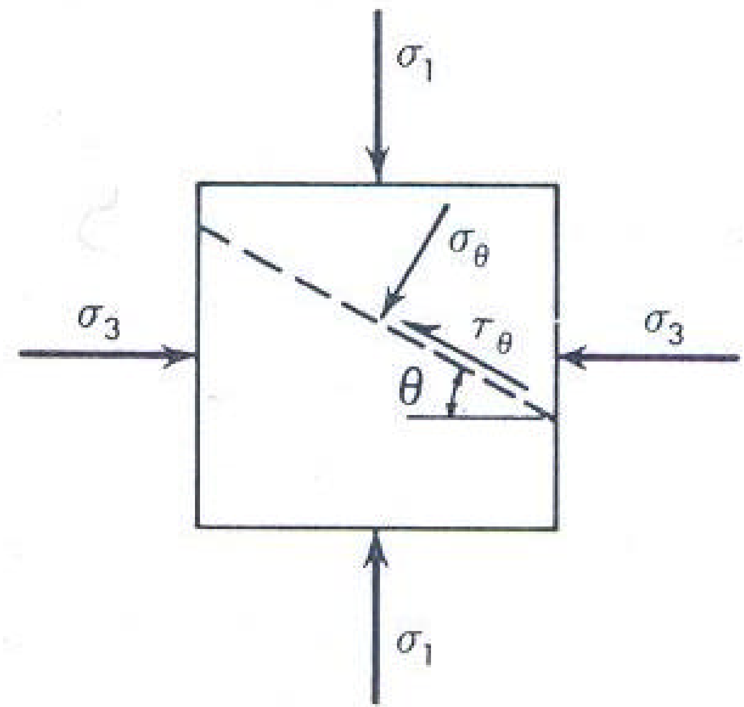
$F_1 = s_1(1x1)$ and $F_3 = s_3(1x1 \tan ?)$

$S F_N = 0 \quad F_N = F_1 \cos ? + F_3 \sin ?$

$F_N = s_1 \cos ? + s_3 \tan ? * \sin ?$

$S F_S = 0 \quad F_S = F_1 \sin ? - F_3 \cos ?$

$F_S = s_1 \sin ? - s_3 \tan ? * \cos ?$



where ? is always measured with respect to the maximum principal stress plane.

Area of ?-plane = 1 / cos ?

But $s_? = F_N$ / Area of ?-plane

$s_? = (s_1 \cos ? + s_3 \tan ? * \sin ?) / (1 / \cos ?)$

$s_? = s_1 + s_3 \sin 2?$

$\therefore \quad s_? = [(s_1 + s_3) / 2] + [((s_1 - s_3) / 2) * \cos 2?]$

and

$t_{?}$ = Fs / Area of ?-plane

$t_{?}$ = [(s $_{1}$ sin ? - s $_{3}$ tan ? * cos ?) / (1 / cos ?)]

∴ $t_{?}$ = [(s $_{1}$ - s $_{3}$) / 2] * sin 2?

Similarly, it is equally easy to find the converse operation of finding the principal stresses s_{1} and s_{3} from the general normal stress $s_{?}$ and the general shear stress $t_{?}$.

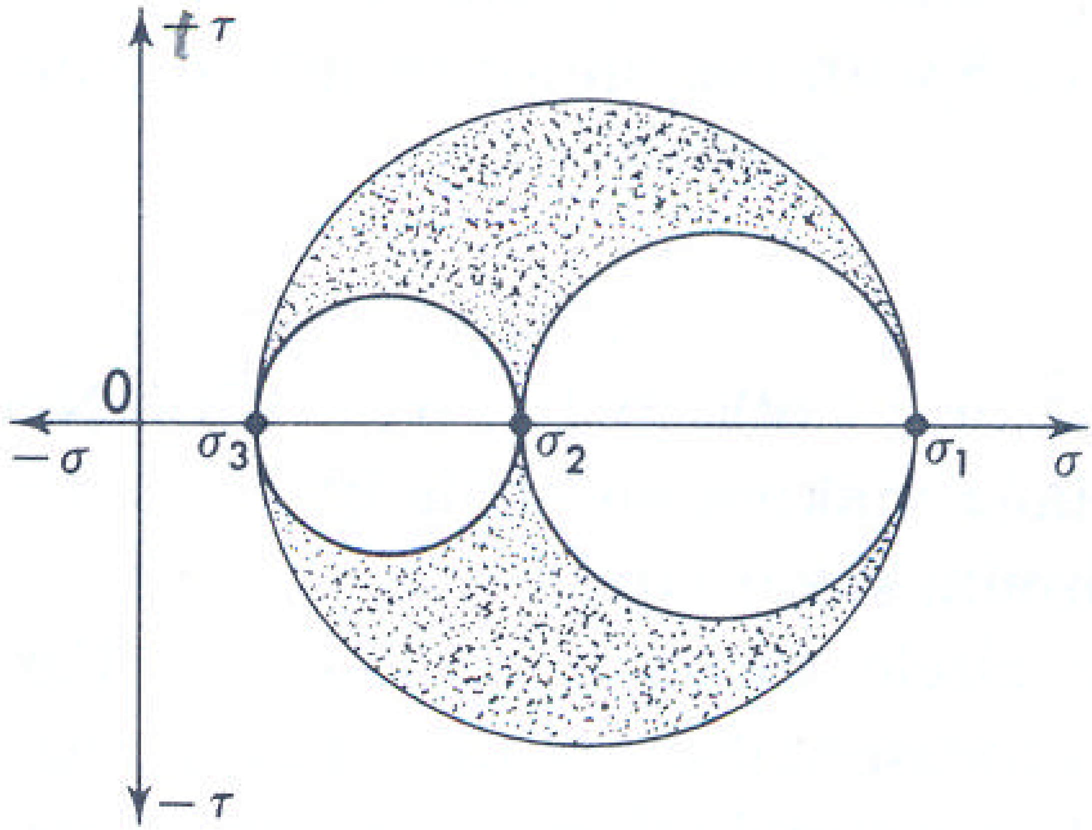
Remarks:

1) The maximum shear stress t_{max} occurs at ? = 45° with respect to the principal plane;

2) The maximum normal stress $s_{1} = s_{max}$ is at ? = 0°;

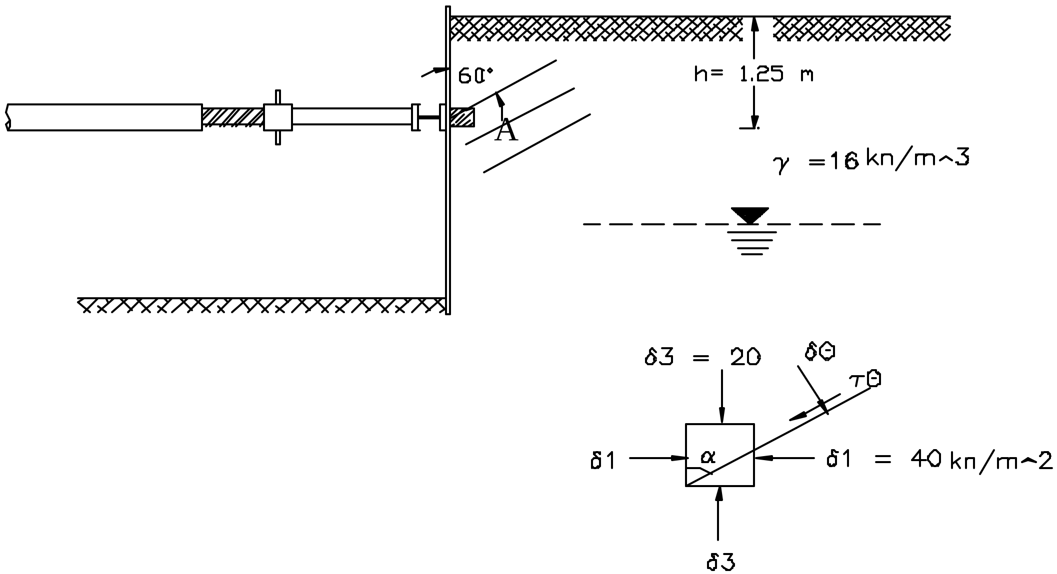
3) The minimum normal stress $s_{3} = s_{min}$ is at ? = 90°; and

4) Shear stresses are equal in magnitude on any two planes perpendicular to each other .



τ
0
−σ
σ3
σ2
σ1
σ
−τ

Example. The temporary evacuation shown below is braced with a steel tube strut. Every morning a mis-guided foreman tightens the screw mechanism on the strut “just to be safe”. The stress on a soil particle at a point A just behind the wall has been measured with a pressure sensor installed by the engineer. It now measures 40 kN/m^2. If the potential failure planes in the soil behind the wall sustain 60° angles with respect to the vertical wall, estimate the normal and shear stresses at a point A along a potential failure plane.



At point A, $\sigma_v = \gamma h = (16\ \text{kN/m}^3)(1.25\ \text{m}) = 20\ \text{kN/m}^2$

Notice that this stress is now the minor principal stress at point A. Since α = ? = 60° with respect to the the major principal stress σ_1 plane, then $\sigma_v = \sigma_3$.

Therefore $\sigma_\theta = (\sigma_1+\sigma_3)/2 + (\sigma_1-\sigma_3)/2\ \text{Cos}2\theta = (40+20)/2 + (40-20)/2\ \text{Cos}\ 120° =$ **25 kN/m^2**

and $\tau_\theta = (\sigma_1 - \sigma_3)/2\ \text{Sin}\ 2\theta = (40 - 20)/2\ \text{Sin}\ 120° =$ **8.7 kN/m^2**

Mohr's Graphical Solutions

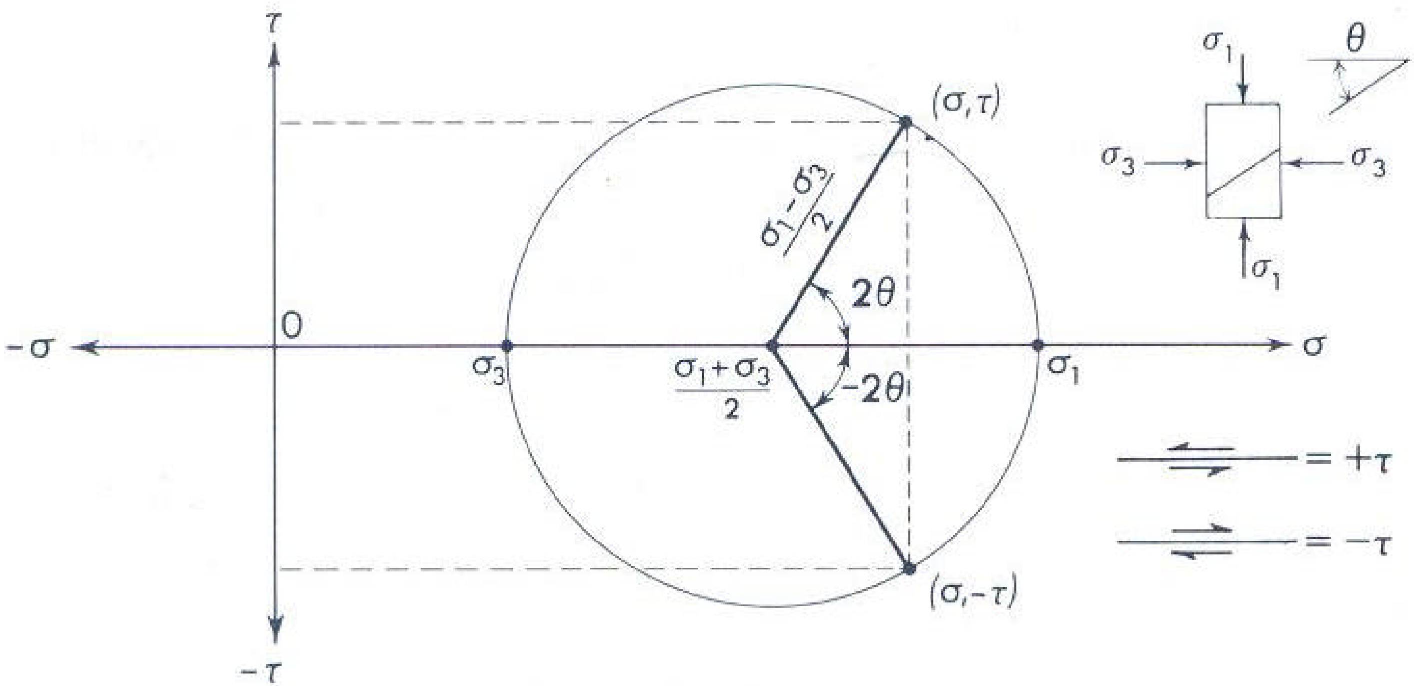
Mohr's Circle.

Mohr proposed a graphical procedure for solving these equations for shear and normal stresses on any plane. The equations,

s (x-coordinate) $S_? = [(S_1 + S_3)/2] + [(S_1 - S_3)/2] * \text{Cos } 2?$
and

t (y-coordinate) $t_? = [(S_1 - S_3)/2] * \text{Sin } 2?$

are known as the ***Mohr Transformation Equations***.



Mohr's circle of stresses

Example. Prove that ? = 45° + f /2 for a purely granular soil. A test with this soil shows that s_1 = 11.5 ksf and s_3 = 3.2 ksf at failure. Find the angle f for this sand.

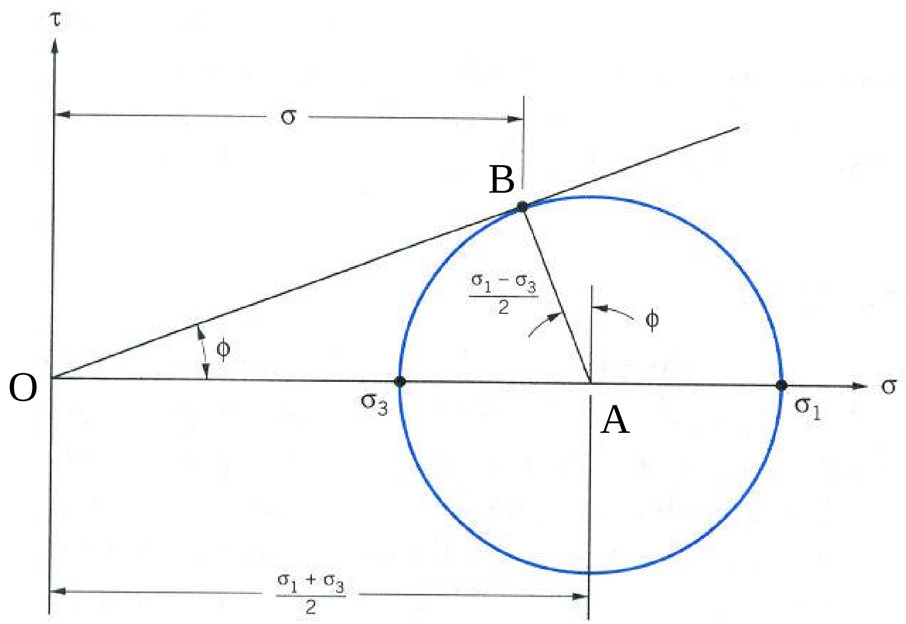
For a purely granular soil c = 0.

By inspection in ? OAB, the sum of the angles is 180°,

(180° - 2?) + 90° + f = 180°

∴ 2? = 90° + f

∴ **<u>? = 45° + f /2</u>**



In the ? OAB, the radius = 1/2 (s_1 - s_3)

∴sin f = [(1/2) (s_1 - s_3)] / [(1/2)(s_1 + s_3)]

= (11.55 - 3.2) / (11.55 + 3.2)

∴ **<u>f = 34.5°</u>**

Example. A sample of clean sand was retrieved 7 m below the surface. The sample had been under a vertical load of 150 kN/m^2, a horizontal load of 250 kN/m^2, and a shear stress of 86.6 kN/m^2. If the angle between the vertical stress and the principal stress is 60°, what is the angle of internal friction f ?

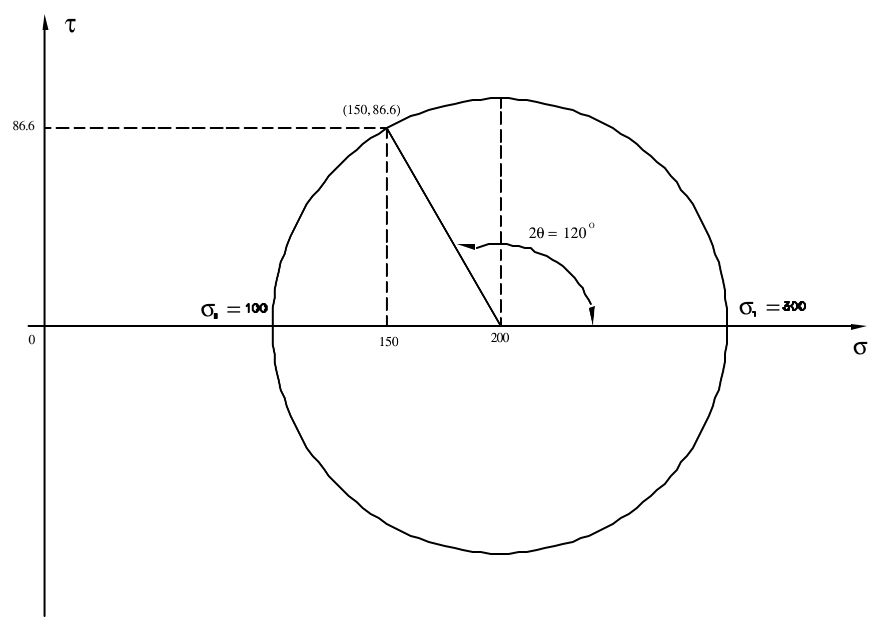
The transformation equation from principal stresses to a general state of stress is,

$$s_1 = \frac{s_V + s_H}{2} + \sqrt{\left(\frac{s_V - s_H}{2}\right)^2 + t^2} = \frac{150+250}{2} + \sqrt{\left(\frac{150-250}{2}\right)^2 + 86.6^2} = 300\ kN/m^2$$

$$\therefore\ s_1 = 300\ kN/m^2 \quad and \quad s_3 = 100\ kN/m^2$$

$$\mathrm{Sin} f = \frac{s_1 - s_3}{2} / \frac{s_1 + s_3}{2} = \frac{300-100}{2} / \frac{300+100}{2} = 0.5$$

therefore, $f = 30°$



Example. a) Derive the equation that transforms the general state of stress to the principal state of stress (hint: use Mohr's circle for a graphical solution). b) Determine the value of the major principal stresses. c) Determine the angle ? between the major principal stress and the state of stress in the figure above.

(a) By inspection :

$$s_1 = s_x + \frac{s_y + s_x}{2} + \sqrt{\left(\frac{s_y - s_x}{2}\right)^2 + t^2}$$

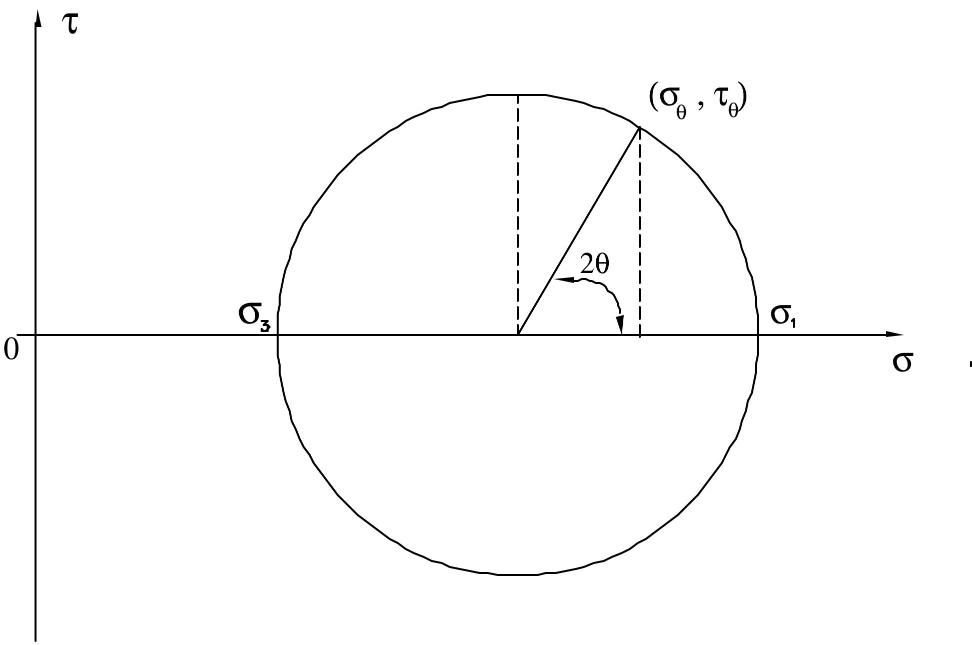
$$= \frac{s_y + s_x}{2} + \sqrt{\left(\frac{s_y - s_x}{2}\right)^2 + t^2}$$

(b) $s_1 = \frac{5+5}{2} + \sqrt{0+2^2} = 7 \text{ kPa}$

(c) $\text{Sin} f = t / \sqrt{\left(\frac{s_y - s_x}{2}\right)^2 + t^2} = 2/\sqrt{0+2^2} = 1$

$\therefore\ a = 90°$

but $a + 2q = 180° \ \therefore 2q = 90° \qquad q = 45°$



Example. A soil particle is found to be subjected to a maximum stress of 14.6 kN/m^2, and a minimum stress of - 4.18 kN/m^2. Find the s and the t on the plane of ? = 50° with respect to the major principal stresses, and t$_{max}$.

The calculated solution is,

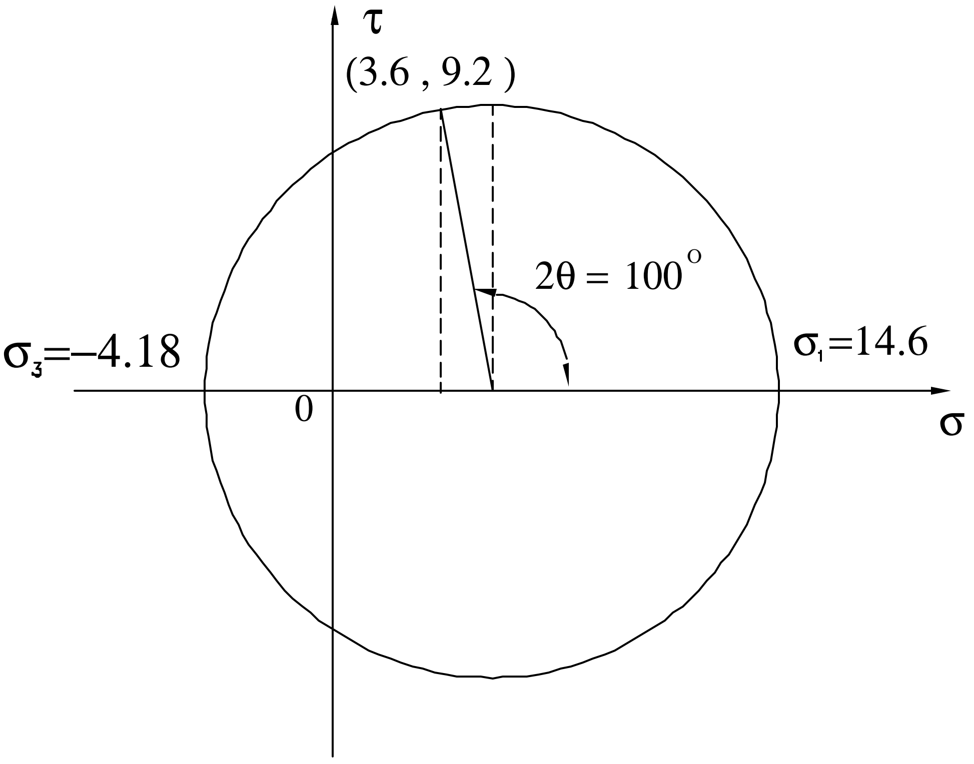
$$s_q = \frac{14.6 - 4.18}{2} + \frac{14.6 + 4.18}{2} * Cos100°$$

$$= 3.6\ kN/m^2$$

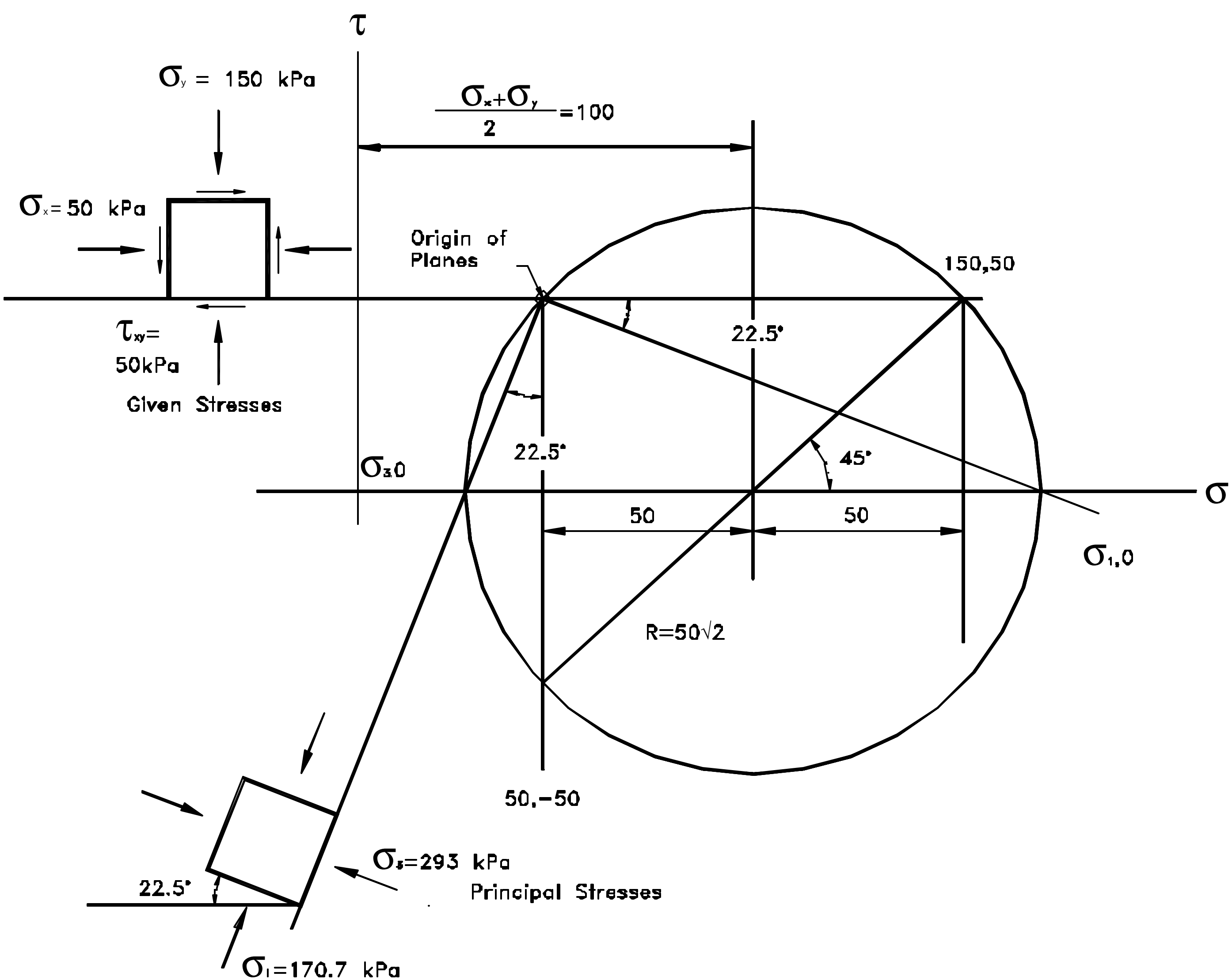
$$t_q = \frac{14.6 + 4.18}{2} * Sin100°$$

$$= 9.2\ kN/m^2$$

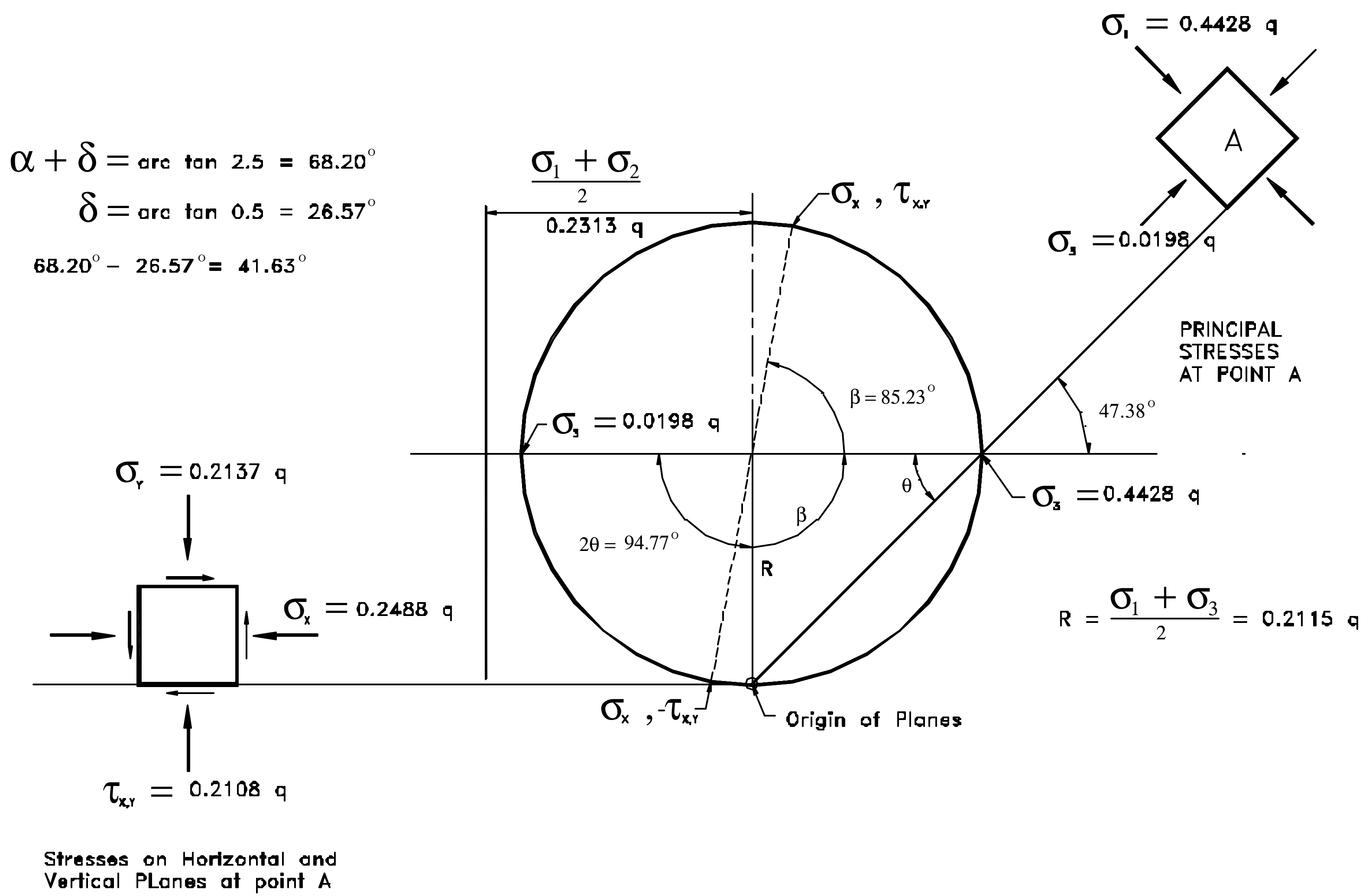
$$t_{max} = \frac{14.6 + 4.18}{2} = 9.4\ kN/m^2$$



Example. Given the stresses at a point in a soil, determine the principal stresses and show them on a properly oriented element.



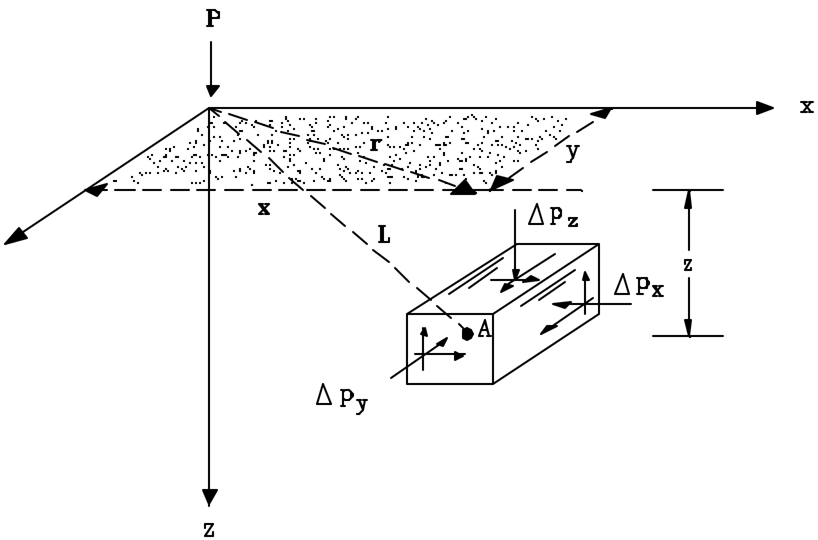
Example. Calculate the vertical stress σ_y, the horizontal stress σ_x, and the shear τ_{xy} at point A if x = 0.75B and y = 0.50B using Mohr's diagram.



Boussinesq's Mathematical Solutions

Stresses due to a Point Load.

Boussinesq published in 1883 a mathematical solution to the problem of finding the stress at any point in a homogeneous, elastic and isotropic medium due to a vertical point load applied upon the surface of an semi-infinitely large space, as shown below.



$$\Delta p_x = \frac{P}{2p}\left[\frac{3x^2 z}{L^5} - (1-2m)\left(\frac{x\mathrm{SUP2} - y^2}{Lr^2(L+z)} + \frac{y^2 z}{L^3 r^2}\right)\right]$$

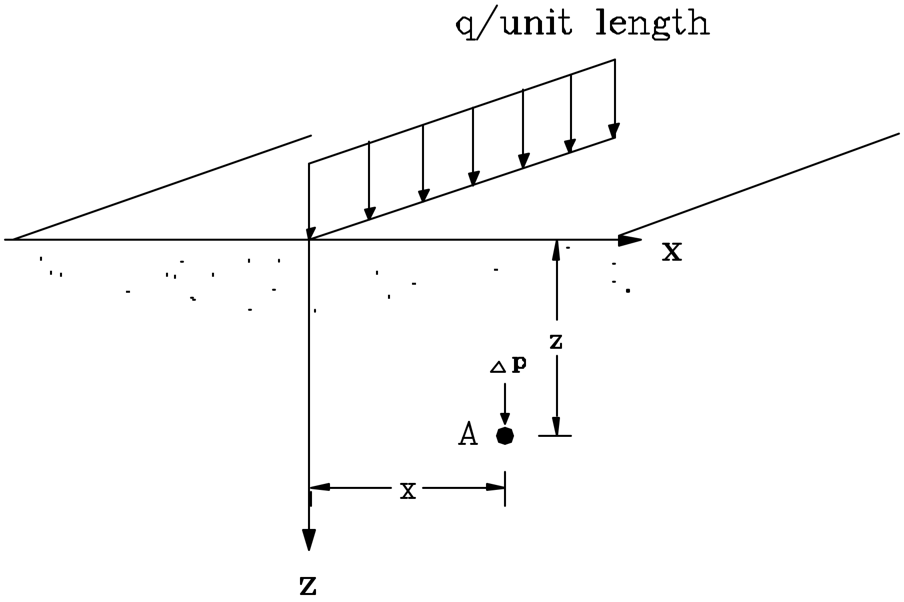
$$\Delta p_y = \frac{P}{2p}\left[\frac{3y^2 z}{L^5} - (1-2m)\left(\frac{y\mathrm{SUP2} - x^2}{Lr^2(L+z)} + \frac{x^2 z}{L^3 r^2}\right)\right]$$

$$\Delta p_z = \frac{3P}{2p}\frac{z^3}{L^5} = \frac{3P}{2p}\frac{z^3}{(r\mathrm{SUP2} + z^2)^{5/2}}$$

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

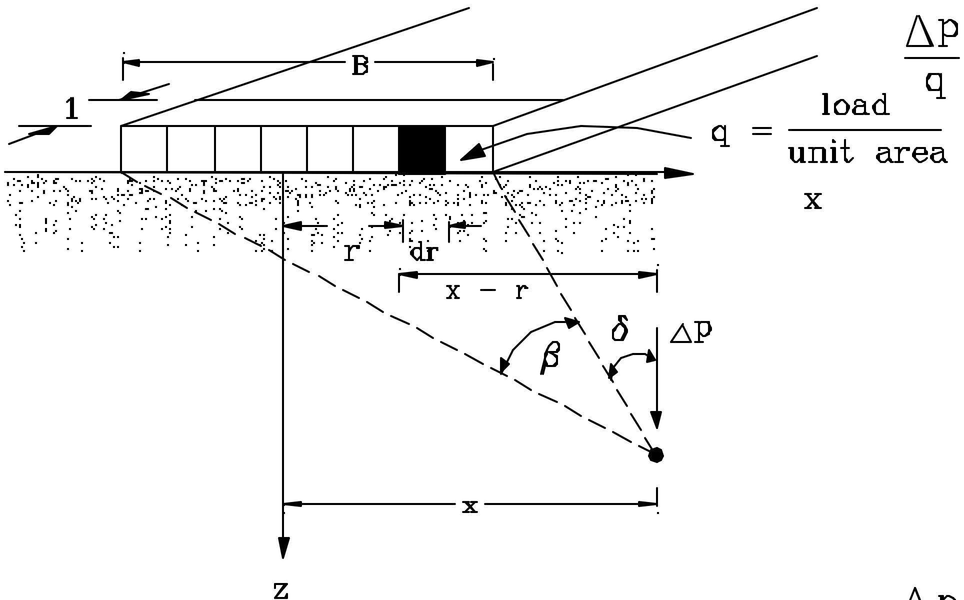
Vertical Stress due to a Line Load.

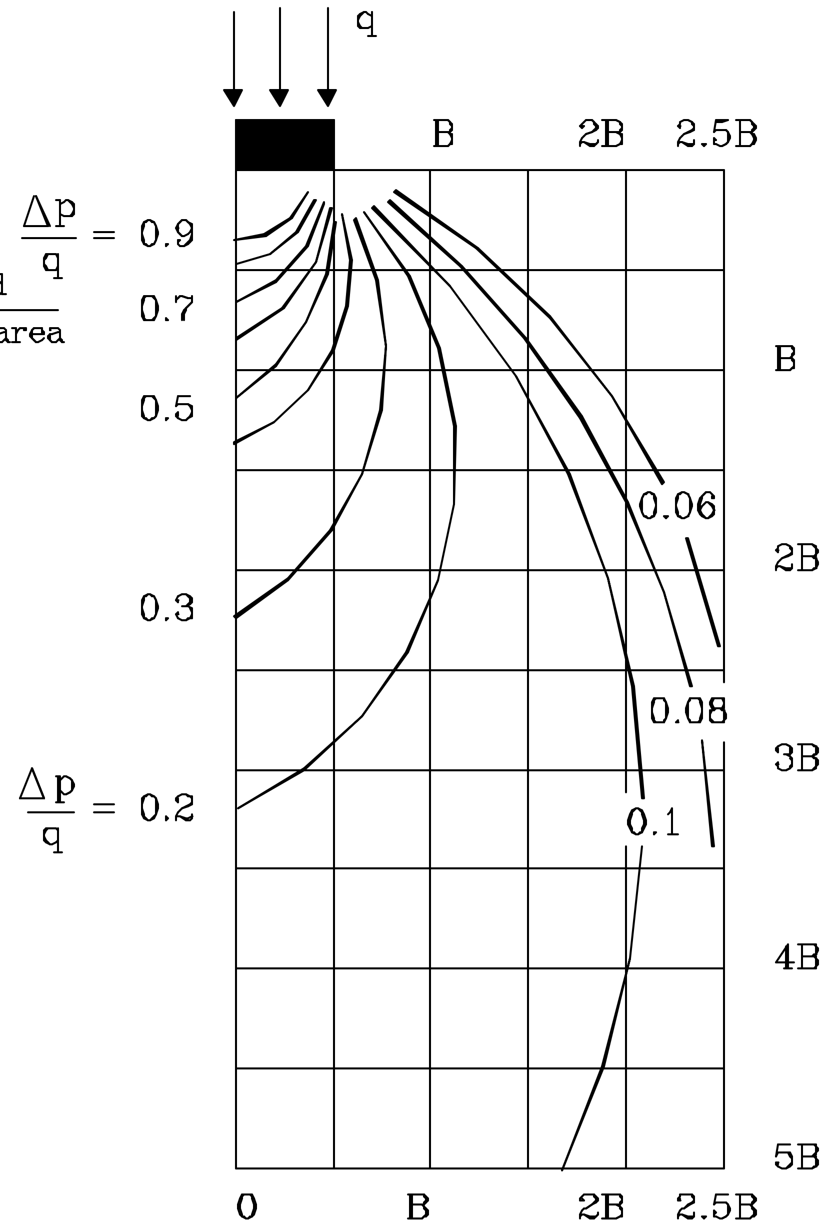
$$\Delta p = \frac{2qz^3}{p(x^2 + z^2)^2}$$



Use superposition for 2 or more links.

Vertical Stress due to a Strip Load.





$$\int dp_z = \int_{-B/2}^{+B/2} \frac{2(q \bullet dr)\, z^3}{p\ [\ (x - r\ )^3 + z^2\ ]^2}$$

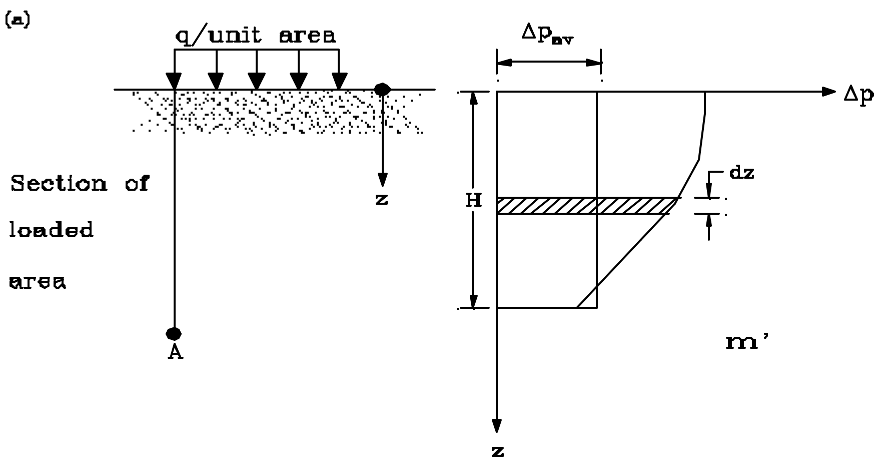
$$\Delta q_z = \frac{q}{p}\,[\, b + \sin b\ \cos(\, b + 2d\ )]$$

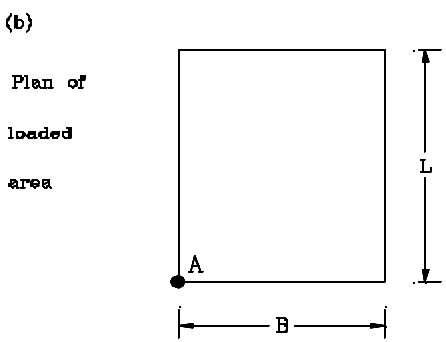
Note: Isobars are for line a – a as shown on the plan.

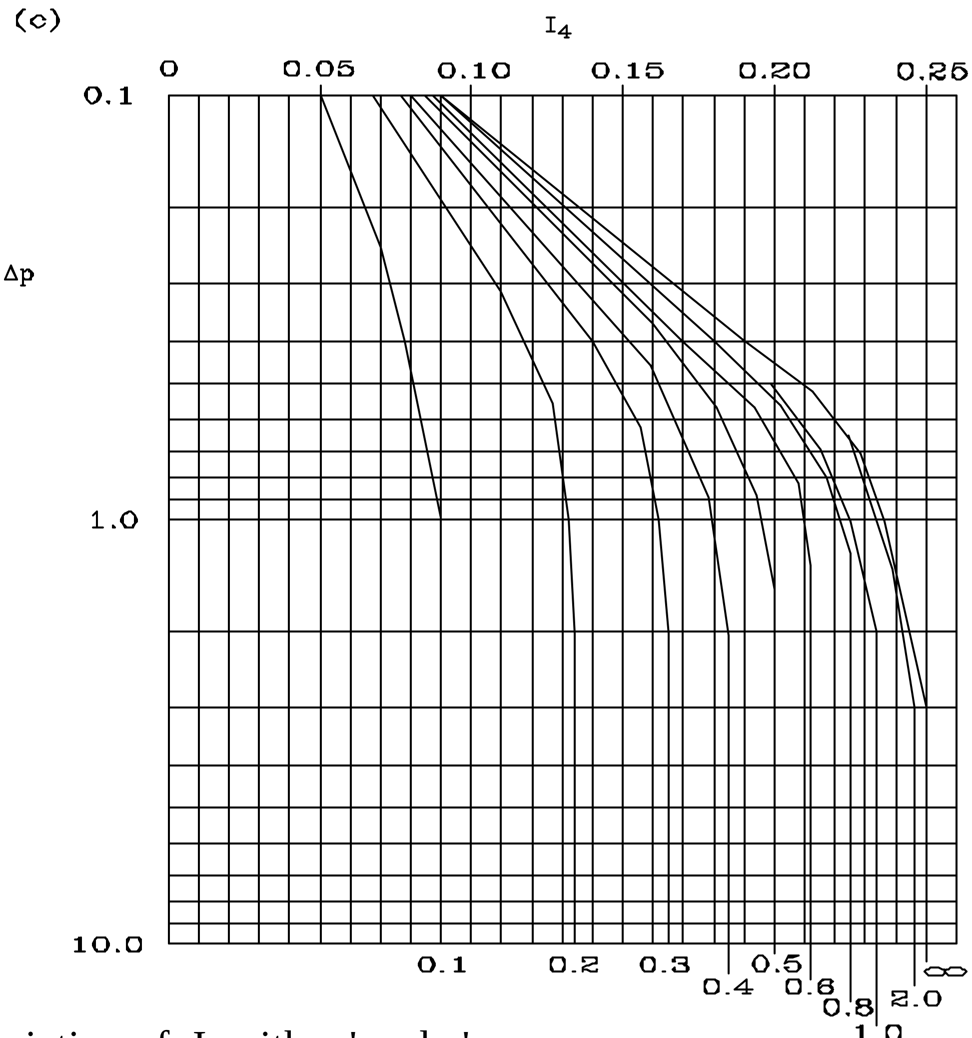
Average Vertical Stress Increase from Rectangular Load.

$$\Delta p_{av} = \frac{1}{H}\int_{O}^{H} (qI_3)\,dz = qI_4$$

where $I_4 = f(m', n')$ $\qquad m' = \frac{B}{H} \qquad n' = \frac{L}{H}$



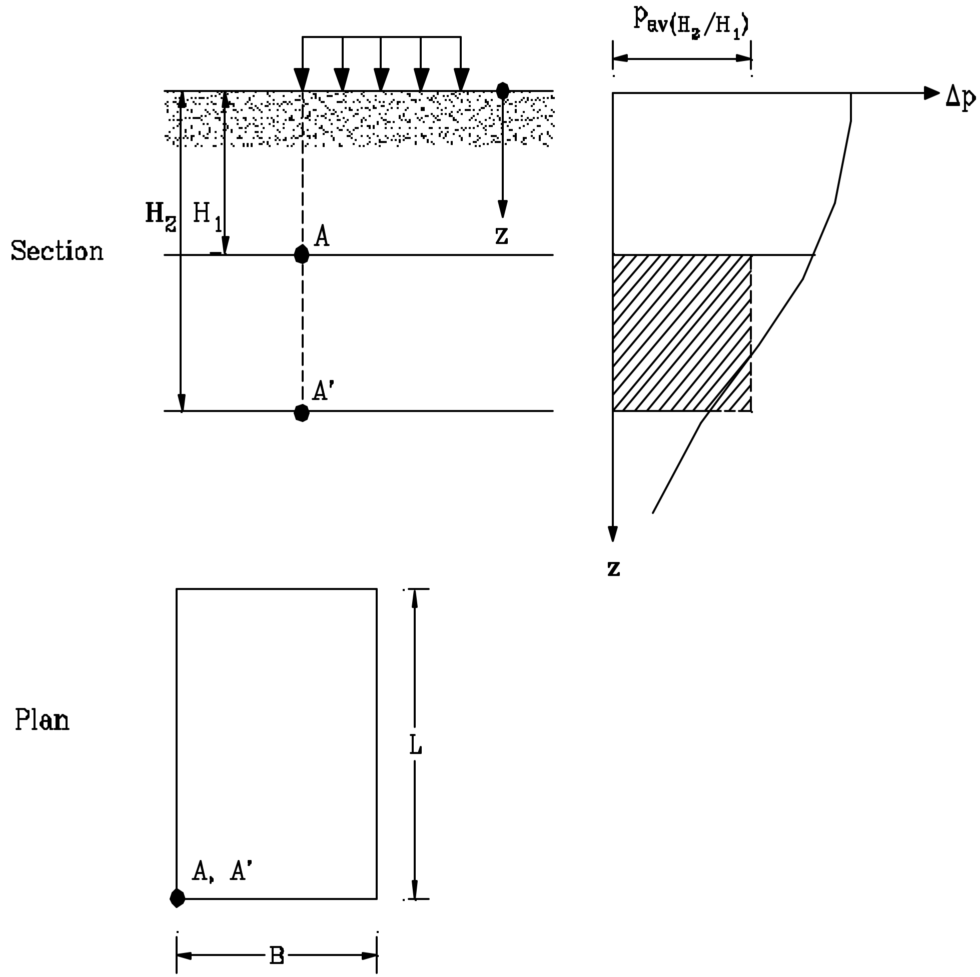


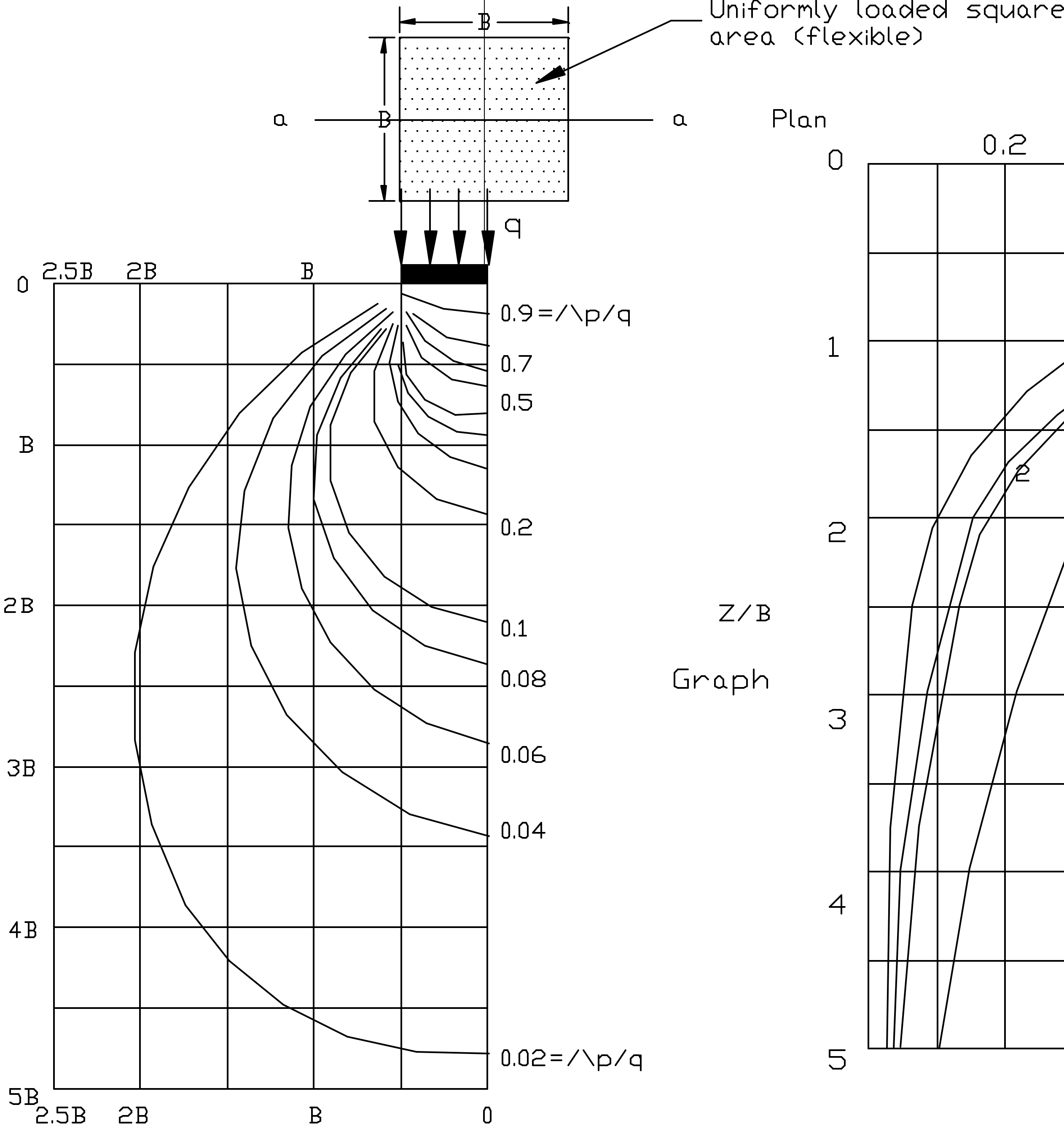


Variation of I_4 with m' and n'

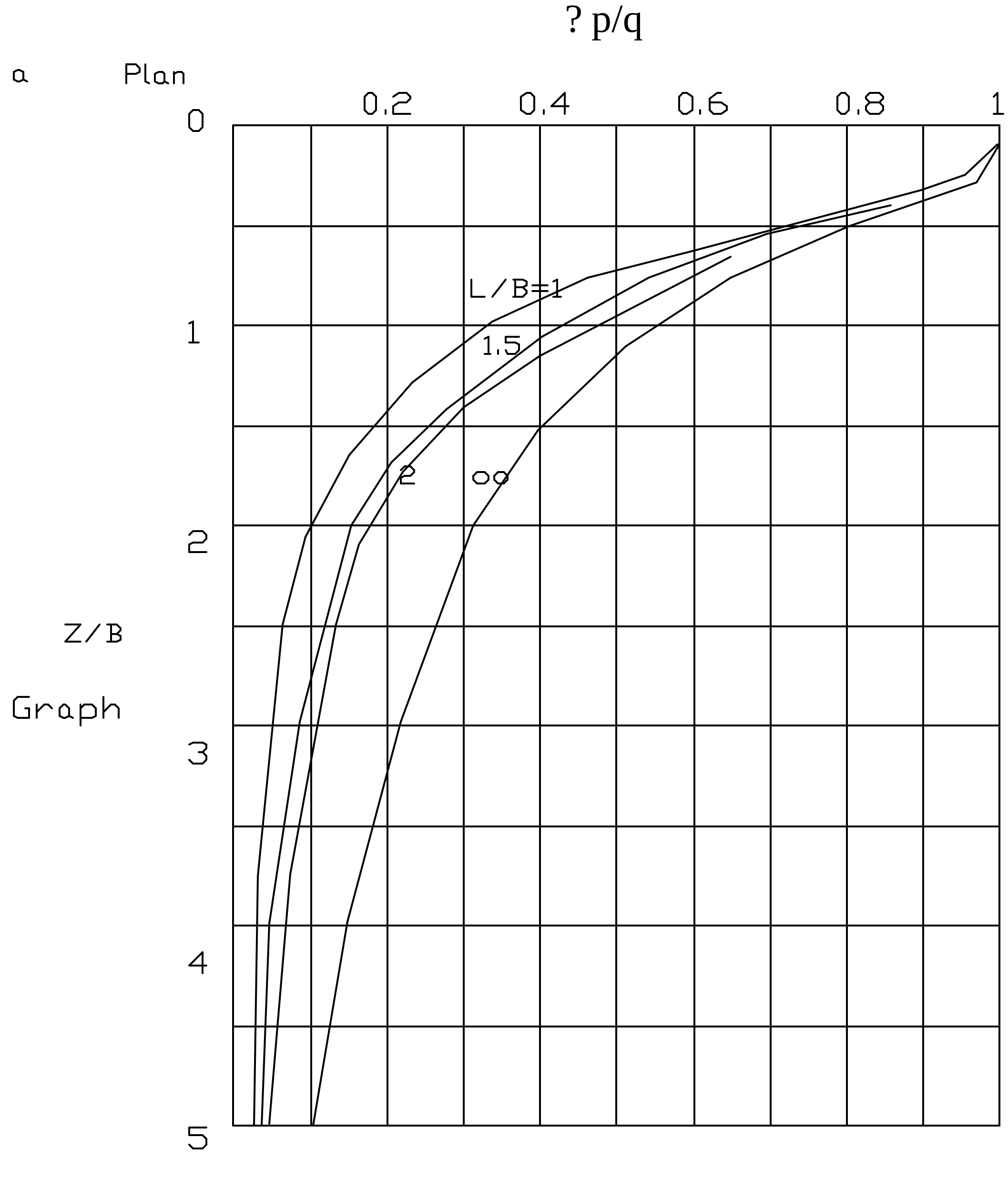
When estimating the consolidation settlement under a foundation, it may be required to determine the average vertical stress increase in only a given layer, that is between $z = H_1$ to $z = H_2$, as shown below. (Reference Griffiths, 1984).

$$\Delta p_{av(H_2{}'H_1)} = q \frac{H_2 I_{4(H_2)} - H_1 I_{4(H_1)}}{H_2 - H_1}$$



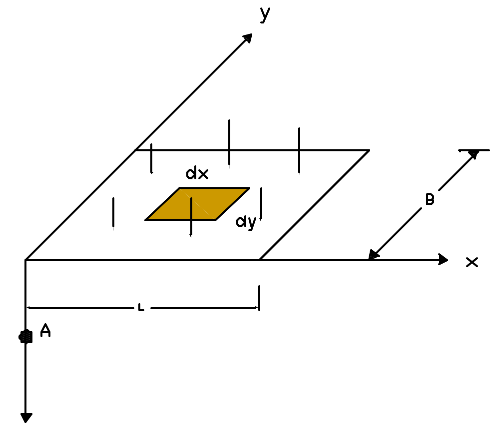


Vertical pressure isobars under a uniformly loaded square area.



Stress increase under the center of a uniformly loaded rectangular flexible area.

The Vertical Stress due to a Rectangularly Loaded Area.



The vertical stress at point A is due to the contribution of each dq of area (dx dy).

$$or\ dq = q\,dx\,dy \qquad dp = \frac{3q\,dx\,dy\,z^3}{2p(x^2+y^2+z^2)^{5/2}}$$

The increase of stress (*dp*) at point A due to the load *dq* can be determined by using the equation above. However, we need to replace the load *P* with *dq = q dx dy* and r_2 with $x_2 + y_2$. Thus, the increase of stress ? p at A due to the entire loaded area can now be determined by integrating the *(dp)* equation. The stress at any point A below the corner of the area,

$$m = \frac{B}{z} \qquad n = \frac{L}{z} \qquad where\ I_3 = \frac{1}{4p}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right)+\tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2-m^2n^2+1}\right)\right]$$

Since Boussinesq's solution for this problem is only provided below a corner of the rectangular area, a solution must be found for the stress anywhere below the loaded area. The technique commonly used is to produce a "virtual" cut of the rectangle into four pieces. The point of interest is the intersection of the cuts. The stress is now found using Boussinesq for each of the four new pieces, and adding the contribution of each using the Principle of Superposition. The stress increase,

$? p_v = q[\ I_3(\text{area 1}) + I_3(\text{area 2}) + I_3(\text{area 3}) + I_3(\text{area 4})]$ where I_3 is the influence of each area.

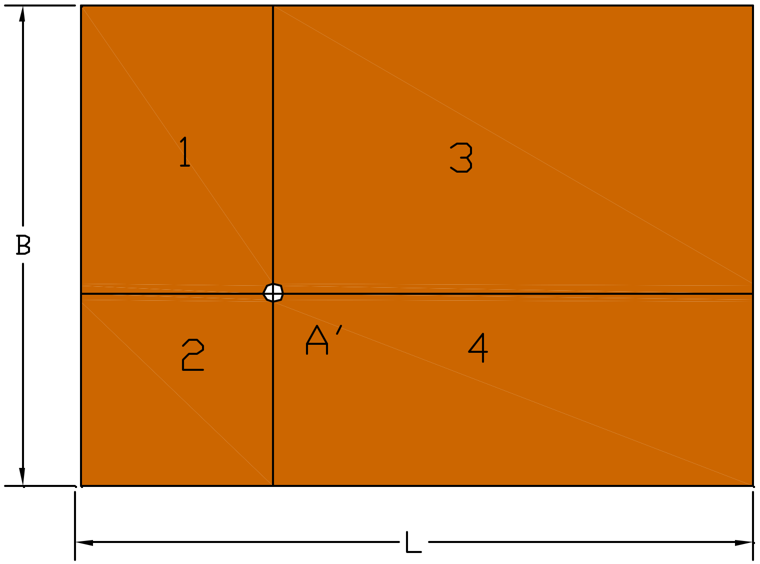
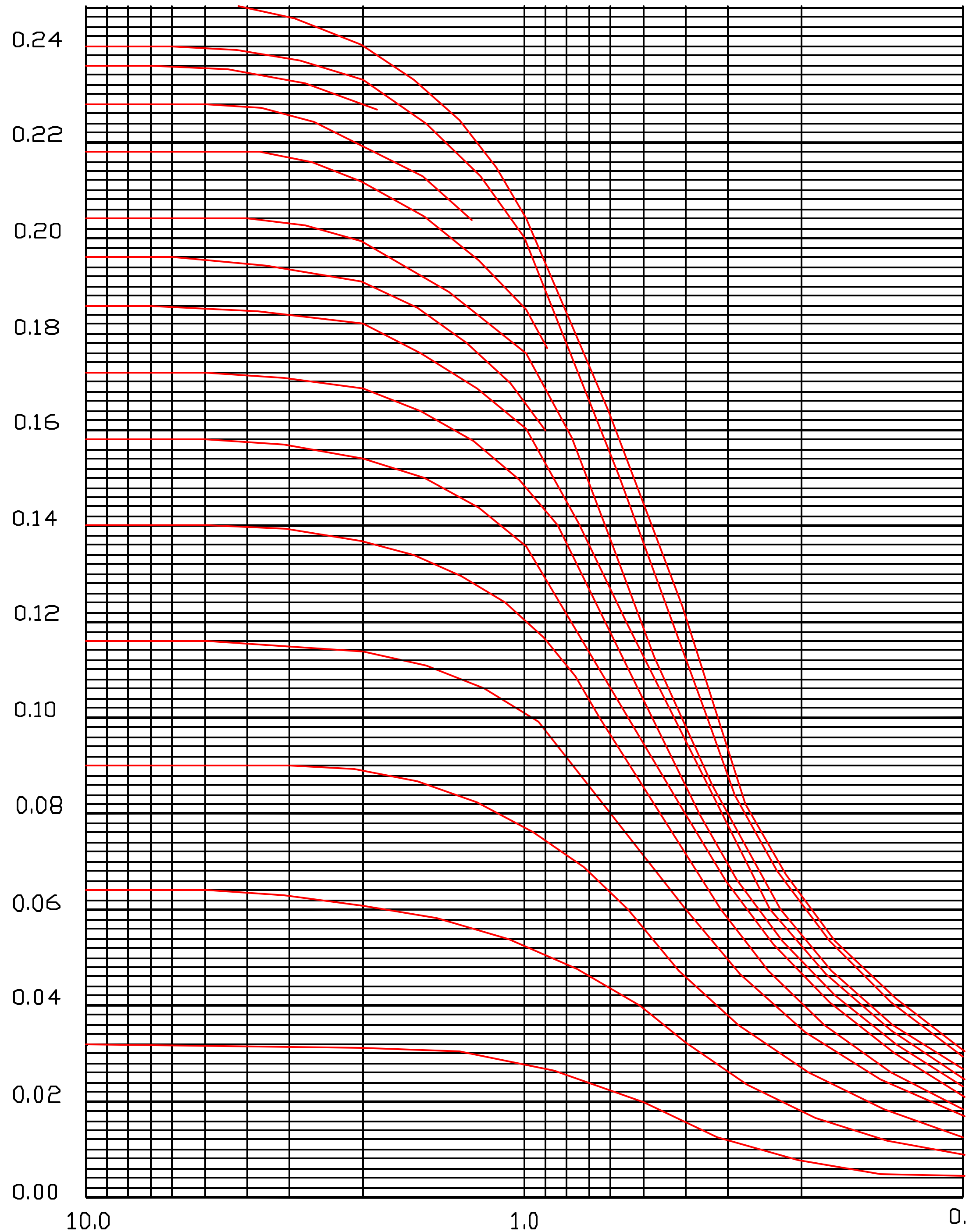


Table with the variation of I_3 with *m* and *n* for a rectangular loaded area.

	m																			
n	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	4.0	5.0	6.0
0.1	0.0047	0.0092	0.0132	0.0168	0.0198	0.0222	0.0242	0.0258	0.0270	0.0279	0.0293	0.0301	0.0306	0.0309	0.0311	0.0314	0.0315	0.0316	0.0316	0.0316
0.2	0.0092	0.0179	0.0259	0.0387	0.0387	0.0435	0.0474	0.0504	0.0528	0.0547	0.0573	0.0589	0.0599	0.0606	0.0610	0.0616	0.0618	0.0619	0.0620	0.0620
0.3	0.0132	0.0259	0.0374	0.0474	0.0559	0.0629	0.0686	0.0731	0.0766	0.0794	0.0832	0.0856	0.0871	0.0880	0.8870	0.0895	0.0898	0.0901	0.0901	0.0902
0.4	0.0168	0.0328	0.0474	0.0602	0.0711	0.0801	0.0873	0.0931	0.0977	0.1013	0.1063	0.1094	0.1140	0.1126	0.1134	0.1145	0.1150	0.1153	0.1154	0.1154
0.5	0.0198	0.0387	0.0559	0.0711	0.0840	0.0947	0.1034	0.1104	0.1158	0.1202	0.1263	0.1300	0.1324	0.1340	0.1350	0.1363	0.1368	0.1372	0.1374	0.1374
0.6	0.0222	0.0435	0.0629	0.0801	0.0947	0.1069	0.1168	0.1247	0.1311	0.1361	0.1431	0.1475	0.1503	0.1521	0.1533	0.1548	0.1555	0.1560	0.1561	0.1562
0.7	0.0242	0.0474	0.0686	0.0873	0.1034	0.1169	0.1277	0.1365	0.1436	0.1491	0.1570	0.1620	0.1652	0.1672	0.1686	0.1704	0.1711	0.1717	0.1719	0.1719
0.8	0.0258	0.0504	0.0731	0.0931	0.1104	0.1247	0.1365	0.1461	0.1537	0.1598	0.1684	0.1739	0.1774	0.1797	0.1812	0.1832	0.1841	0.1847	0.1849	0.1850
0.9	0.0270	0.0528	0.0766	0.0977	0.1158	0.1311	0.1436	0.1537	0.1619	0.1684	0.1777	0.1836	0.1874	0.1899	0.1915	0.1938	0.1947	0.1954	0.2956	0.1957
1.0	0.0279	0.0547	0.0794	0.1013	0.1202	0.1361	0.1491	0.1598	0.1684	0.1752	0.1851	0.1914	0.1955	0.1981	0.1999	0.2024	0.2034	0.2042	0.2044	0.2045
1.2	0.0293	0.0573	0.0832	0.1063	0.1263	0.1431	0.1570	0.1684	0.1777	0.1851	0.1958	0.2028	0.2073	0.2103	0.2124	0.2151	0.2163	0.2172	0.2175	0.2176
1.4	0.0301	0.0589	0.0856	0.1094	0.1300	0.1475	0.1620	0.1739	0.1836	0.1914	0.2028	0.2102	0.2151	0.2184	0.2206	0.2236	0.2250	0.2260	0.2263	0.2264
1.6	0.0306	0.0599	0.0871	0.1114	0.1324	0.1503	0.1652	0.1774	0.1874	0.1955	0.2073	0.2151	0.2203	0.2237	0.2261	0.2294	0.2309	0.2320	0.2323	0.2325
1.8	0.0309	0.0606	0.0880	0.1126	0.1340	0.1521	0.1672	0.1797	0.1899	0.1981	0.2103	0.2183	0.2237	0.2274	0.2299	0.2333	0.2350	0.2362	0.2366	0.2367
2.0	0.0311	0.0610	0.0887	0.1134	0.1350	0.1533	0.1686	0.1812	0.1915	0.1999	0.2124	0.2206	0.2261	0.2299	0.2325	0.2361	0.2378	0.2391	0.2395	0.2397
2.5	0.0314	0.0616	0.0895	0.1145	0.1363	0.1548	0.1704	0.1832	0.1938	0.2024	0.2151	0.2236	0.2294	0.2333	0.2361	0.2401	0.2420	0.2434	0.2439	0.2441
3.0	0.0315	0.0618	0.0898	0.1150	0.1368	0.1555	0.1711	0.1841	0.1947	0.2034	0.2163	0.2250	0.2309	0.2350	0.2378	0.2420	0.2439	0.2455	0.2461	0.2463
4.0	0.0316	0.0619	0.0901	0.1153	0.1372	0.1560	0.1717	0.1847	0.1954	0.2042	0.2172	0.2260	0.2320	0.2362	0.2391	0.2434	0.2455	0.2472	0.2479	0.2481
5.0	0.0316	0.0620	0.0901	0.1154	0.1374	0.1561	0.1719	0.1849	0.1956	0.2044	0.2175	0.2263	0.2324	0.2366	0.2395	0.2439	0.2460	0.2479	0.2486	0.2489
6.0	0.0316	0.0620	0.0902	0.1154	0.1374	0.1562	0.1719	0.1850	0.1957	0.2045	0.2176	0.2264	0.2325	0.2367	0.2397	0.2441	0.2463	0.2482	0.2489	0.2492



0.24
0.22
0.20
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
0.00
10.0
1.0
0.1

Example. Determine the stress increase in a soil at a depth of 6 m, caused by a newly built spread footing, 3 m x 4 m, located on the ground surface, with a columnar axial load of N = 1800 kN.

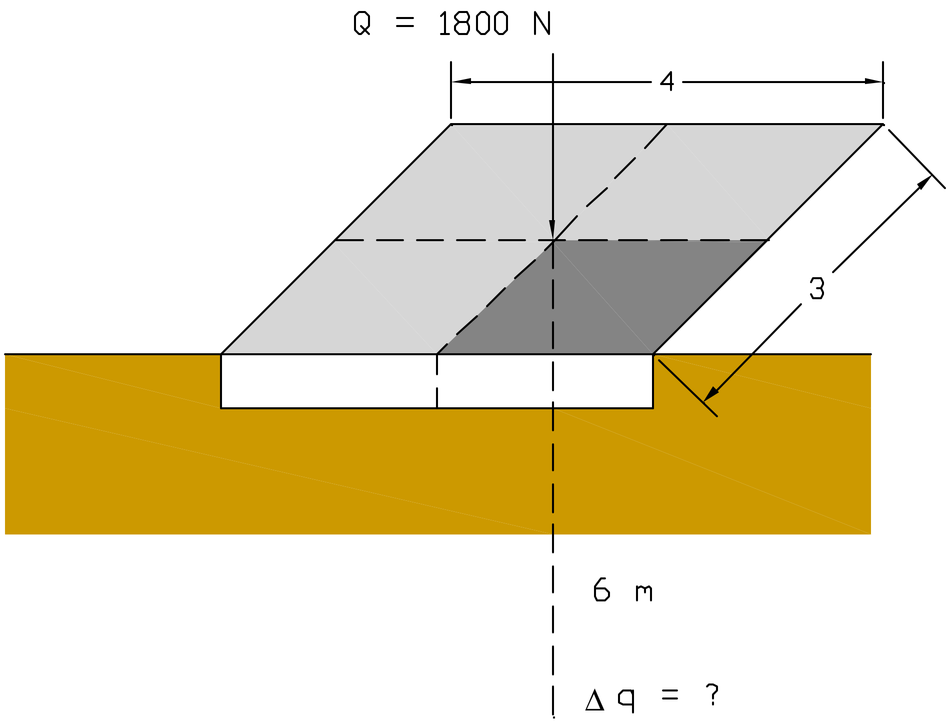
Reduced parameters for shaded area:

B_1 = 1.50 m and L_1 = 2.00 m

Therefore m = B/z = 1.50/6.00 = 0.25
and n = L/z = 2.00/6.00 = 0.33

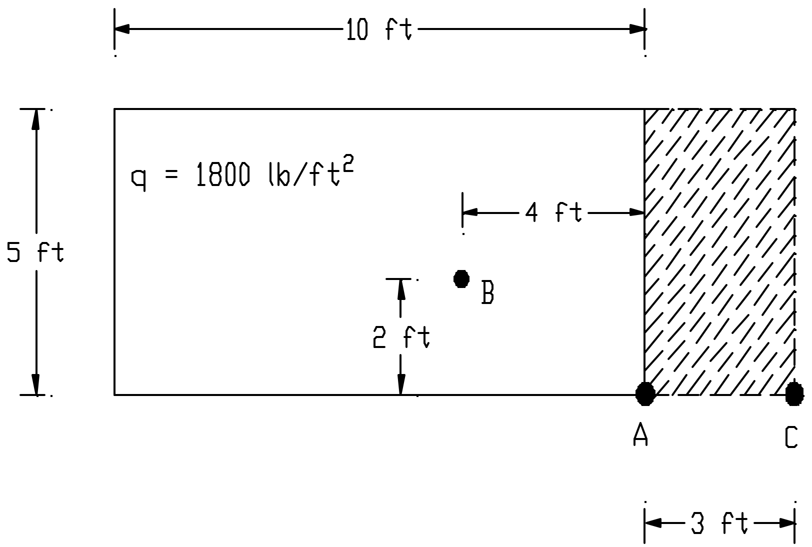
These parameters correspond to an I_3 =0.03444
and since each area has the same I_3,

? q = $q_o(4I_3)$ = (1800 kN)(4)(0.03444) / (3)(4)
? q = 20.7 kPa



∴

Example. For the flexible footing shown below, determine the increase in the vertical stress ? p at a depth of z = 5 feet below the point C, for the uniformly distributed surface load q.



To solve the problem, expand the footing to reach the point C. The "new" footing is a 13' by 5'. The influence value I_3 is found for this condition,

m = B/z = 5/5 = 1
n = L/z = 13/5 = 2.6

therefore I_3 = 0.200

Now the shaded expanded area of 3'x5" is also analyzed and then subtracted from the previous result.

m = B/z = 3/5 = 0.6
n = L/z = 5/5 = 1
therefore I_3' = 0.137

Therefore ? p = q(I_3 - I_3') = (1800 lb/ft^2)(0.200 - 0.137)
= **117 psf**

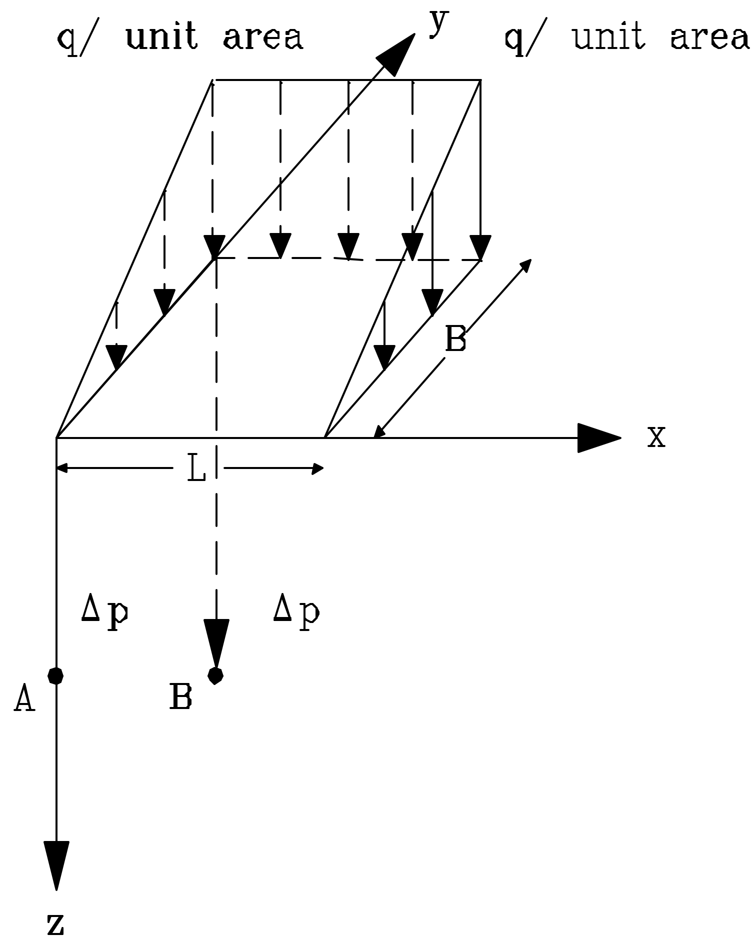
Vertical Stress due to a Linearly Increasing Load.

The stress under point A:

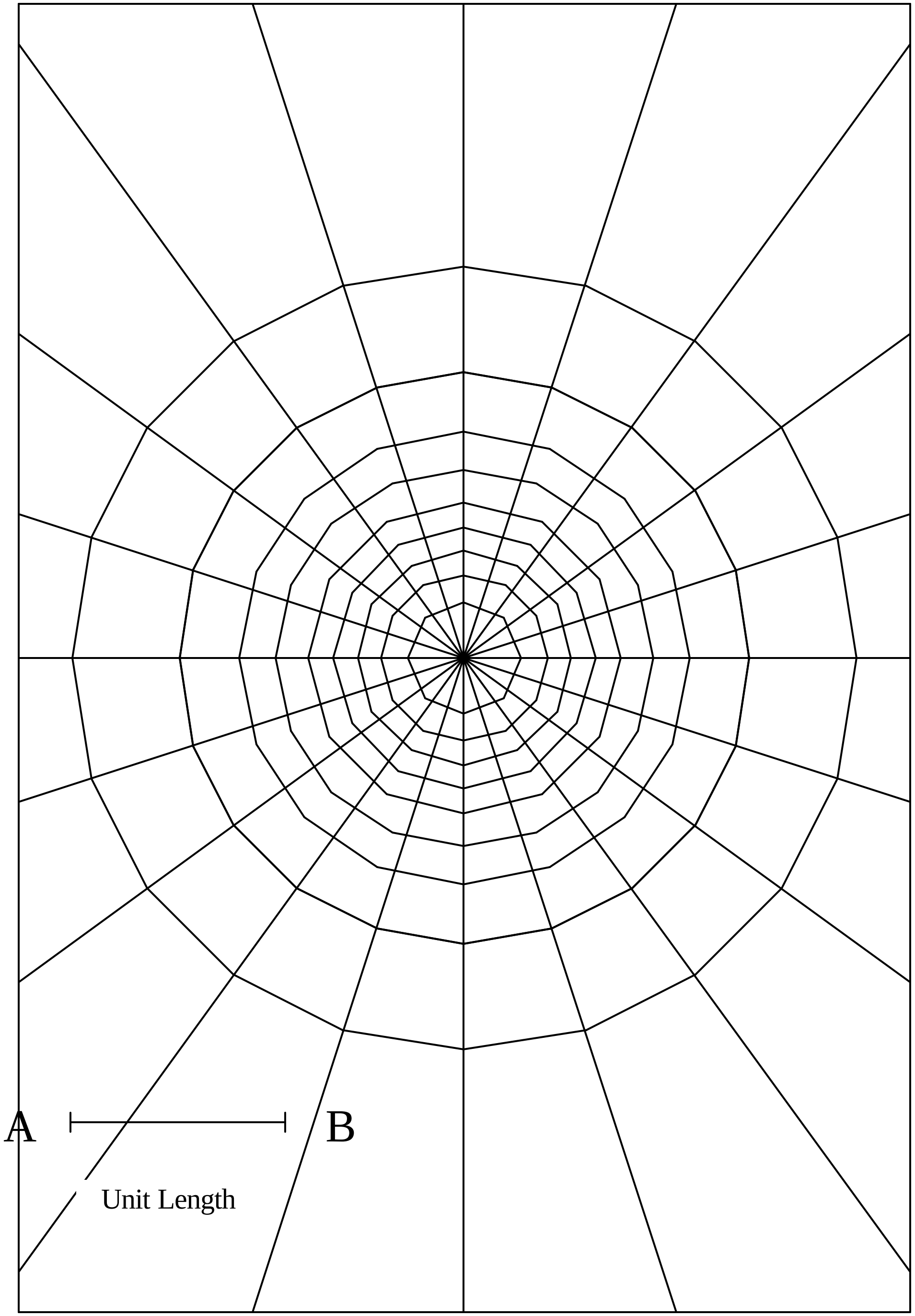
$$\Delta p = \frac{q}{2p} \frac{Bz}{z^2 + B^2}$$

The stress under point B:

$$\Delta p = \frac{q}{2p} \left[\frac{Bz}{z^2 + B^2} + \frac{p}{2} - \tan^{-1}\left(\frac{z}{B}\right) \right] - \frac{q}{2p} \left(\frac{Bz}{z^2 + B^2} \right)$$



Newmark's Influence Chart Solutions



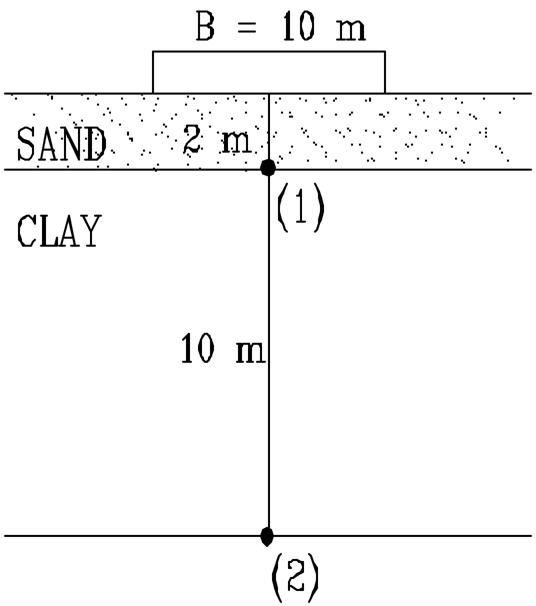
Newmark proposed a graphical solution to Boussinesq's equations by transforming them into a uniformly loaded flexible circular area.

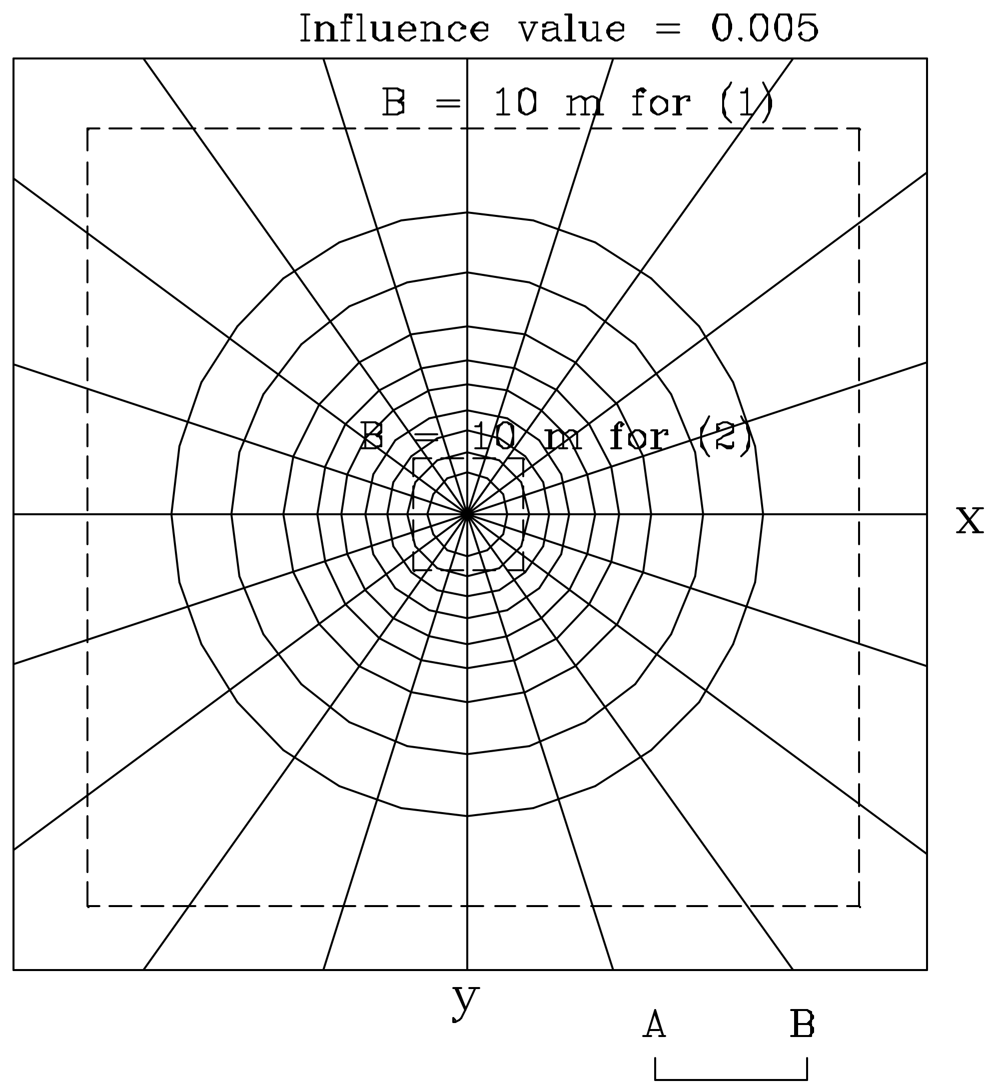
This influence chart provides the vertical pressures based on Boussinesq's theory (adapted from Newmark, 1942).

Influence value of each area = 0.005

Influence value = 0.005 comes from $\frac{1}{N} = \frac{1}{200}$ elements

Example. A site has a surface layer of aeolic sand, 2 m thick, underlaid by a 10 m thick clay stratum. The project involves placing a wastewater treatment tank, 10 m square with a contact pressure of 400 kN/m^2. Find the stress at mid-tank, at the top and the bottom of the clay stratum using Newmark's influence chart.





For ? p_1, AB = 2 m.

? p_1 = (IV)p_0M = (0.005)(400 kN/m^2)(190)

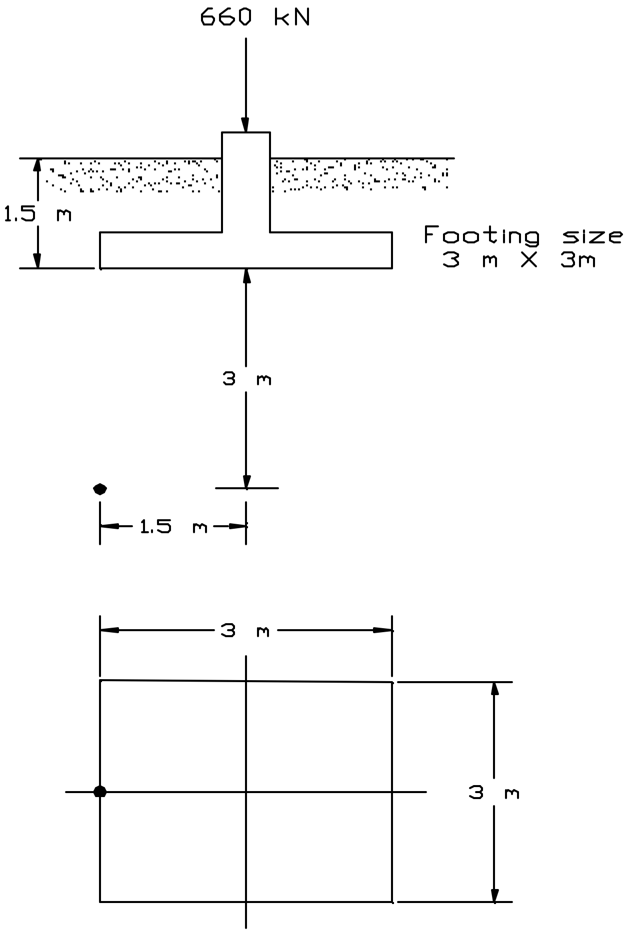
? p_1 = 380 kN/m^2

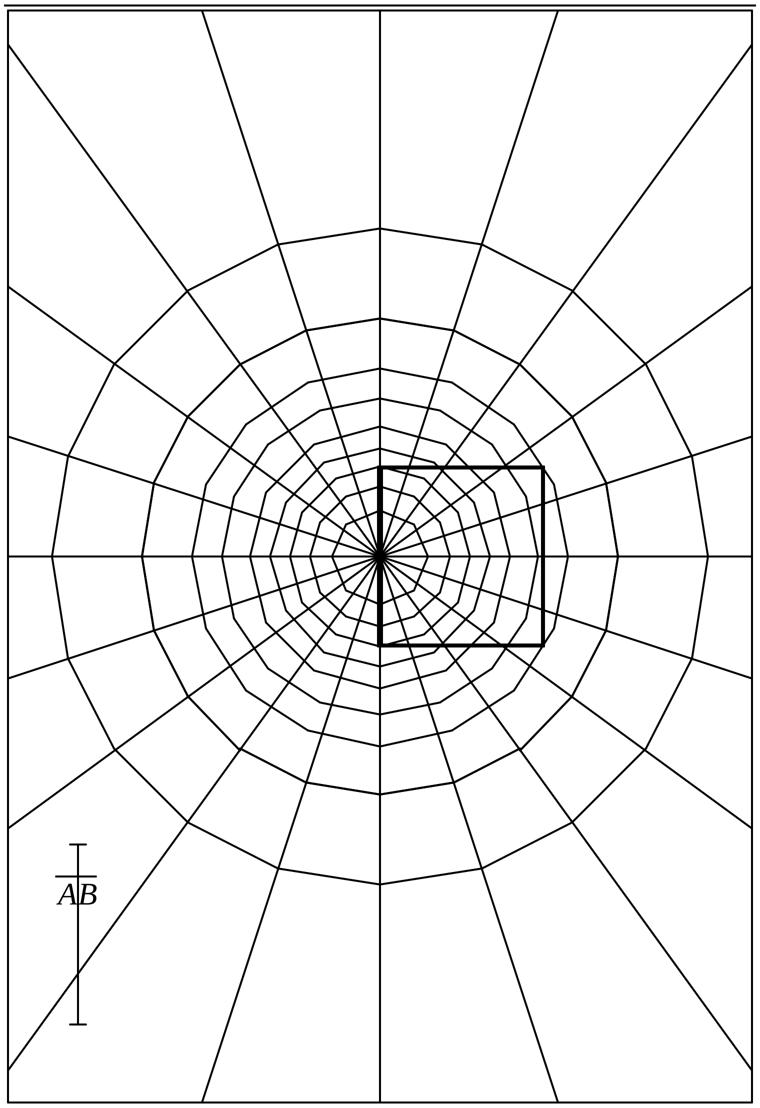
For ? p_2, AB = 12 m.

? p_2 = (0.005)(400 kN/m^2}(42)

? p_2 = 84 kN/m^2

Example. Find the stress at a point *A* located at a depth of 3 m below the bottom of the footing.





Influence value = 0.005

The number of elements inside the outline of the plan is about 48.5. Hence,

$$\Delta p = (IV)qM = 0.005\left(\frac{660}{3\times 3}\right)48.5 = 17.78 kN / m^2$$

Example. A circular oil storage tank is 20 m in diameter, and 15 m high. The tank sits upon a 2 m thick sand deposit, that rests upon a clay stratum 16 m thick. The water table is practically at the surface. Find the stress increase from a fully loaded tank at mid-clay stratum, (a) directly under the center of the tank, and (b) at its outer edge using the Newmark influence chart.

The surface contact stress is $q_o = ?_{oil}\,(h)$

$q_0 = (0.95)(9.81\ kN/m^3)(15\ m) =$ **140 kN/m²**

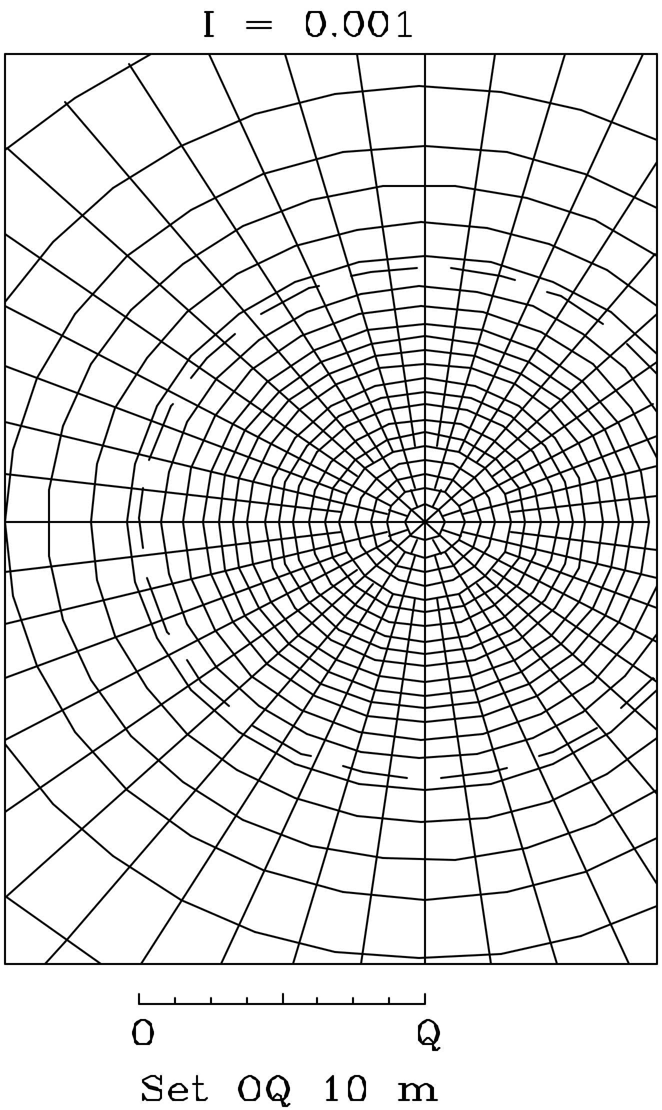
The stress at mid-clay depth and centerline of tank at depth = 10 m; therefore OQ = 10 m.

IV = 0.001 and N = (4)(162) = 648

Therefore,

$s_v' = (q_0)IV(N)$

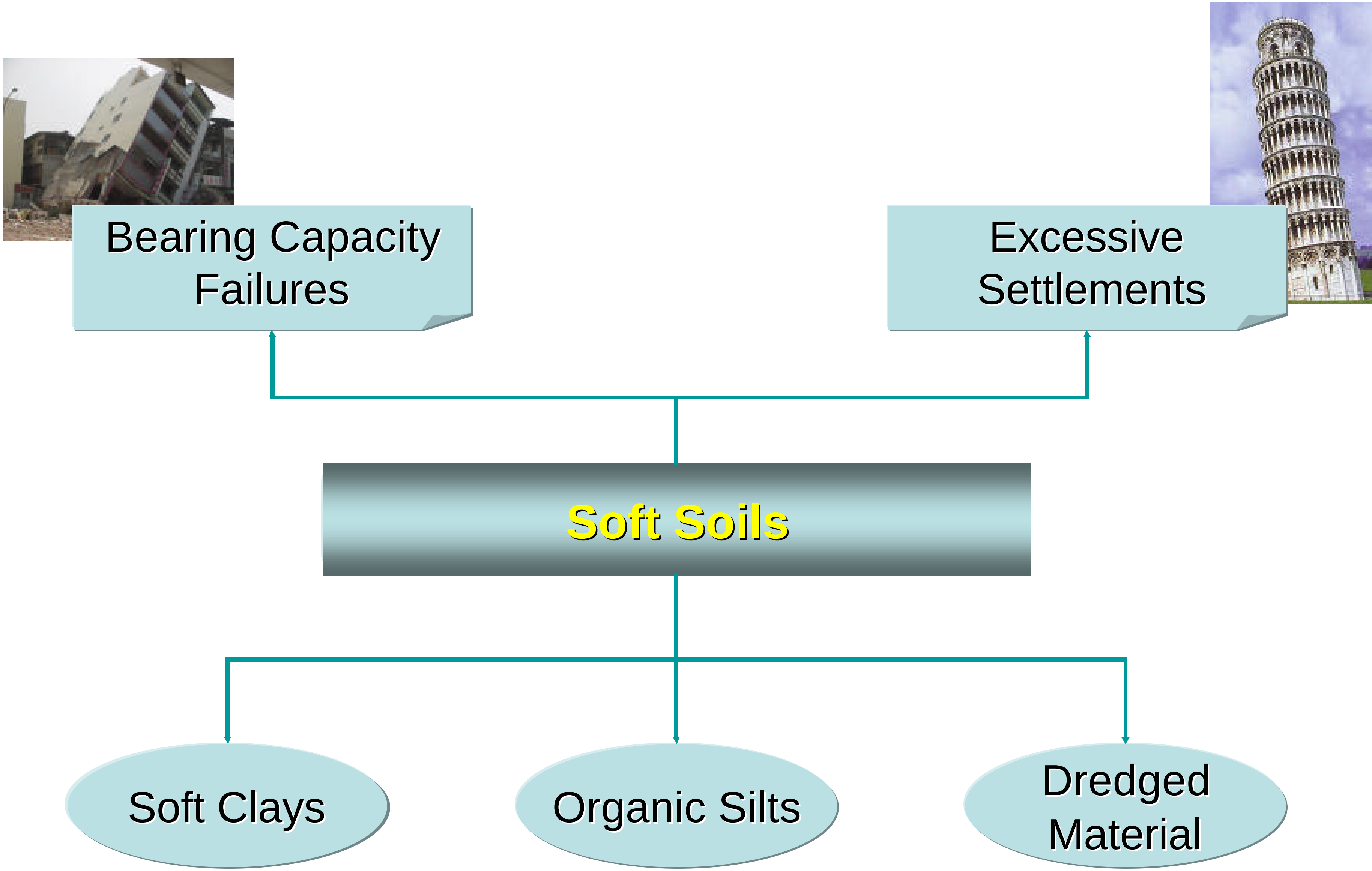
$= (140)(0.001)(648) =$ **91 kN/m²**

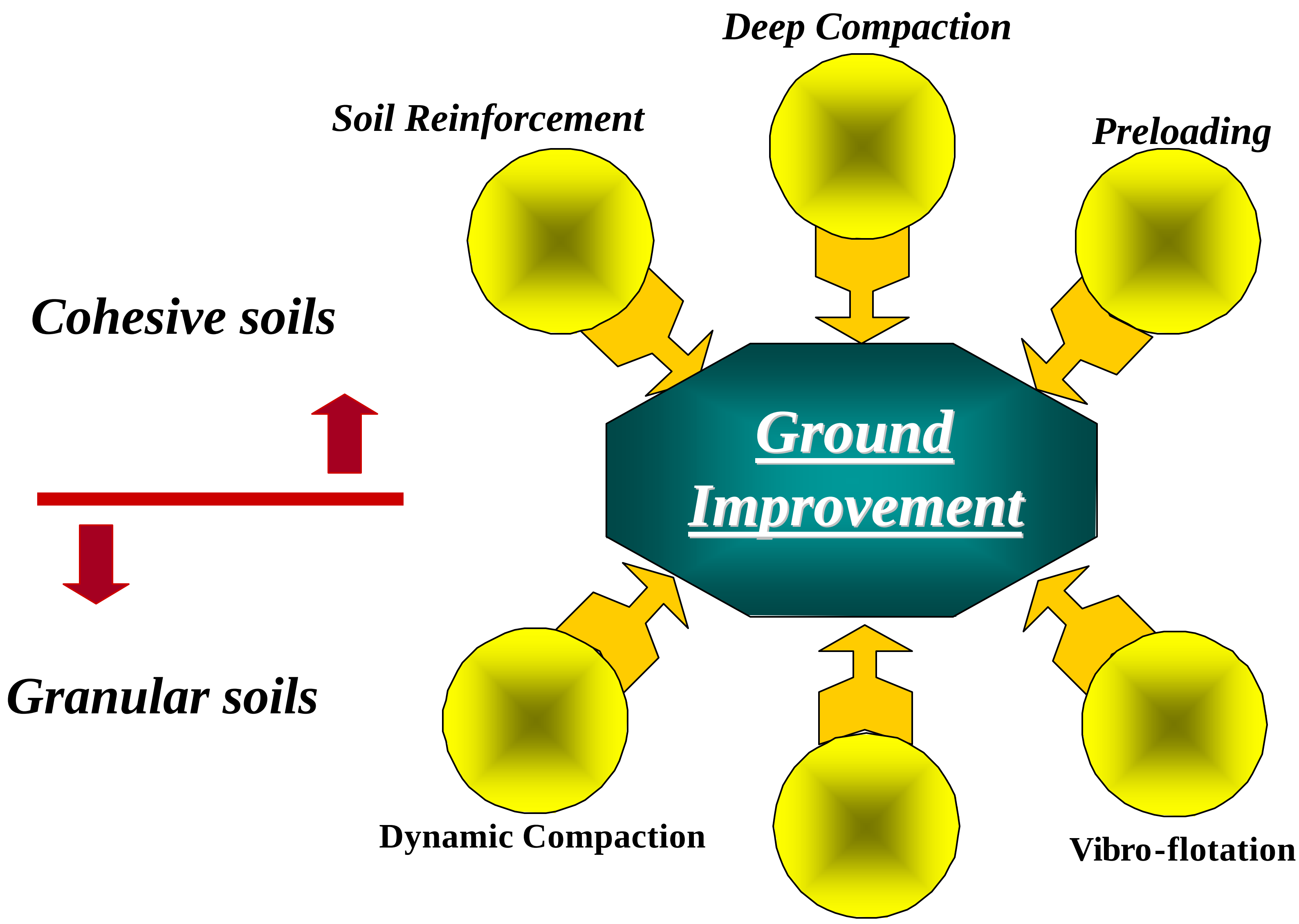


Soil or Ground Improvement

1) *Compaction*
2) *Pre-loading (surcharging)*
3) *Dynamic compaction*
4) *Jet Grouting*
5) *Soil-Cement Pile/Column*
6) *Geotextiles*

Other techniques, in addition to compaction, seek to improve soft soils so that structures placed upon them do not fail through bearing capacity or excessive settlement. The rest of this lecture will show a few other methods of improve soils.




Deep Compaction
Soil Reinforcement
Preloading
Cohesive soils
Ground Improvement
Granular soils
Dynamic Compaction
Vibro-flotation
Resonance Compaction

1. Compaction

The oldest method of improving soil strength is the millennium old technique of ***compaction***. This topic is so important that it is treated exclusively in our next lecture.

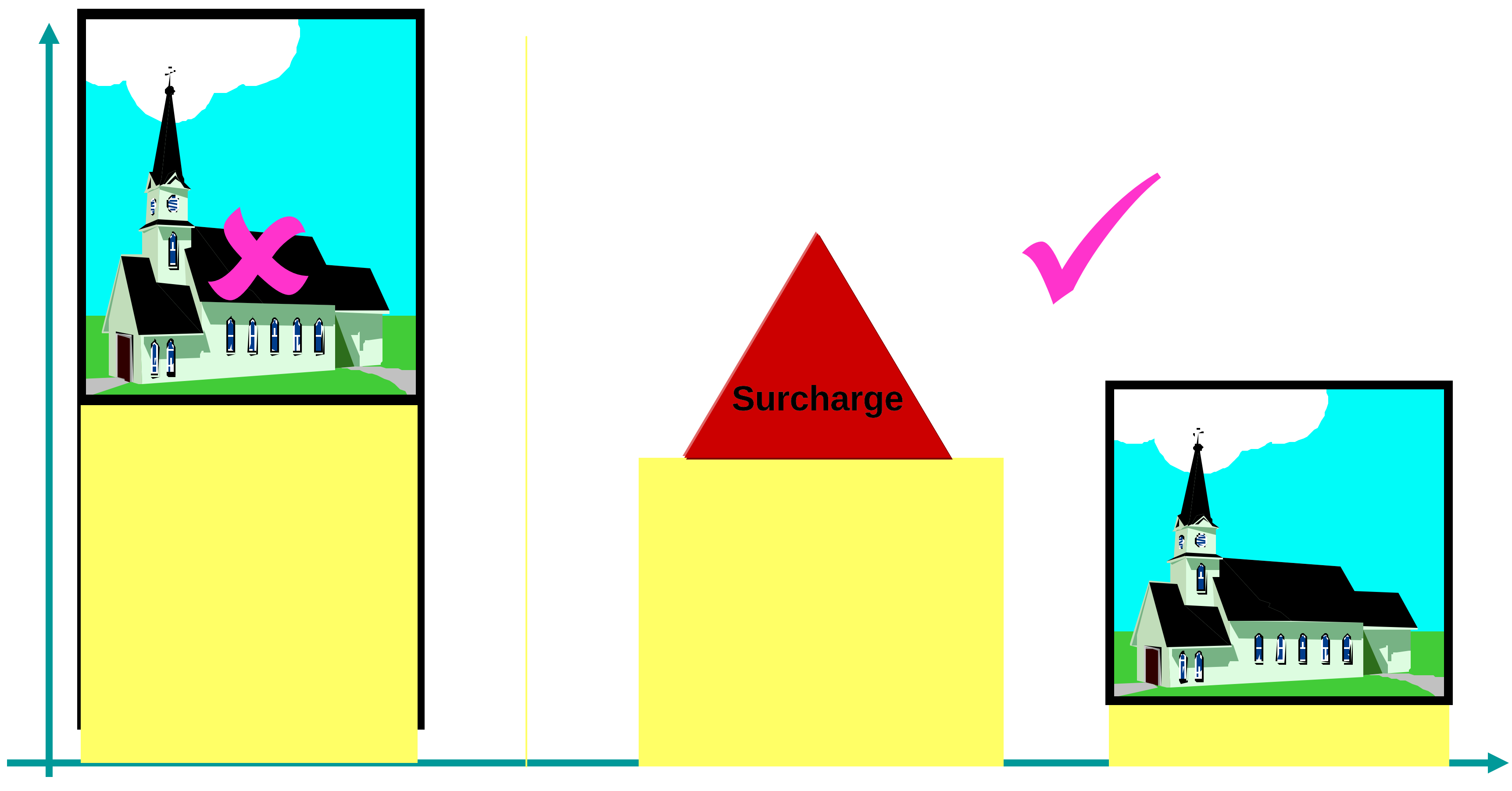
2. Preloading (or surcharging)

Preloading, surcharging or pre-compression is a process where a site will be loaded before construction begins. The additional stress to the soil increases its early settlement, thereby reducing settlements when the actual building is placed on the site. The preloading drives out water from the compressible soil voids, and the water is removed from the site via pumps or open channels. The voids previously occupied by the water now disappears, and the summation of the voids produce the net settlement of the ground surface.

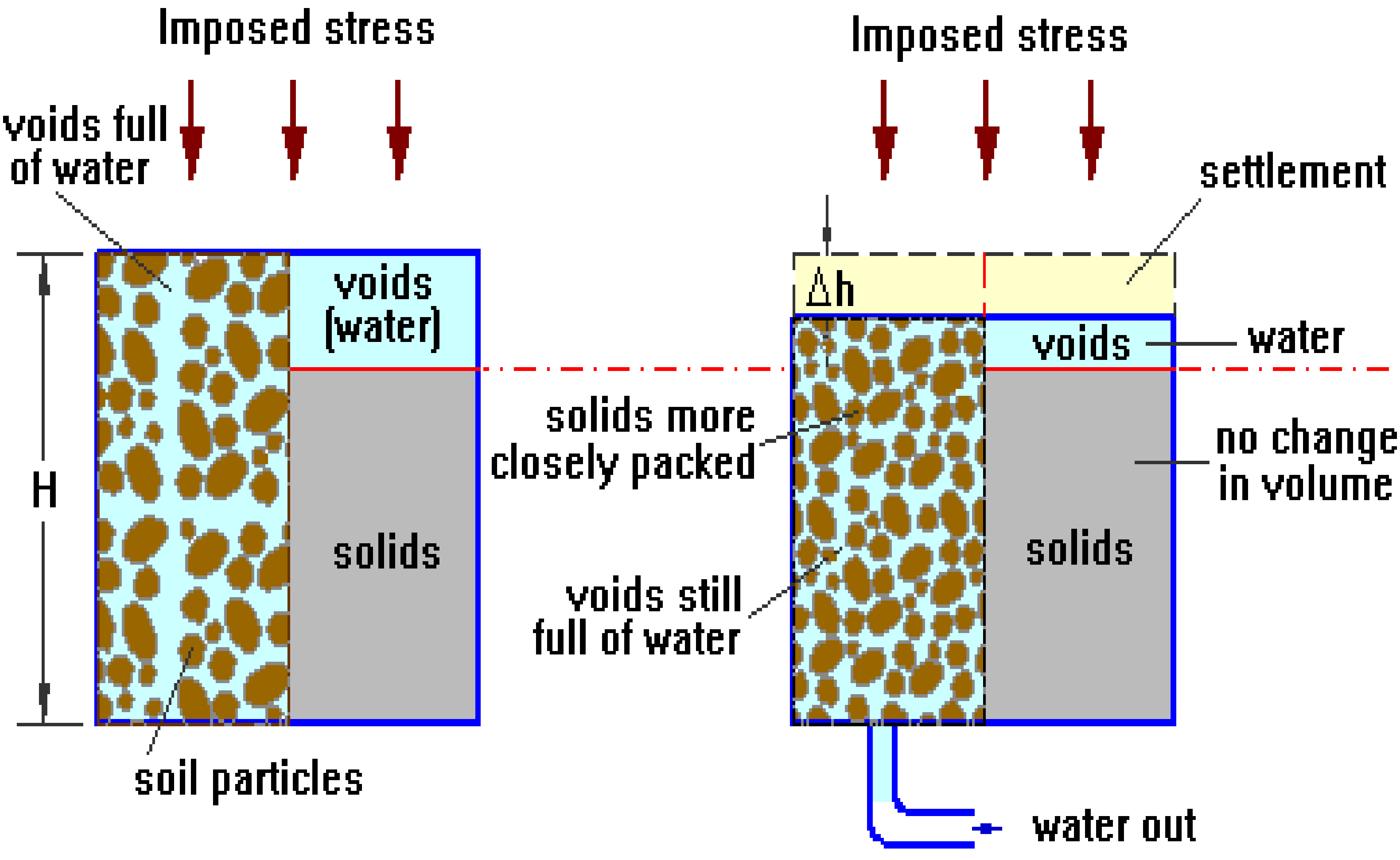
One of the methods used to accelerate the drainage of water is to decrease the path that the water particles have to take to exit the soil. A grid of vertical wicks (drains) is an effective method, as outlined below:

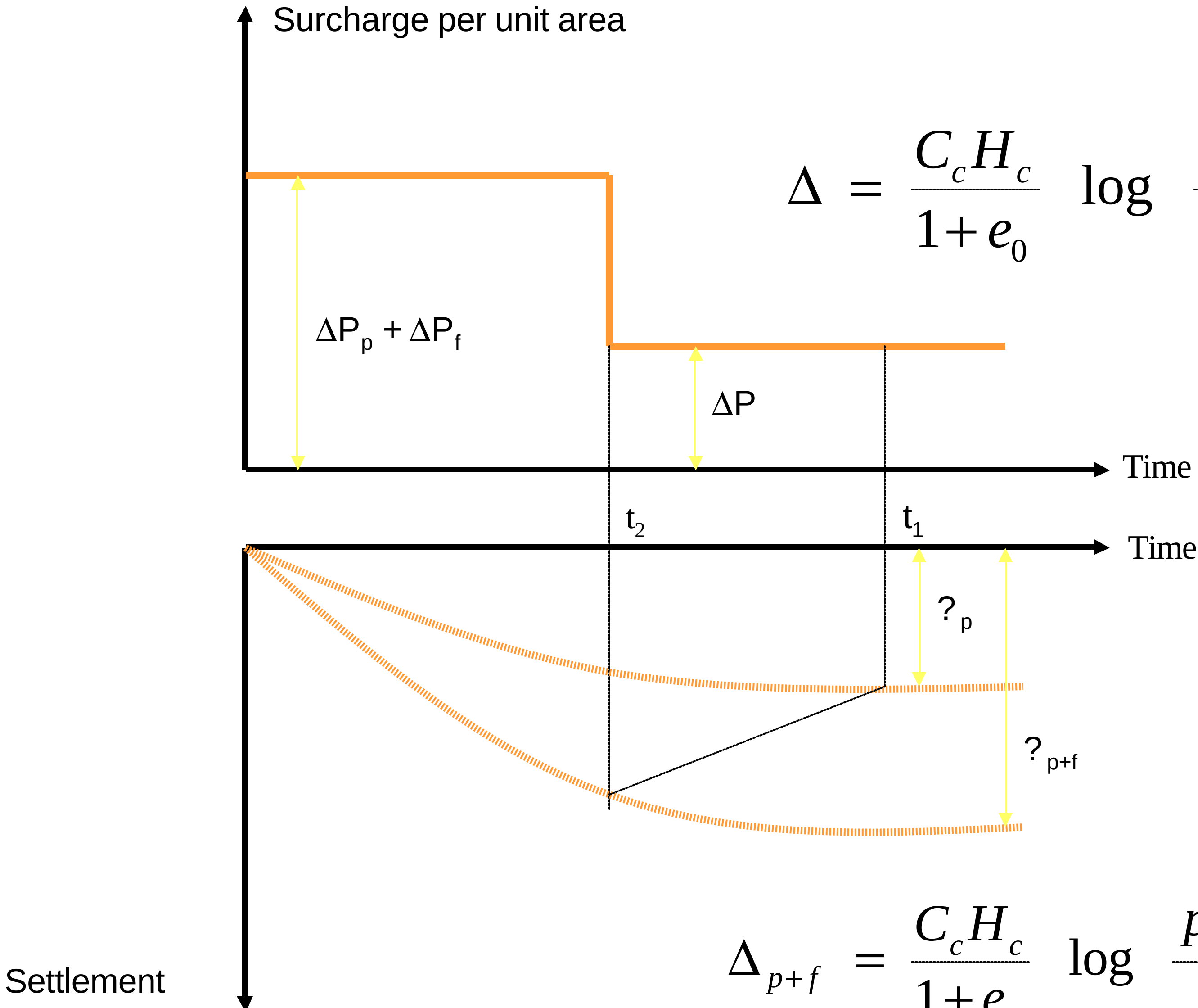
Advantages of prefabricated vertical drains:

- ☑ Increases the rate of consolidation.
- ☑ Reduces the pre-construction time.
- ☑ Increases the stability of slopes.
- ☑ Cheapest and easy to install.


Surcharge

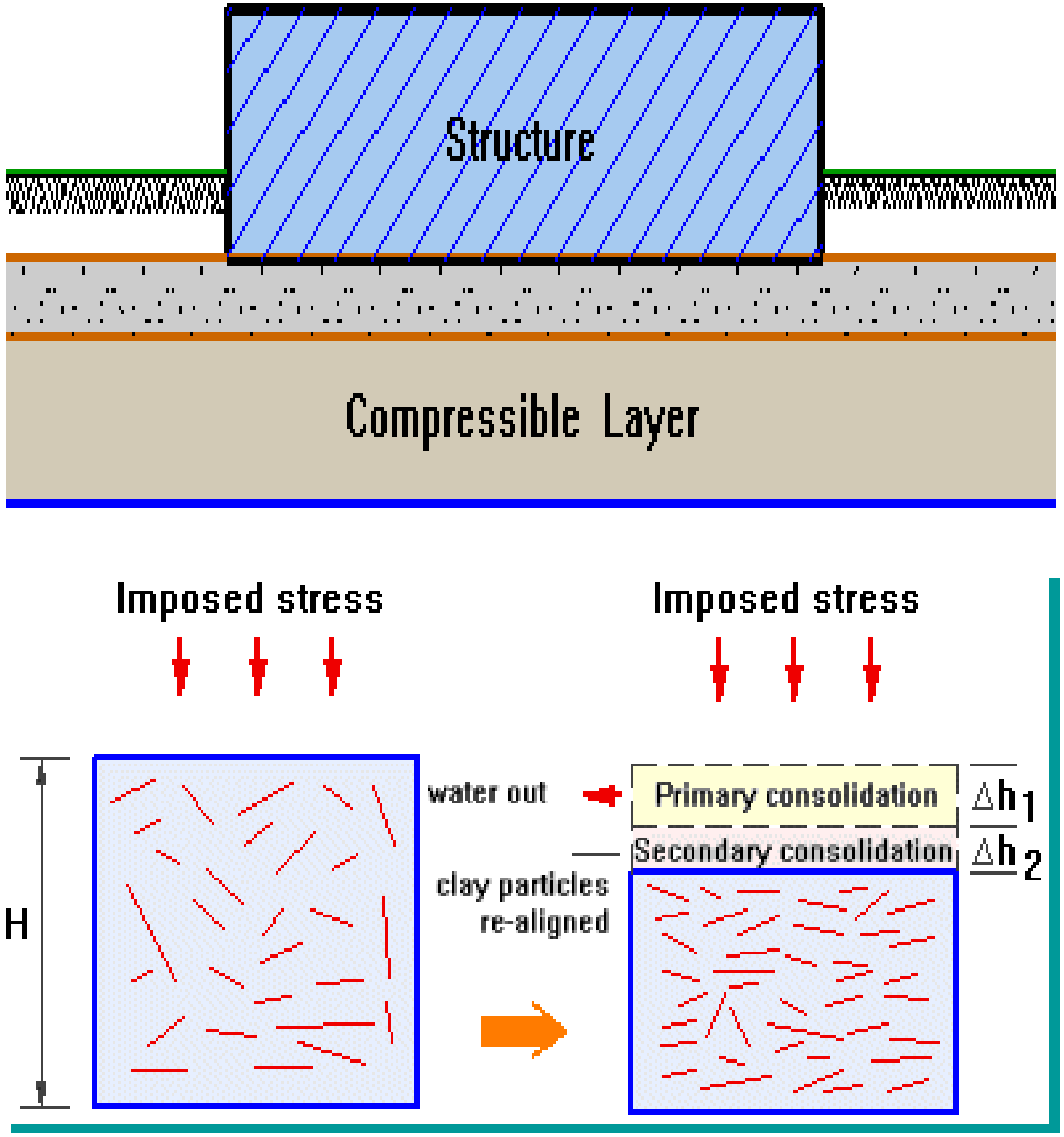
Preloading, surcharging or precompression is the process of placing additional vertical stress on a compressible soil to reduce a portion of the volume of voids and thereby reduce the post construction settlements and increase the soil shear strength.





$$\Delta = \frac{C_c H_c}{1+e_0} \log \frac{p_0 + \Delta p_p}{p_o}$$

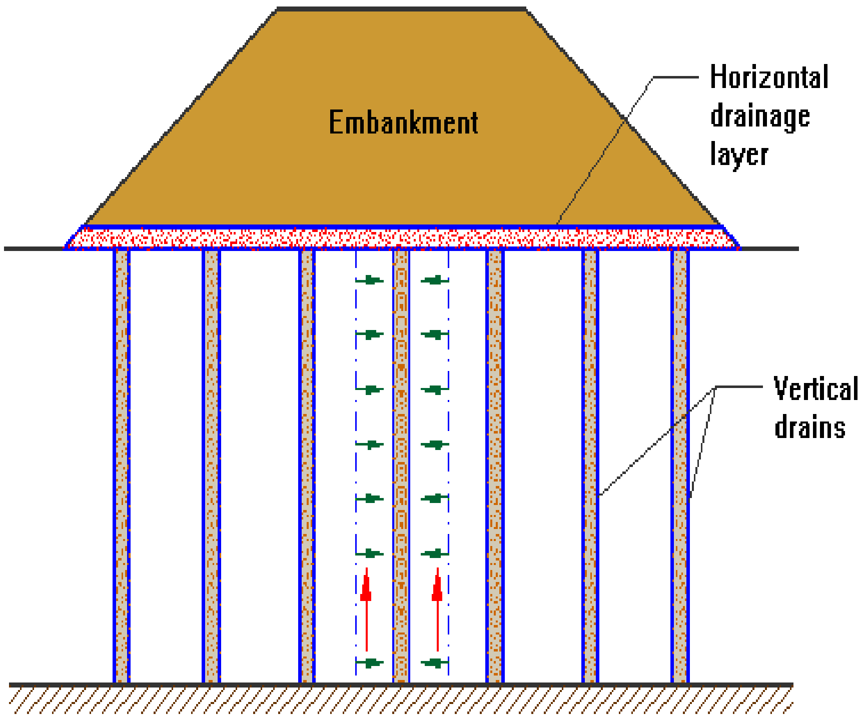
$$\Delta_{p+f} = \frac{C_c H_c}{1+e_0} \log \frac{p_0 + (\Delta p_p + \Delta p_f)}{p_o}$$

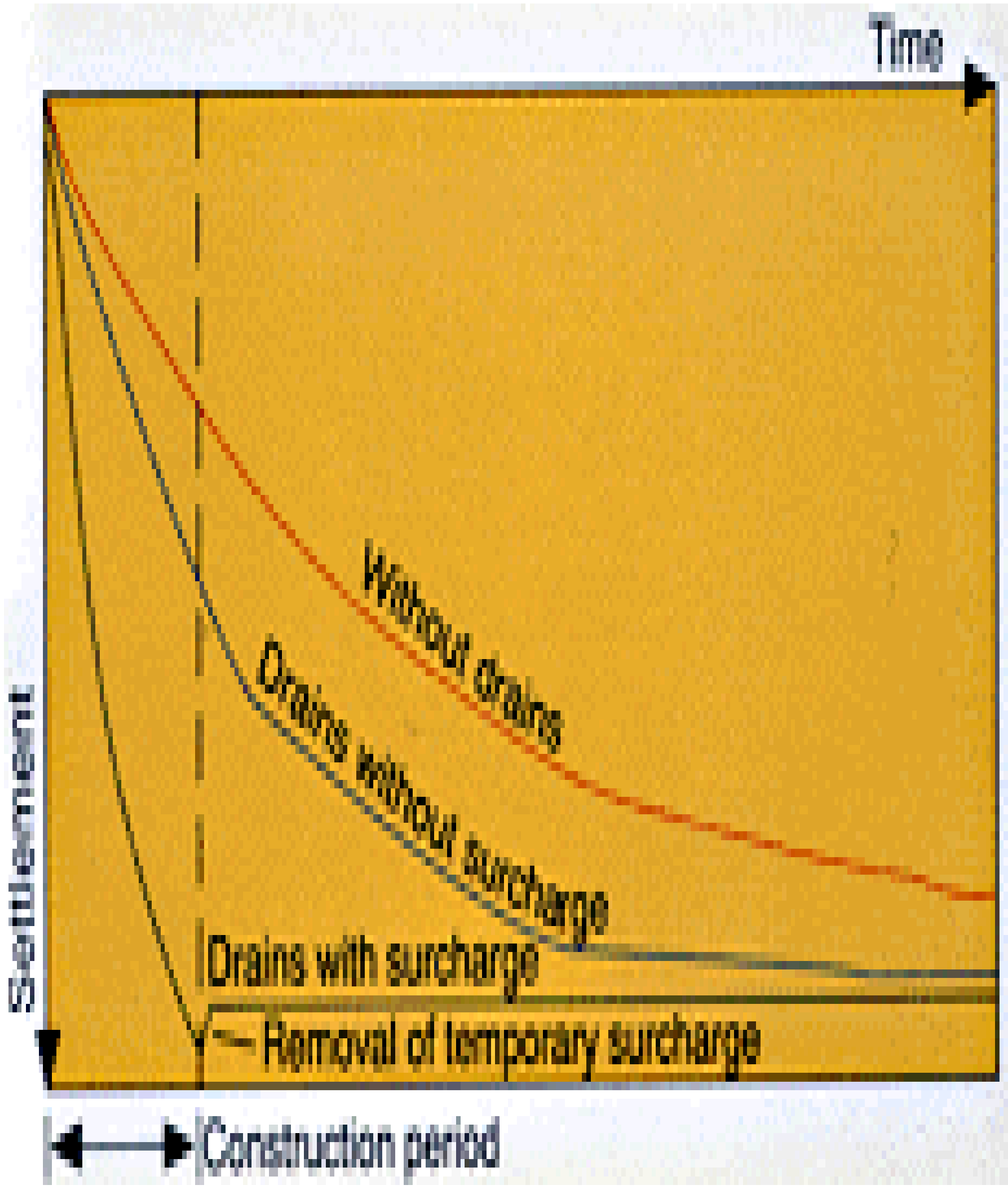


Advantages:

- Reduce Post-Construction Settlement
- Reduce Secondary Compression
- Densification
- Improve Bearing Capacity

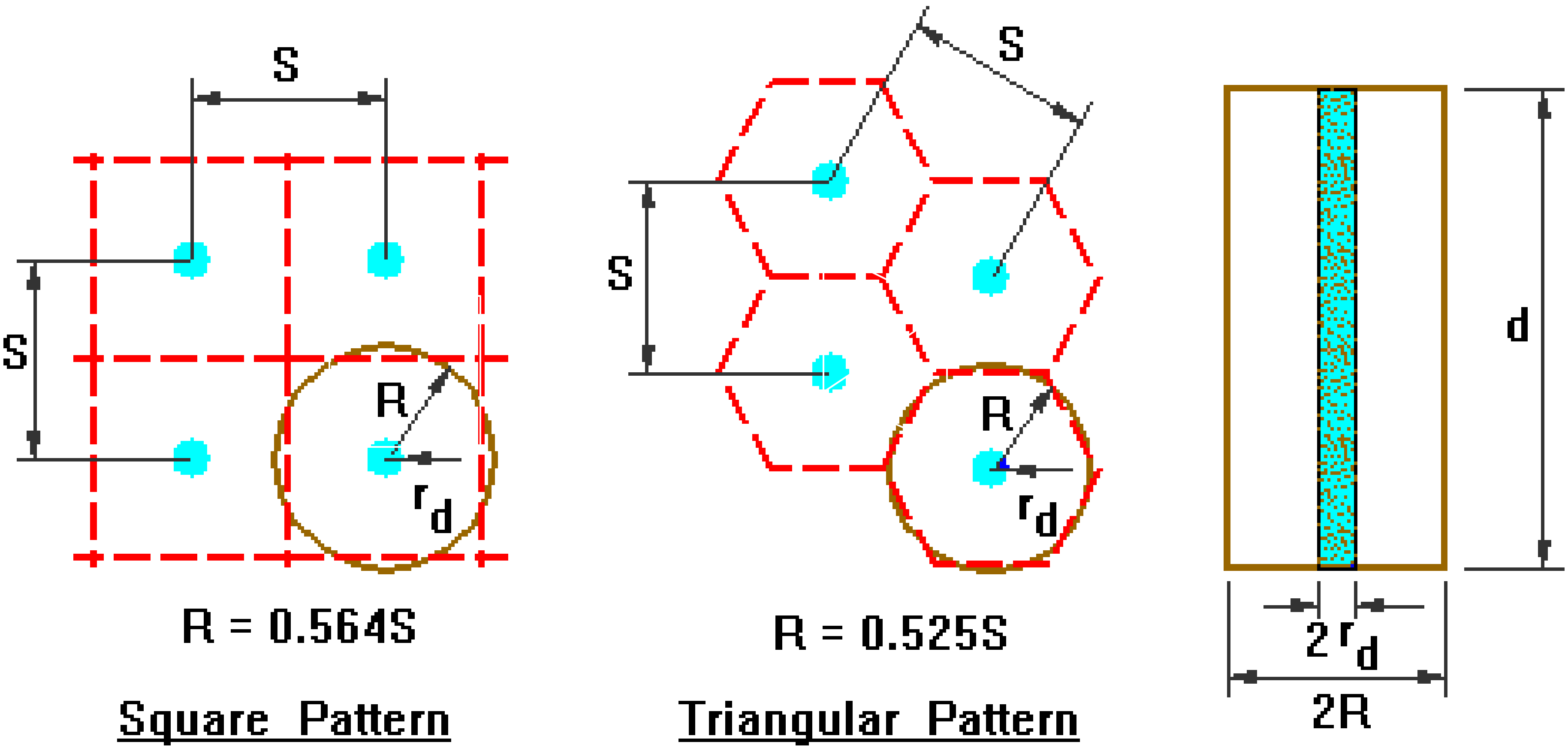
Vertical Drains





$$T_v = \frac{c_v t}{h_{drain}^{2}}$$

Typical arrangements of the vertical drains

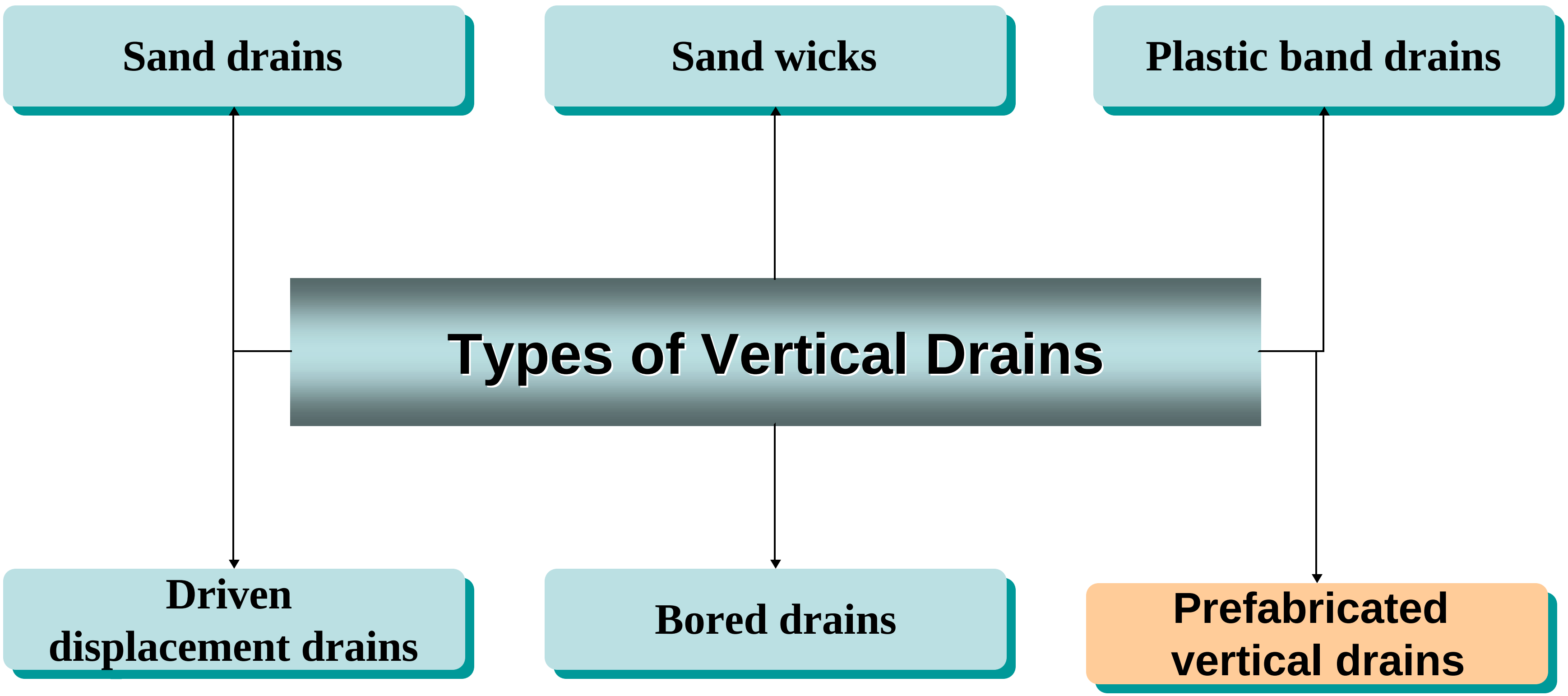


R = 0.564S

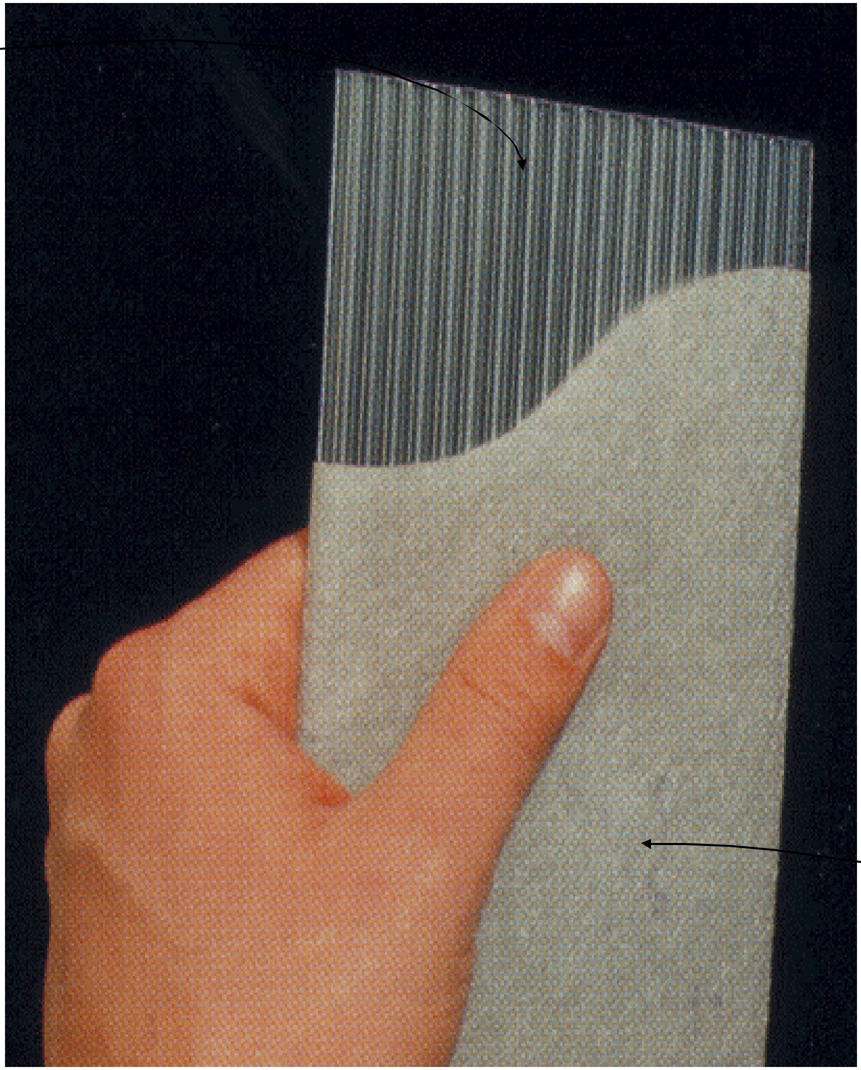
Square Pattern

R = 0.525S

Triangular Pattern

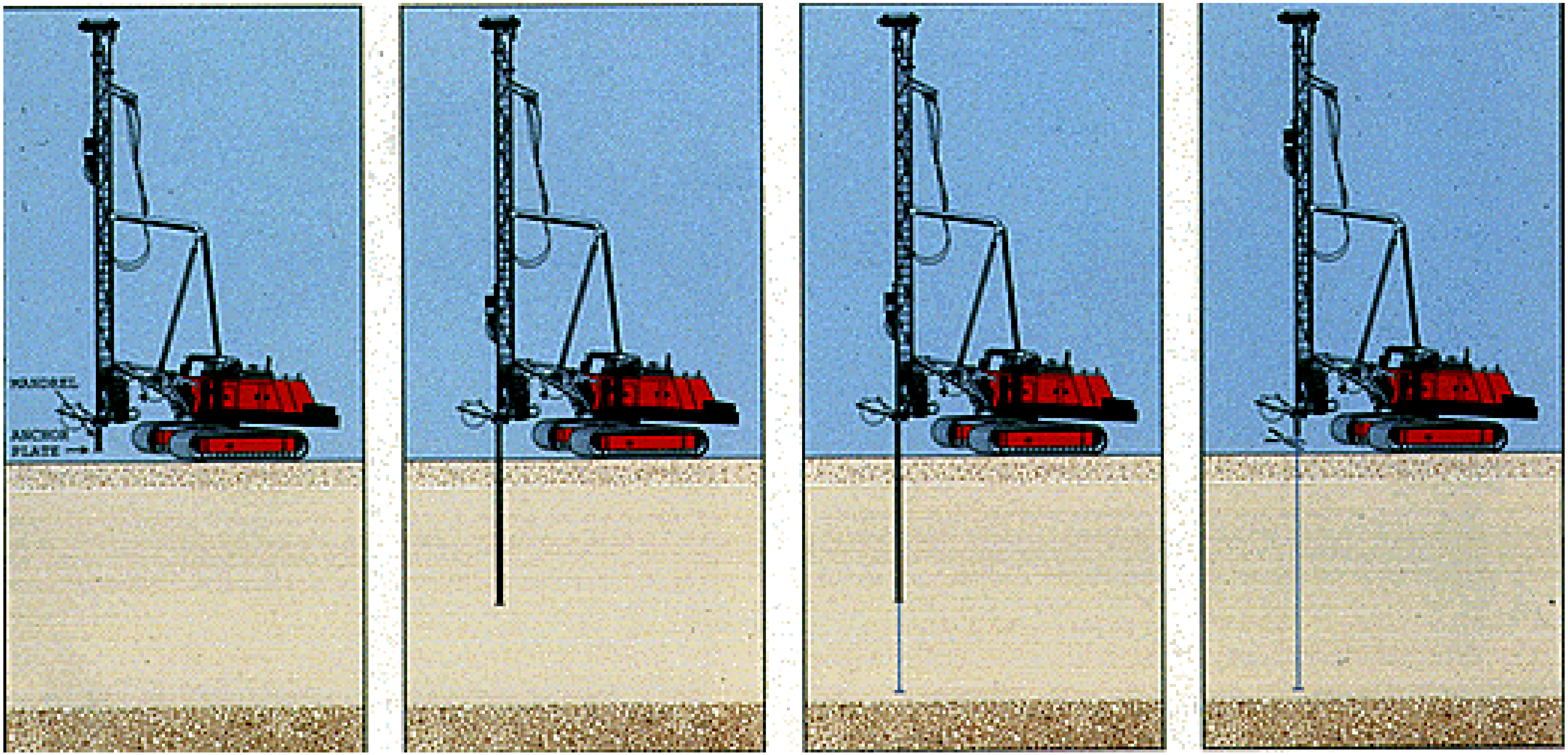


Sand drains
Sand wicks
Plastic band drains
Types of Vertical Drains
Driven displacement drains
Bored drains
Prefabricated vertical drains

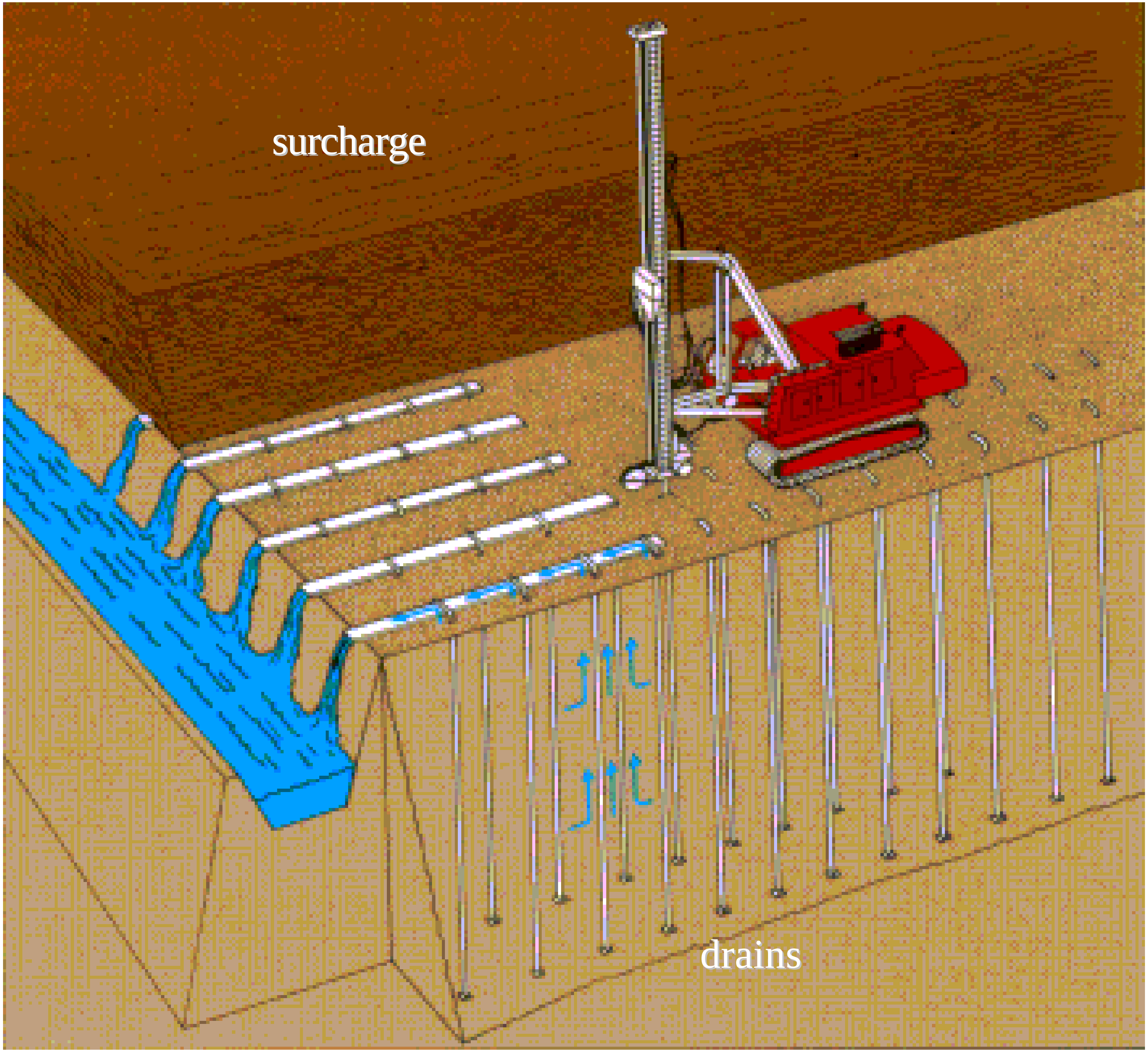


Prefabricated vertical drains.

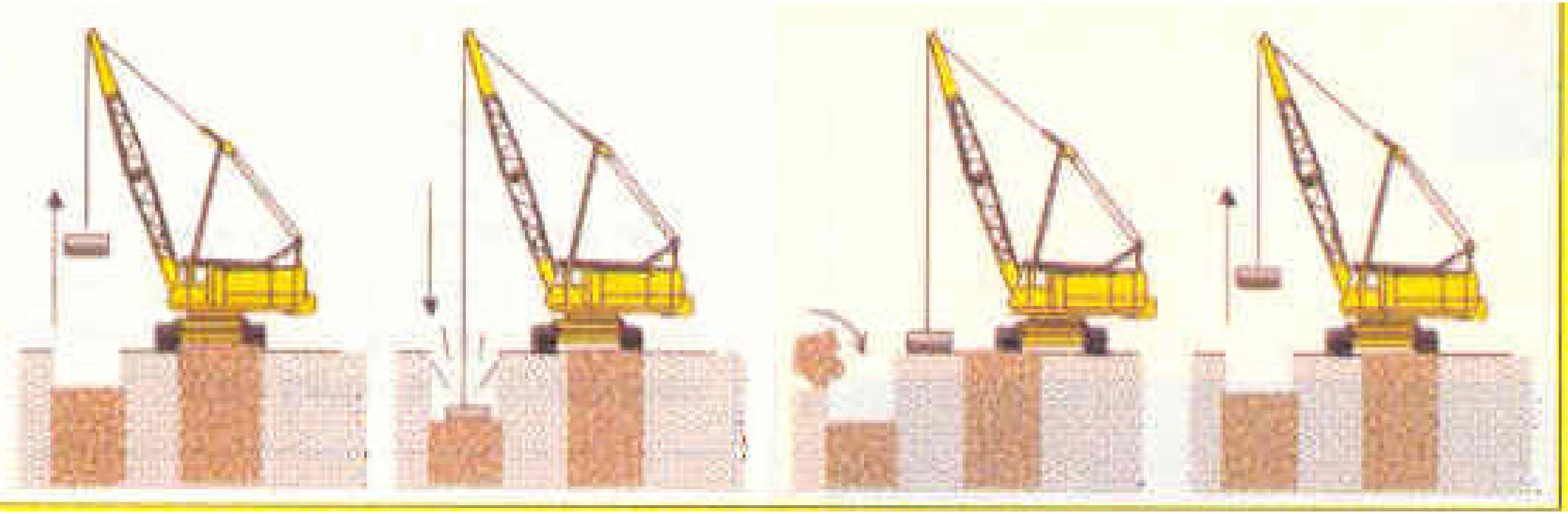
Installation of prefabricated drain wicks



Setup the equipment | Drive the mandrel | Extract the mandrel | Cut off the drain wick

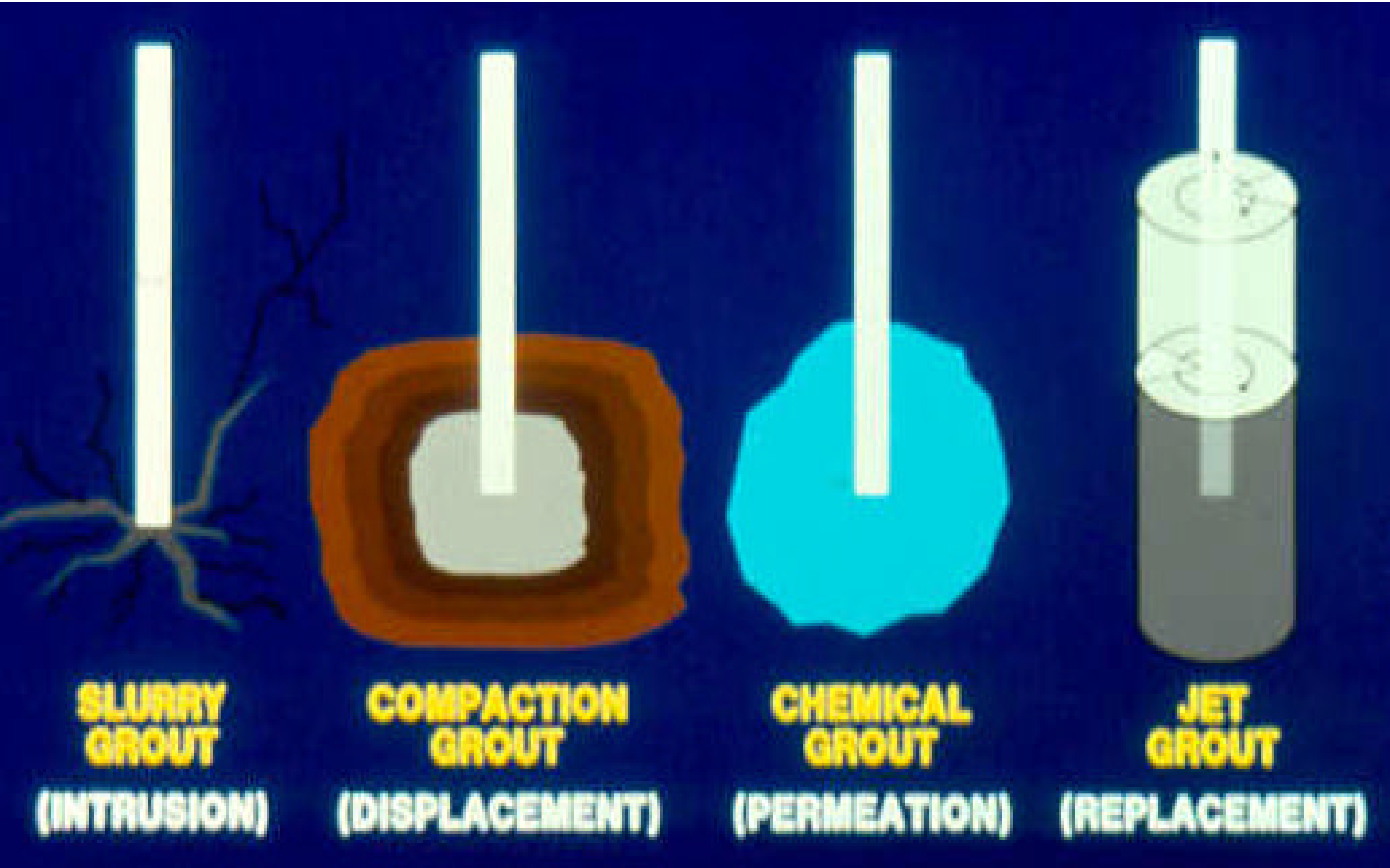

surcharge
drains

3. Dynamic Compaction



4. Jet Grouting

Common Techniques for Ground Modification:



Slurry	intrusion under pressure of flowable particulate grouts into open cracks and voids and expanded fractures
Compaction	injection of very stiff, “zero-slump” mortar grout to displace and compact soils in place
Chemical	permeations of sands with fluid grouts to produce sandstone-like soil masses to carry loads
Jet	cutting geometric shapes in soils with high-pressure liquid jets and filling the spaces with grout

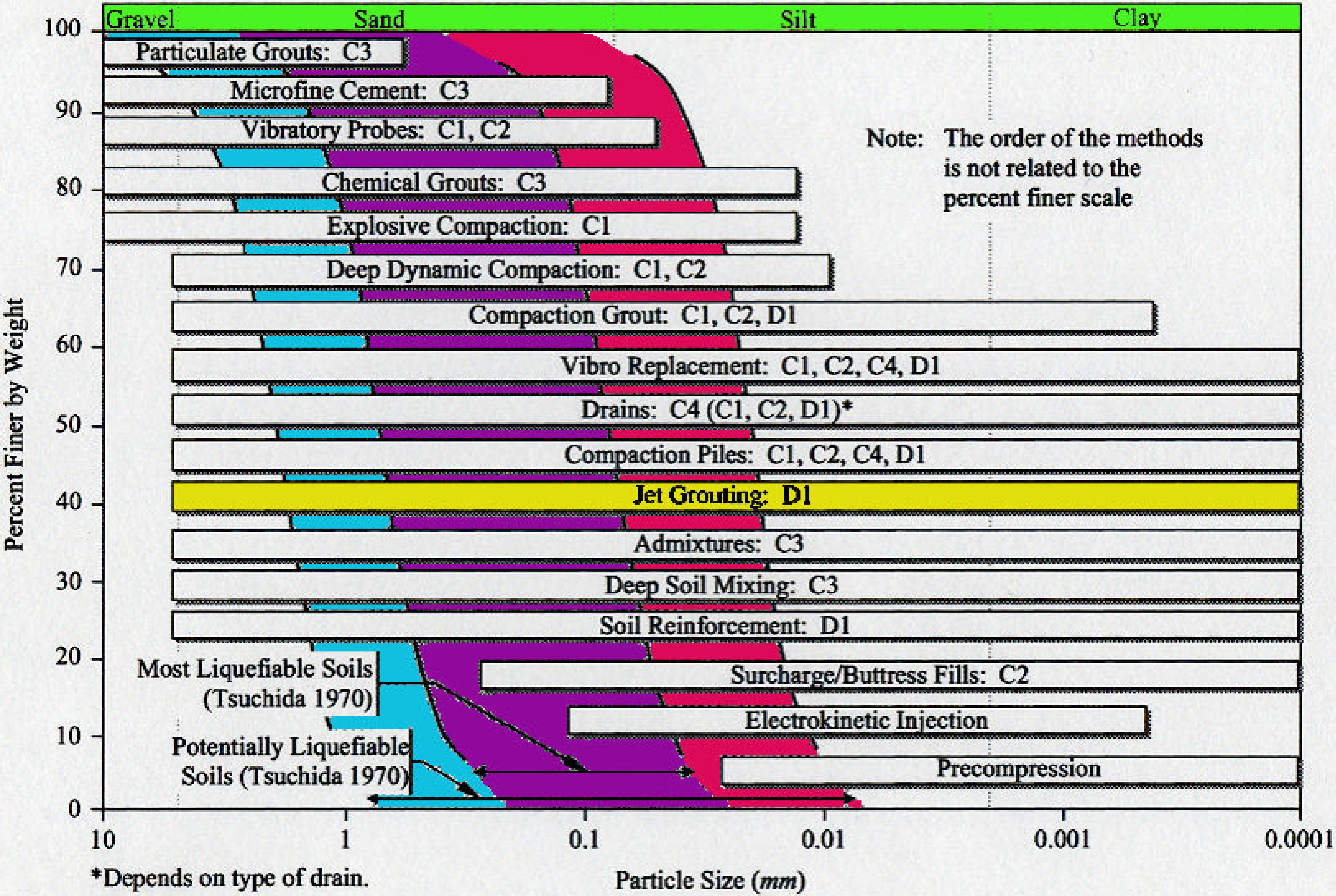


Figure 7-2. Applicable grain-size ranges for liquefiable soil improvement methods. (Adapted from Mitchell and Gallagher 1998).

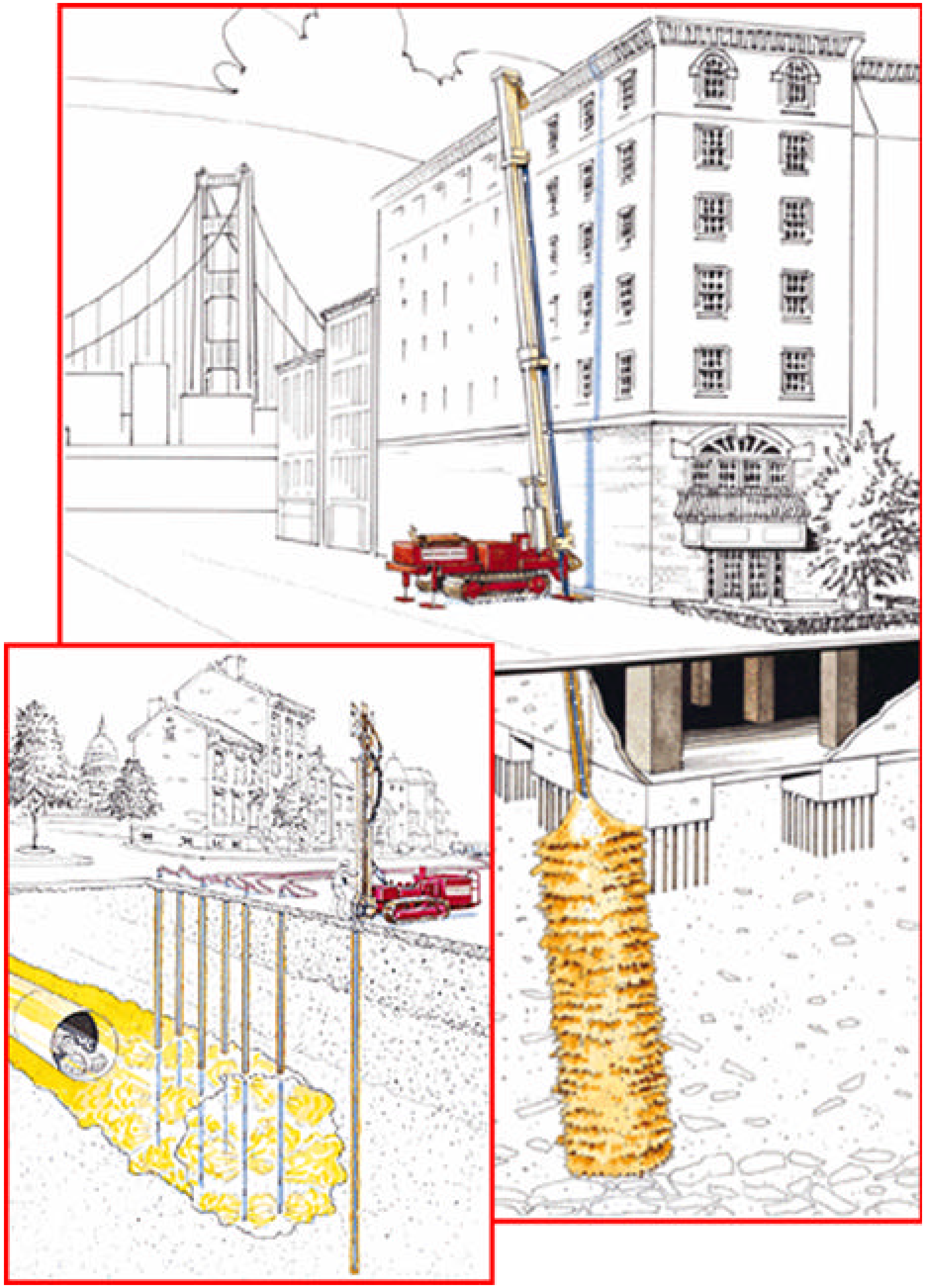
Approaches

Table 7-1. Approaches to increasing *Capacity* and decreasing *Demand*.

Capacity ↑	*Demand* ↓
C1) Increase Soil Density	D1) Soil Reinforcement/Seismic Shear Stress Redistribution
C2) Increase Effective Confining Pressure	D2) Shift Fundamental Period of Soil Profile
C3) Prevent Collapse of Soil Skeleton i) Bond Soil Particles Together ii) Fill Voids with Grout	
C4) Provide Mechanism for Rapid Dissipation of Excess Pore Pressures	

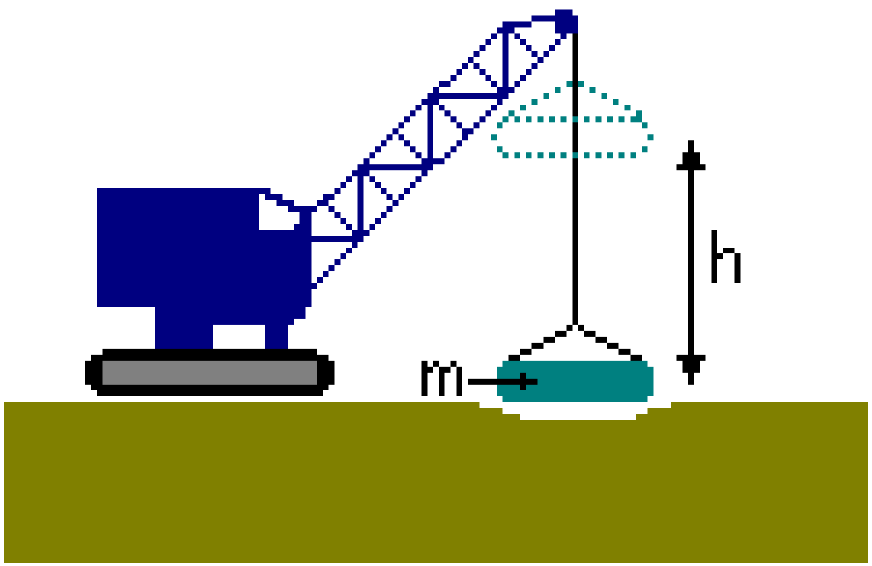
Jet grouting. This system differs substantially from the other grouting techniques in that it breaks-up the soil structure completely and mixes the soil particles *in-situ* with a binder to create a homogenous mass which in time solidifies. This solidified ground is known as Soilcrete.

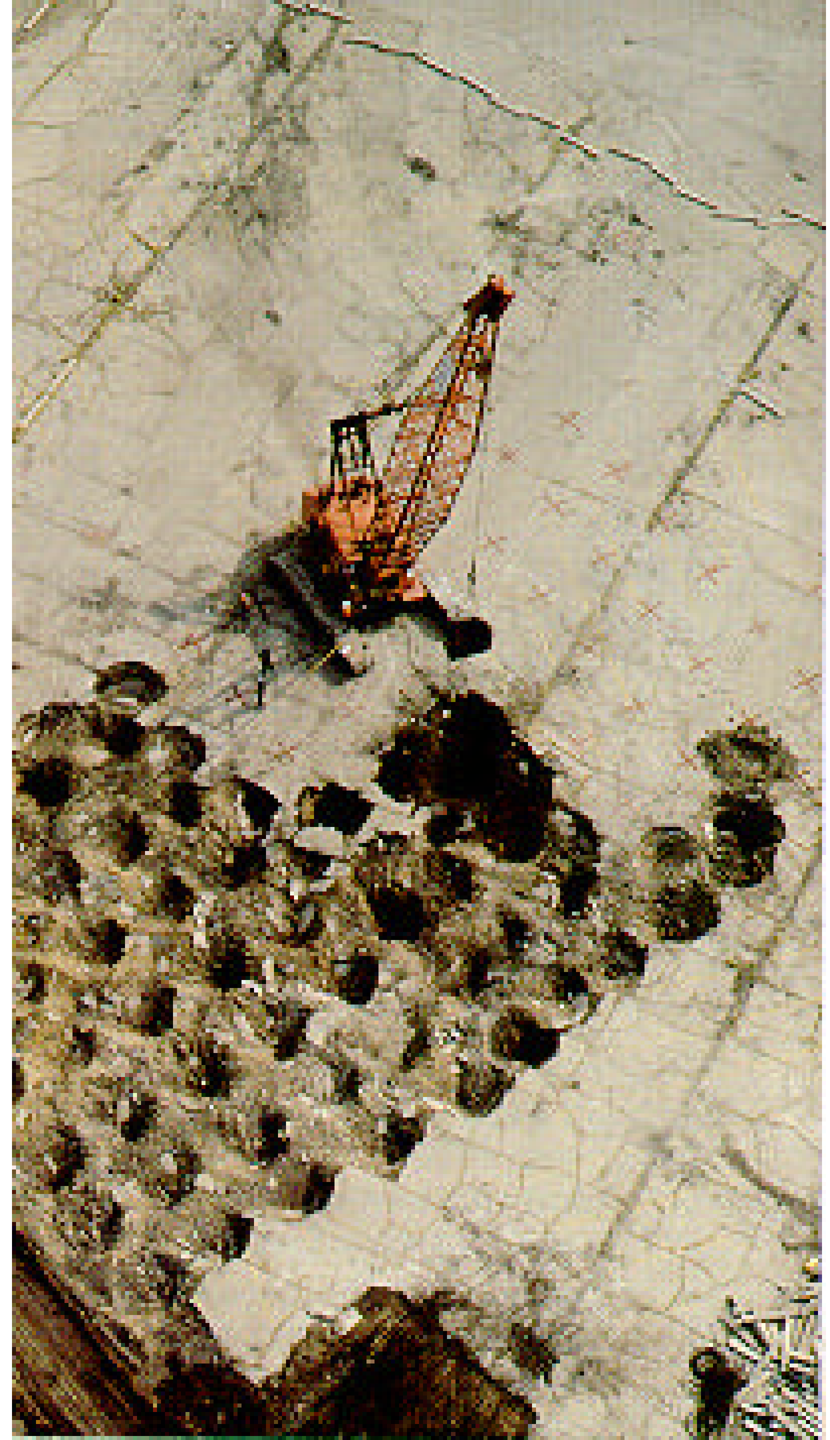
The technique, originating in Italy, can be used regardless of soil type, permeability, grain size distribution, etc. In theory, therefore, it is possible by jet grouting to treat most soils, from soft clays and silts to sands and gravels.



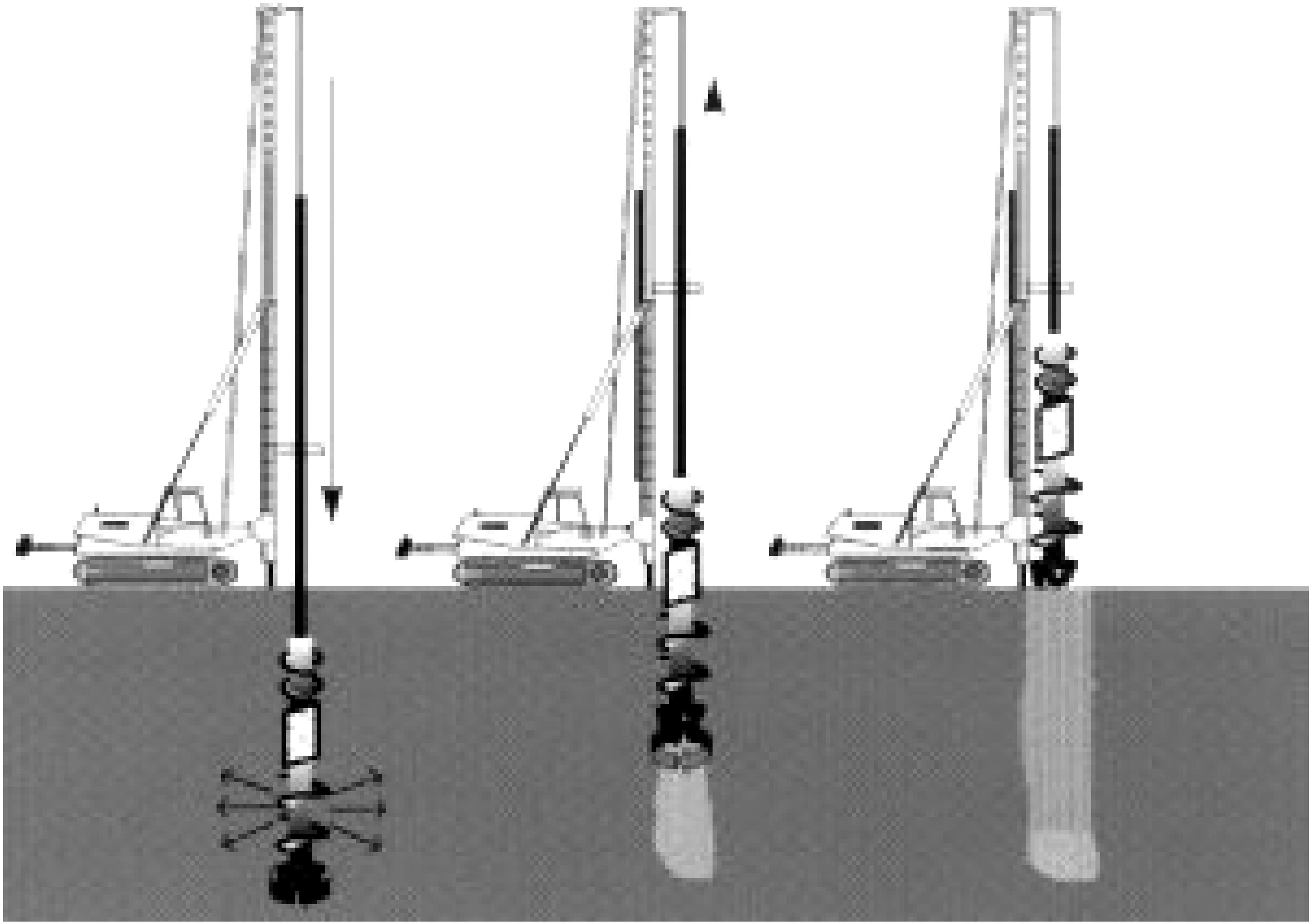
Deep Dynamic Compaction (DDC):

- **Heavy weights (5-40 ton) are dropped from heights of 6 to 30 m.**
- **The impact of the falling weight compacts the soil to significant depths.**



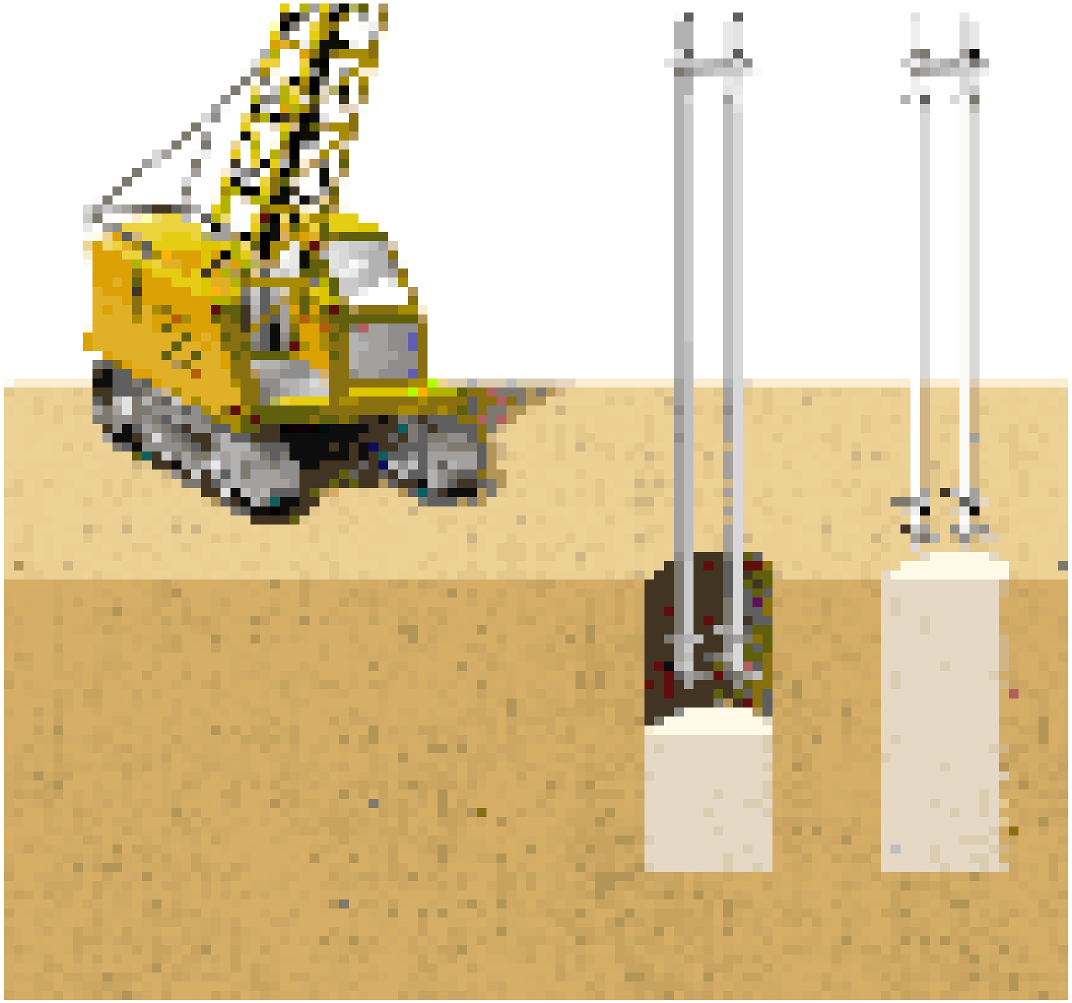


5. Soil-Cement Pile Columns



Applications for Soil-Mixing:

1. Ground water barriers;
2. Retaining barriers;
3. Foundation walls.



Deep soil-mixing (DSM) uses a special soil drill auger bits that advance into the soil, cutting and grinding the soil. Simultaneously the bits inject a cement slurry additive to improve the soil shear strength.

The diameters of the auger bits vary from 0.6 to 1 m can produce wall lengths varying from 1.5 m to 10 m, with maximum depths to 35 m.

Each rig is equipped with three overlapping mixing paddles (or augers). The mixing paddles are of limited length so that they do not carry soil up to the ground surface, but rather continually mix the soil to depth. Grout, in this case a mixture of water and cementing agents, will be pumped through ports in the mixing paddles, and subsequently mixed with the soil.

The mixing paddles periodically require maintenance due to the wear and abrasion on parts.

An on-site batch plant provides automated mixing of the grout materials, which is then pumped to the deep mixing rigs.

A coring rig is used to obtain solid cores of the hardened soil-cement.

The piles within the soil-cement grid will support a wharf that will be constructed.

The grid of soil-cement walls extends down through soft soils into harder competent soils, and acts to increase the stability of the channel slope.

A close-up view of the hardened soil-cement at one location shows pockets of clay within the soil-cement matrix. These clay pockets occur where there has been insufficient mixing.

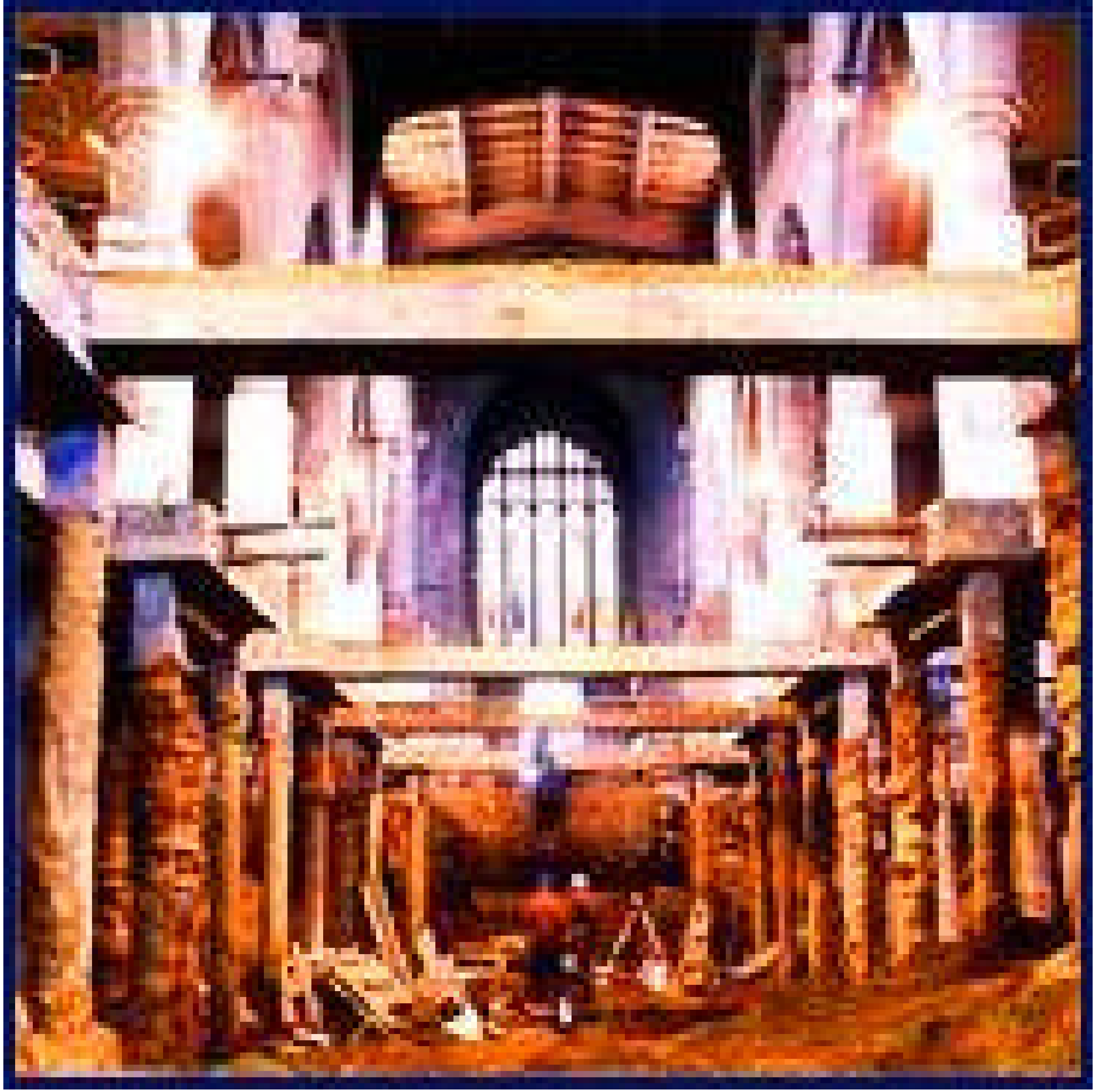
Each vertical run of the mixing rig produces three overlapping columns These columns must overlap with adjacent sets of columns to produce a continuous wall.

This is a side view of soil-cement columns exposed by an excavation.

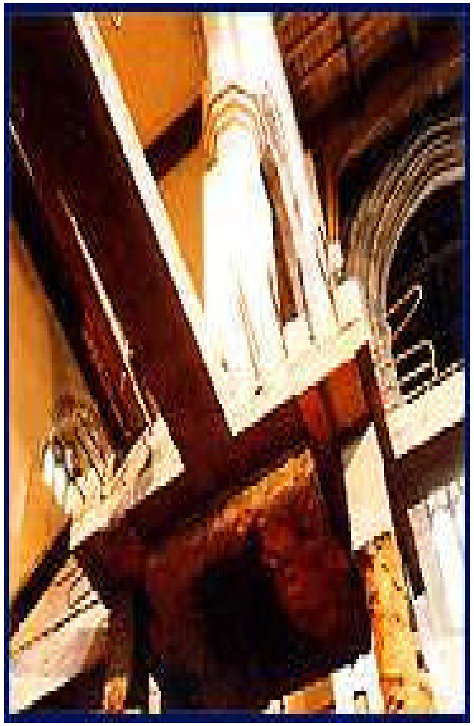
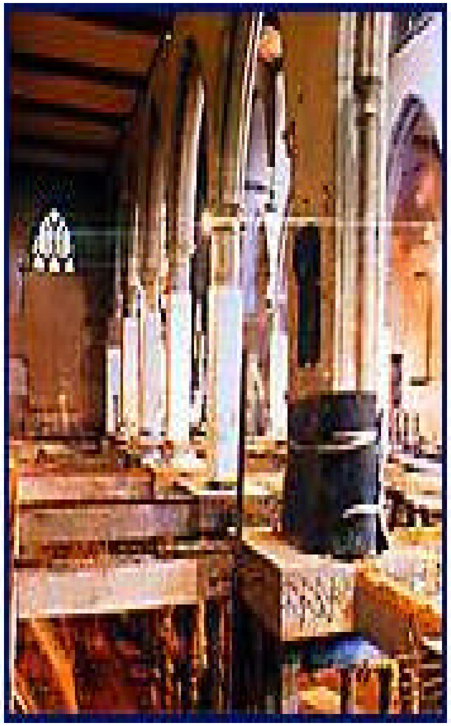
Notice that the columns are horizontally stratified, reflecting the variation in soil type with depth.

The diagonal cracks that can be seen in the columns are primarily due to impacts by the backhoe that made the excavation.

Following excavation and construction of new columns below the old, the temporary support piles were removed.



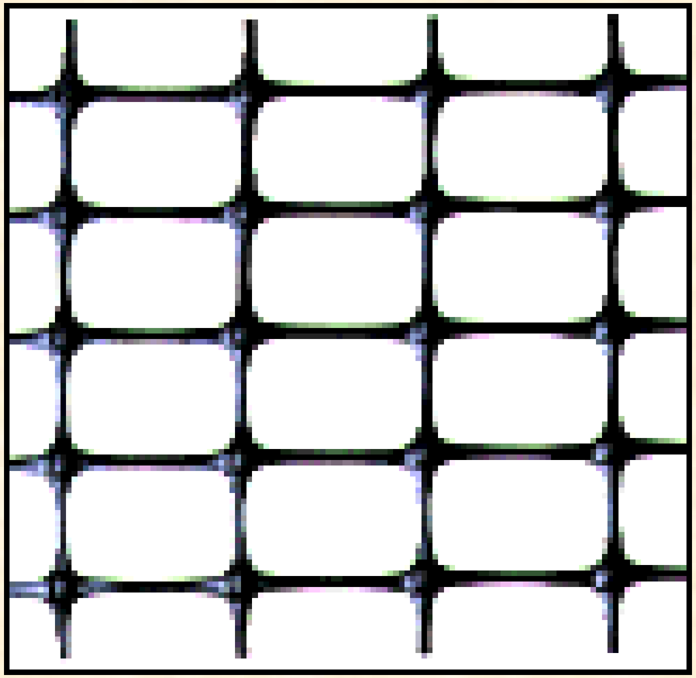
The ancient church of St. Nicholas, Sevenoaks, Kent, used mini piles to underpin the 800 year old church to support the building whilst providing a new undercroft.

The structure dates from the 12th century, and archaeological investigations below the floor revealed the existence of lead coffins and brick walled tombs dating back to the 13th century. These excavations also exposed the original friable ironstone foundations of the church.

Great care was needed in constructing the mini piles to minimize vibrations. With the piles in place, a concrete collar was formed around each column, encasing the existing foundations and column base.

A grid of ground beams was then cast between the collars to form a new temporary structural support system.



6. Geotextiles

This photo is shown in the Cooper-Hewitt National Design Museum, named “Extreme Textiles: Designing for High Performance”. 2 East 91st Street, New York).

Geotextiles are being characterized as the materials of the future: stronger, lighter and safer.

They can be used in many forms, such as weaving, knitting and braiding of these fibers that are stronger than steel but retain the flexibility of textiles.

Visit: www.ifai.com or, www.fabricarchitecture.info.

A 35 m high reinforced slope for a water retention pond in Taichung City,Taiwan.

Applications of geotextiles are applied to civil engineering problems:

- *Road structures (embankments, abutments, ramps, etc.)*
- *Erosion and sediment control*
- *Subsurface drainage*
- *Reinforced soil structures*
- *Waste contaminant systems*

The advantages of using geotextiles are,

- *Geotextiles occupy significantly less volume than comparable soil and aggregate layers.*
- *Geotextiles are manufactured under controlled factory conditions which minimizes material variation, while soil and aggregate are proportioned at batch plants.*

3. *Geotextiles require limited field connections, while soil and aggregate layers are actually constructed in place and therefore are subject to variations caused by weather, handling and placement.*
4. *Geotextiles are less expensive to purchase, transport and install than soils and aggregates.*
5. *Geotextiles have been engineered for optimum performance, including greater strength, drainage efficiency and clogging resistance than soils and aggregates.*

The material of Geotextiles: Polyester versus Polypropylene.

Table 1.

Geotextile Polymer Characteristics

Property	**Polyester**	**Polypropylene**
Specific Gravity	1.36 (sinks in water)	0.9 (floats in water)
Melting Point	247-254 C	160-188 C
Chemical Resistance	Very Good	Excellent
U.V. Resistance	Very Good	Good(stabilized)

Three basic categories of Geotextiles

Woven: *Used primarily for sediment control and road stabilization.*

Woven geotextiles are constructed from impermeable material made into narrow strips and then are woven together.

Non-Woven: *Used for slope stabilization and erosion control.*

A non-woven geotextile is a polymer that is melted and forced through extrusion dies having numerous small holes, thereby creating a "grid".

Needle-punched: *Used ideally for subsurface drainage.*

This geotextiles is bonded by mechanically entangling the fibers through the use of barbed needles, thereby causing a large number of voids.

Tests to confirm design criteria:

Survivability:

Mass per unit area
Thickness
Grab tensile strength and elongation
Puncture strength
Tearing strength

Performance Tests:

Tensile strength
Seam strength
Friction
Asphalt retention
Permittivity

Durability Tests:

Chemical resistance
Ultraviolet stability
Clogging resistance
Bio-clogging resistance
Creep
Abrasion resistance

Problem #1:

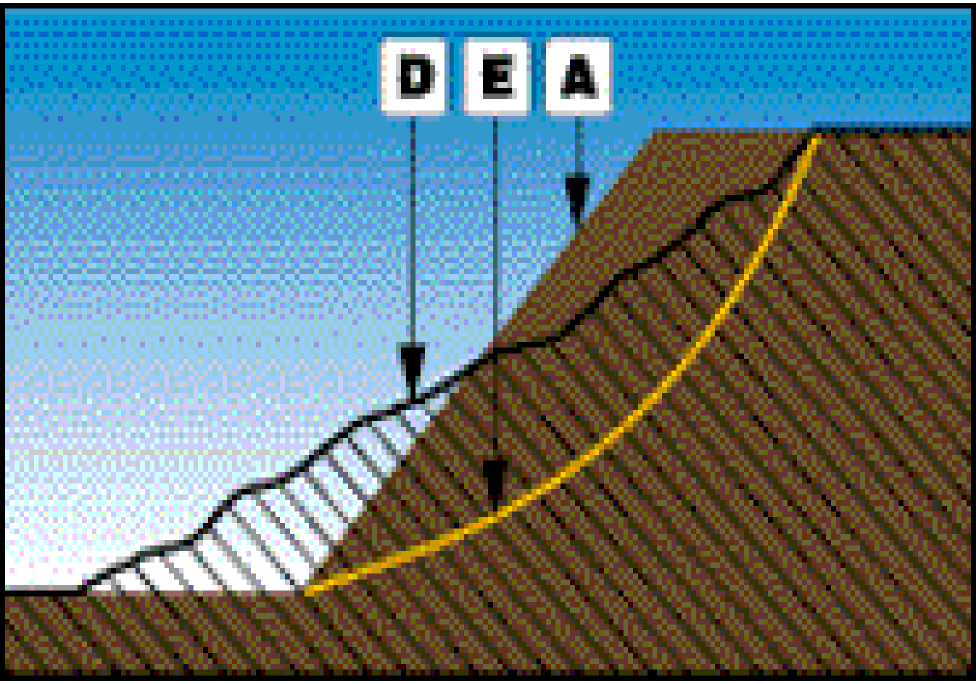
In the central part of Taiwan, a housing project required extending construction on a mountainous area.

The property owner planned to maximize the usable land space to fit in luxury villas and townhouses.

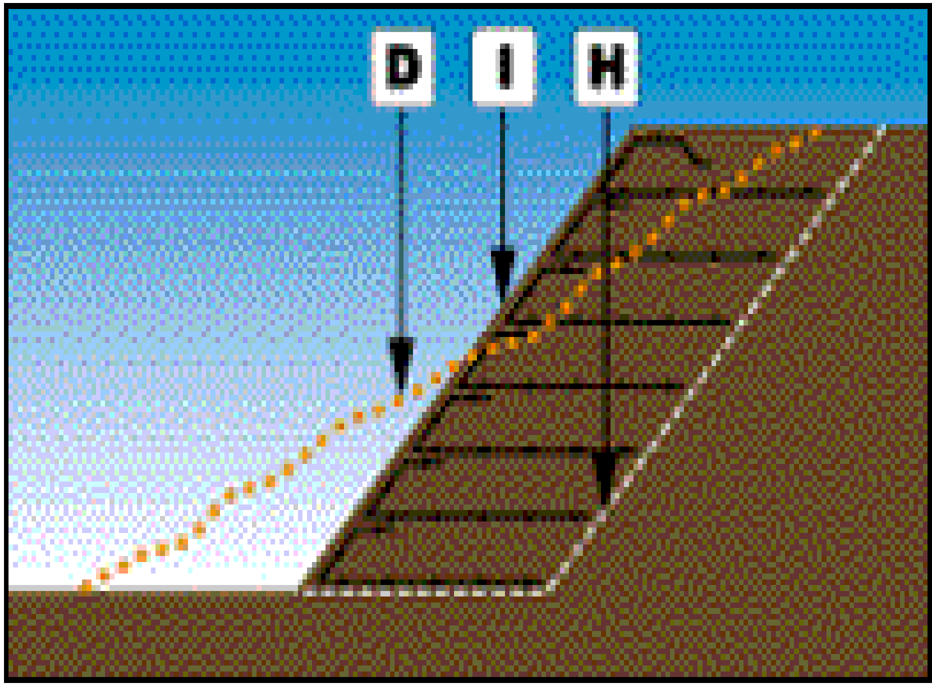
Solution:

Legend:

A - Original profile
D - Profile after failure
E - Failure surface
I - Reinforced slope profile
H - Excavation profile



Typical slope failure



Slope reinforcement

Conclusions:

1. *Low cost, fast construction and ease to shape the slopes*;
2. *Fast and excellent vegetation of the face*;
3. *Excellent stability;*
4. *Positive drainage.*

Problem #2:

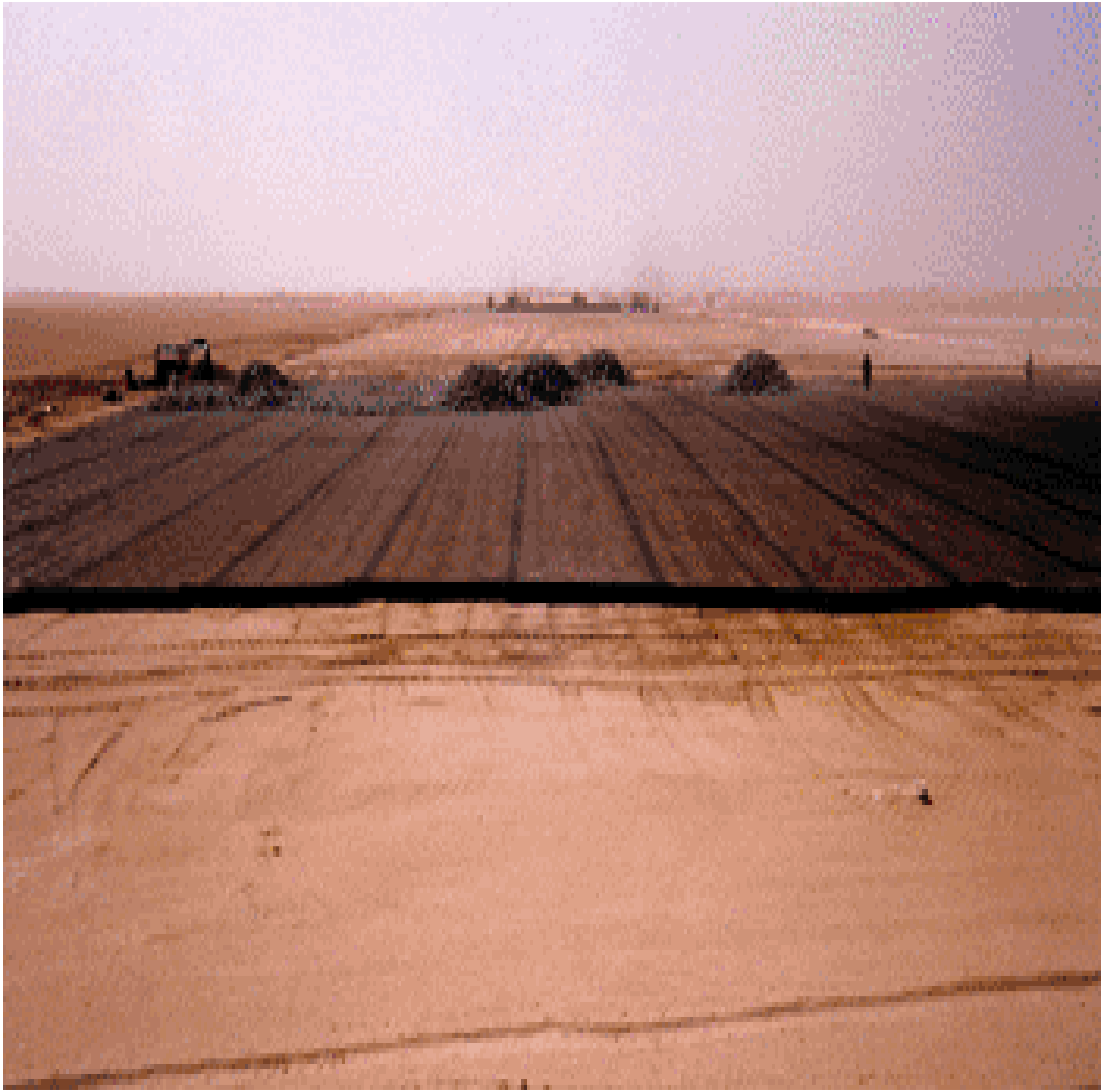
Required to stabilize a road widening and reconstruction of the Tarif-Madinat-Liwa Interchange, in order to increase the traffic flow on the main highway connecting the Emirate of Abu Dhabi with the Saudi Arabian border. It was decided to widen the existing highway from four to eight lanes.

The subgrade was identified as a typical Subkha soil with a very low bearing capacity. Importing engineered fill material would have been costly. The designers were required to increase both the bearing capacity and decrease the thickness of the fill material to decrease costs.

Solution:

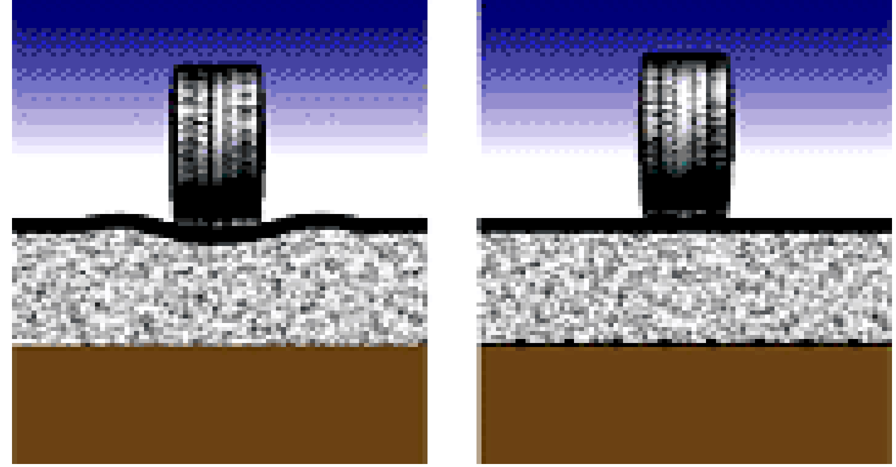
To stabilize the road embankment and to avoid differential settlement, the consultant engineers opted for an integral extruded geo-grid offering both high junction strength and resistance to mechanical damage. The same product has been used also to reinforce and decrease the thickness of imported fill of roads.

The geotextile is seen to be in place, upon the compacted in-situ soil. The subgrade material is being delived by the truck, and is being spread by the front loader in the background.

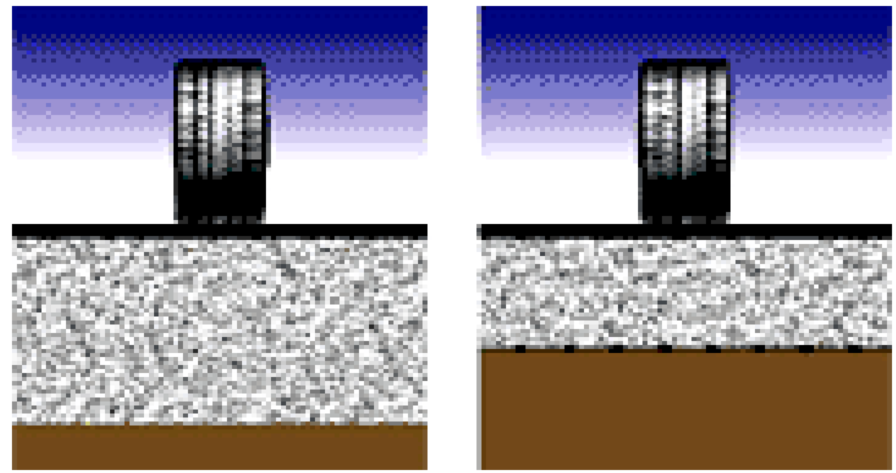


Stabilization of roads.

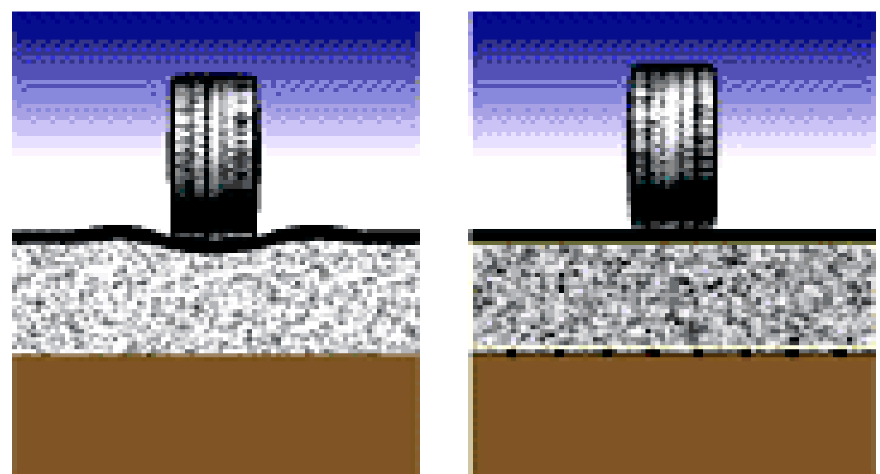
1. Reduce the progressive failure of flexible paved roads: Road service life can be increased up to a factor of 10 without changing the properties of the road components.



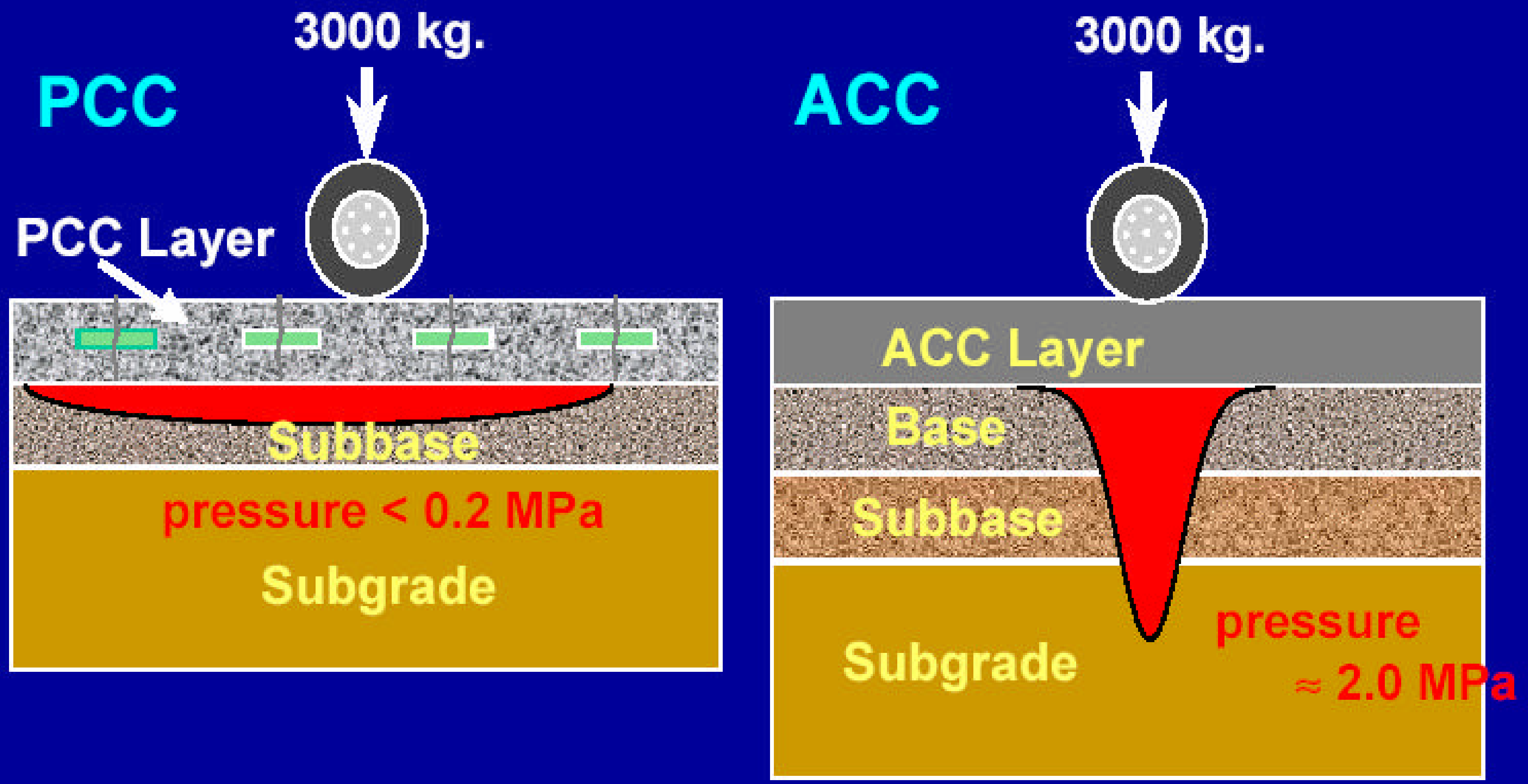
2. Reduce the thickness of the aggregate base layer without loosing performance or structural capacity.



3. The geotextile permits the use a fill material that has structural and drainage properties inferior to the one typically required, without changing the overall road thickness.



Subgrade of Pavements



Weak and/or non-uniform subgrade often cause

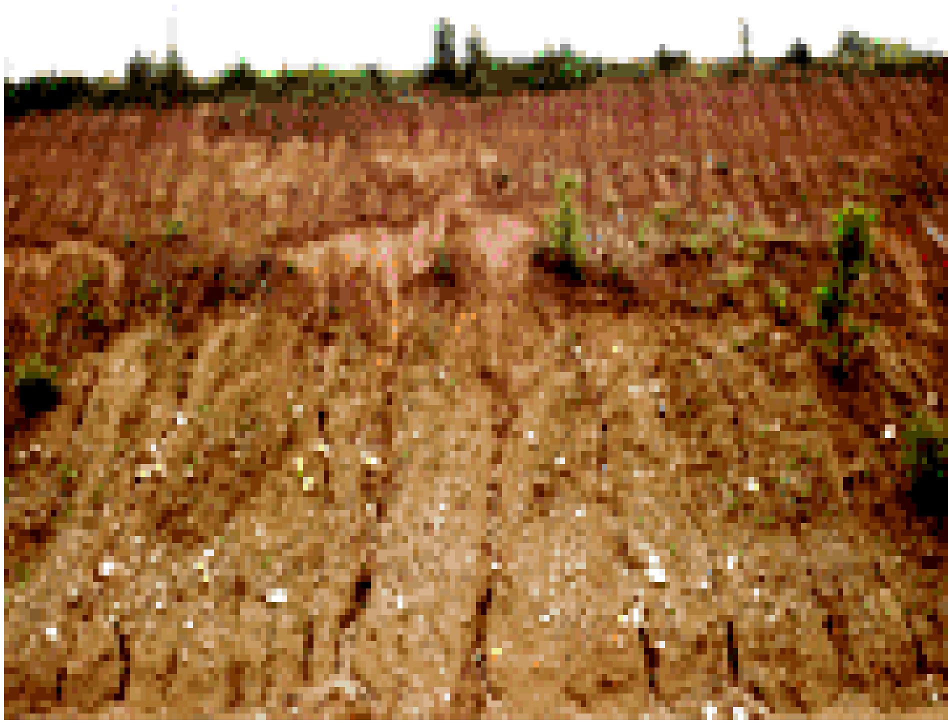
- Longitudinal cracks - Rutting - Corner Break

Problem #3:

To reduce the severe erosion of the lateral slopes of a major Italian road close to Milan.

In order to reduce the environmental impact, a new highway was built by excavating it below the existing ground elevation. Erosive forces on the surface layer undermined its stability and the loss of soil cohesion. In the long term, the slope stability was going to be compromised starting from the cut face of the upper slope where run-off, superficial erosion and small clumps of soil sliding were more likely to happen.

It was necessary to minimize the erosive effect of the flowing water and to enhance the shear resistance of the topsoil. In addition, eroded soil and pebbles were sliding down the cut slope to the road side.



Solution:

This photo shows the geotextile mats being applied to the smaller slopes to test their performance. The results were excellent, and the solution was extended to much steeper slopes, seen on the next slide.





Problem #4:

Problems ensued from hazardous drainage coming from the landfill capping system at a Cerro Maggiore (Milan, Italy) waste disposal facility. A capping system was designed to guarantee the impermeability of the landfill sides slopes, allowing for proper gas venting, with an adequate drainage system on top to avoid ground water infiltration, and with a vegetated cover system to minimize environmental impact and to protect the new geo-synthetic liner.

Solution:

The slope was waterproofed using a bentonite geocomposite (GCL) which guaranteed a performance equivalent to 1 m of clay soil with a permeability of 10-12 10^{-4} cm/s.

The geocomposite offered a complete system of "filter-drainage-protection“ which means that the geotextile acts as a water/soil filter for the geocomposite and prevents the intrusion within the drainage core of the bentonite migrating from the GCL during hydration and intrusion of the loose fill material laid directly above, thus allowing for a long term hydraulic flow.