Lecture 28

BEARING CAPACITY IMPROVEMENT using geosynthetics

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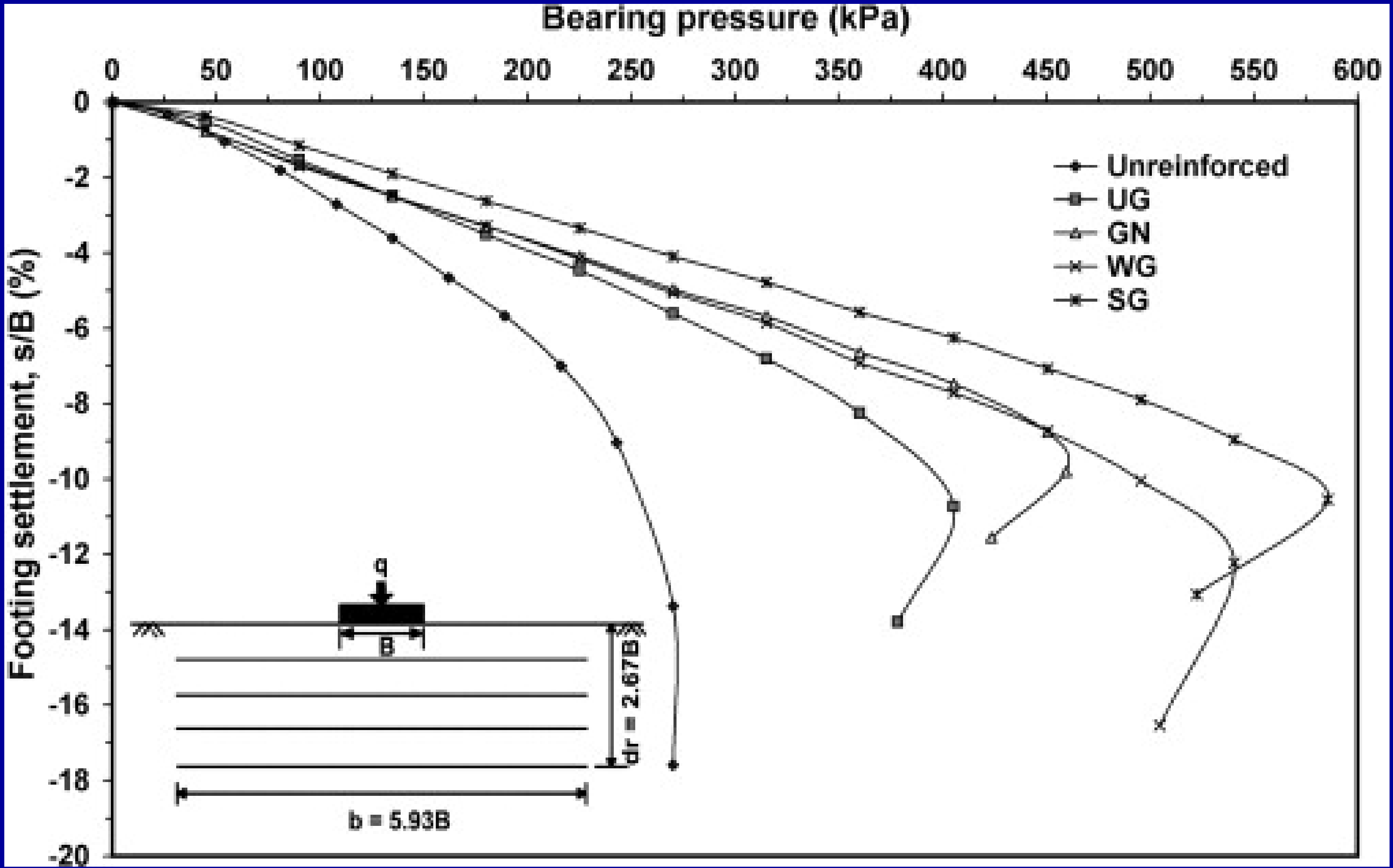
- **Modes of failure**
- **Forces in reinforcement**
- **Tension failure and pull out**
- **Design steps**
- **Examples**

Introduction

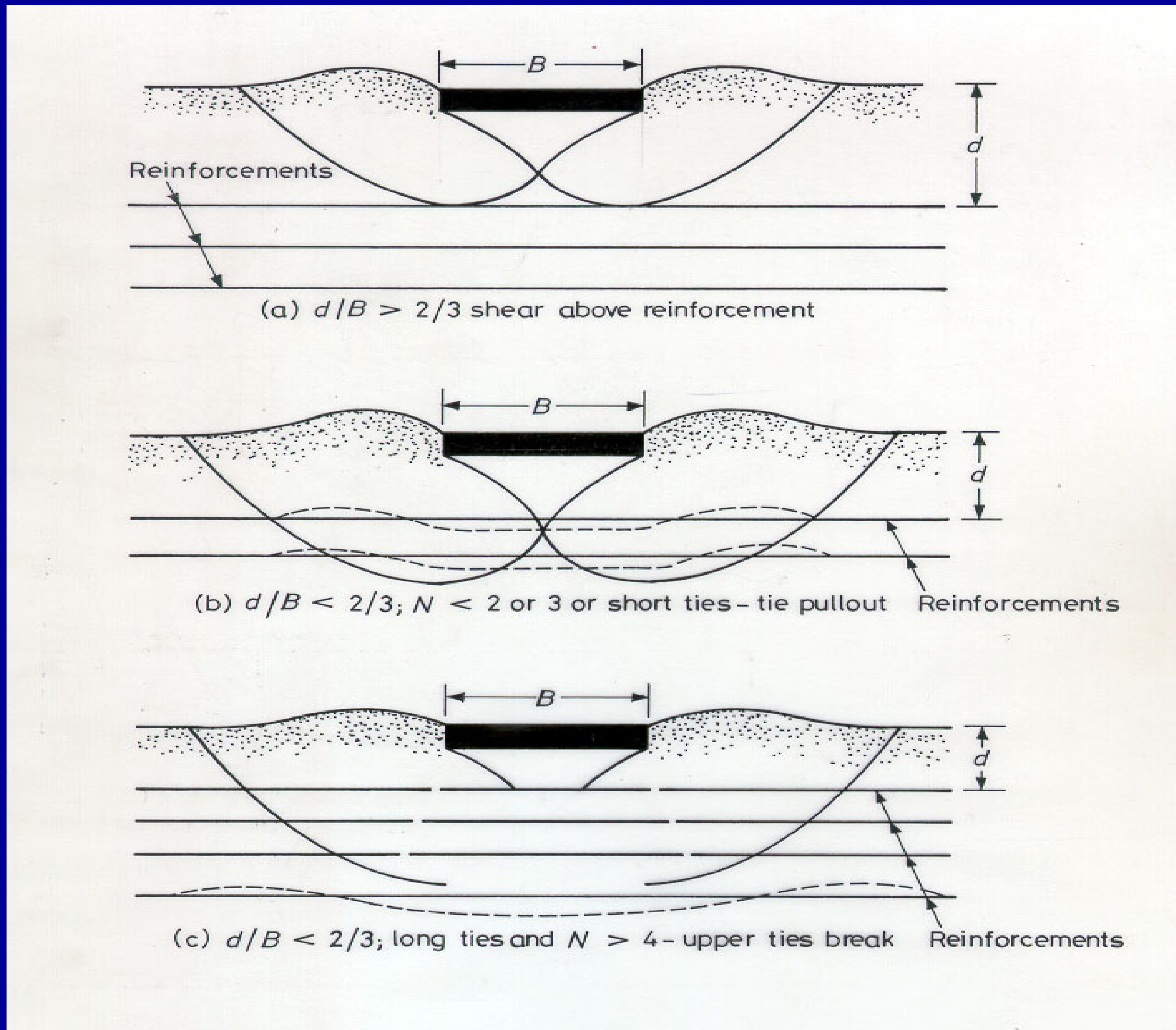
Reinforced soil technique has been used very effectively is in the improvement of bearing capacity of foundation soils. Studies by several investigators (e.g. Binquet and Lee, 1975a, 1975b; Guido et. al. 1986) clearly indicated the advantages and possibilities of improvement in bearing capacity and stiffness in terms of the load-settlement behaviour by reinforcing foundation soil in the case of sands.

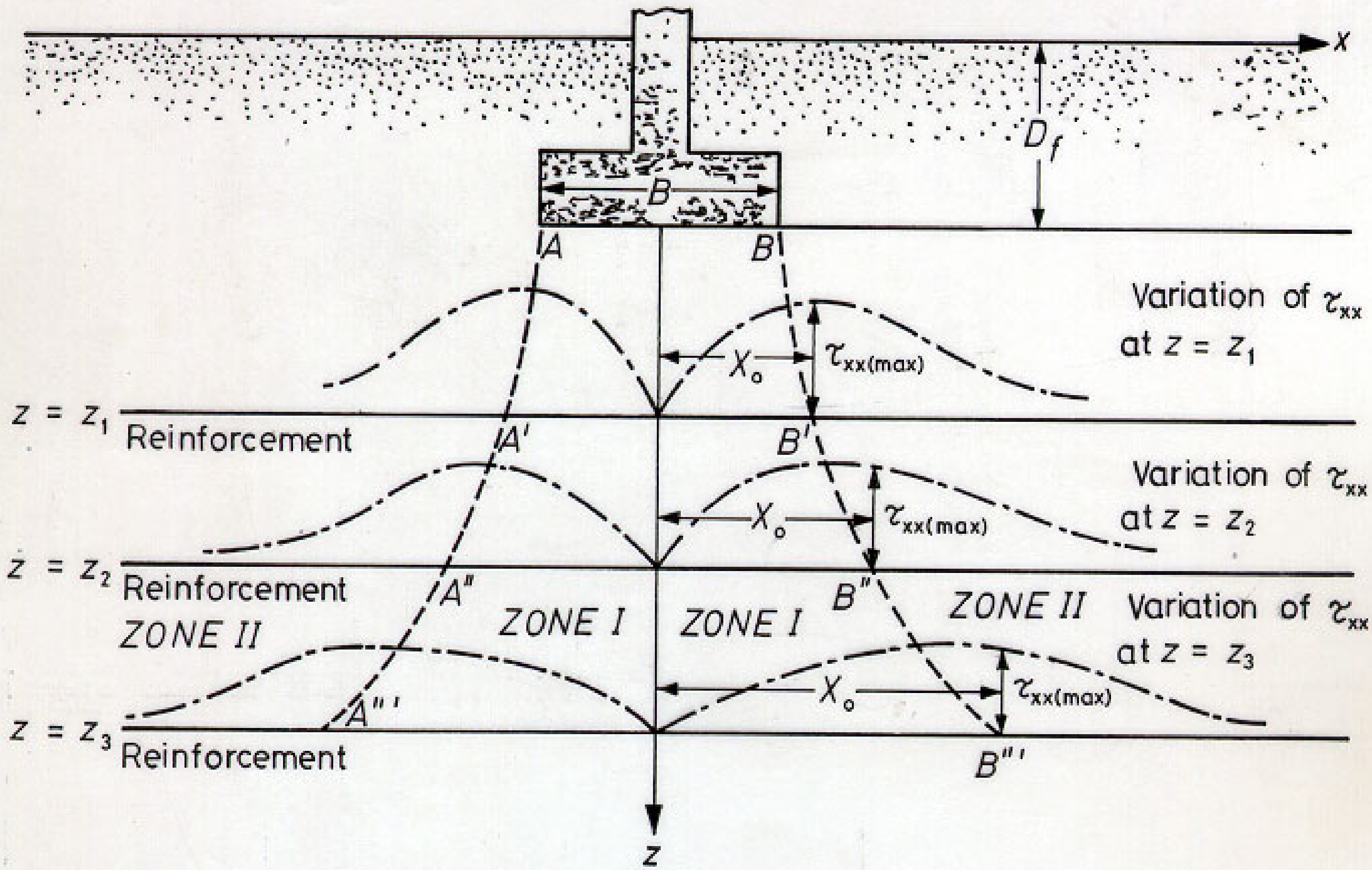
Considerable research work using experimental and analytical and numerical methods is available in literature. A number of experimental and theoretical studies have been carried out and in evolving methodologies for determination of improvement of bearing capacity in terms of bearing capacity ratio (BCR), defined as ratio of improved (or required) bearing capacity to the bearing capacity of foundation soil with out reinforcement.



Bearing capacity Improvement for different geosynthetic products (Madhavi Latha and Amit Somwanshi 2009)

Binquet and Lee (1975) proposed a method to evaluate the improvement in bearing capacity. When some form of reinforcement is used in the foundation soil, it is necessary to identify the modes of failure due to the presence of reinforcement so that the effectiveness of the reinforced foundation is examined. To develop a design procedure, Binquet and Lee idealized that the following mechanisms of failure are likely to occur in a reinforced soil foundation.


B
d
Reinforcements
(a) $d/B > 2/3$ shear above reinforcement
B
d
Reinforcements
(b) $d/B < 2/3$; $N < 2$ or 3 or short ties – tie pullout
B
d
Reinforcements
(c) $d/B < 2/3$; long ties and $N > 4$ – upper ties break



x
D_f
B
A
B'
Variation of τ_{xx} at $z = z_1$
X_o
$\tau_{xx(max)}$
$z = z_1$
Reinforcement
A'
B'
Variation of τ_{xx} at $z = z_2$
X_o
$\tau_{xx(max)}$
$z = z_2$
Reinforcement
A''
B''
ZONE II
ZONE I
ZONE I
ZONE II
Variation of τ_{xx} at $z = z_3$
X_o
$\tau_{xx(max)}$
A'''
$z = z_3$
Reinforcement
B'''
z

(a)

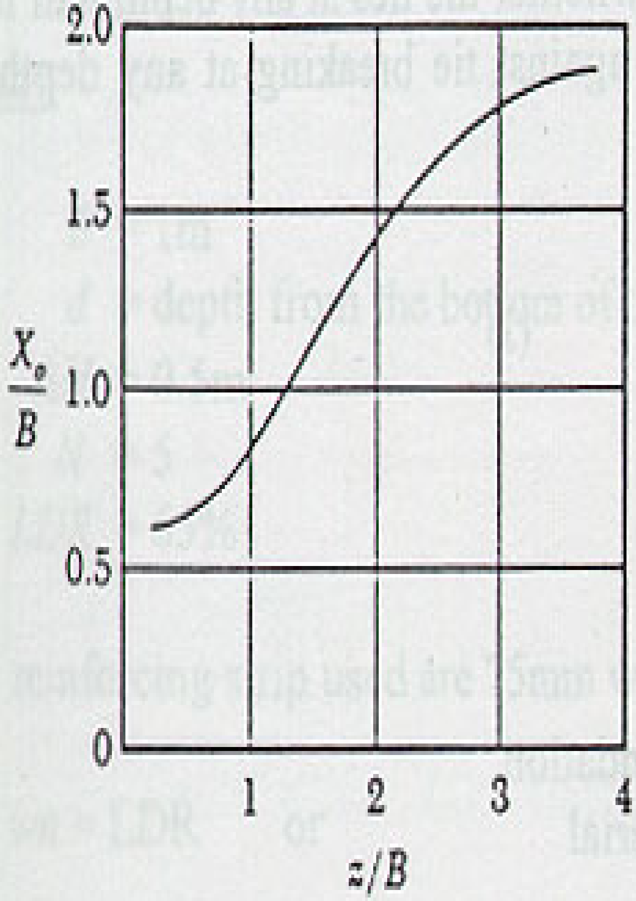
The points A’, A”, A’”, …. , and B’, B”, B’”,….., which define the limiting lines between zones I and II, can be obtained by considering the shear stress distribution, Υxz’ in the soil caused by the foundation load. The term τxz refers to the shear stress developed at a depth z below the foundation at a distance x measured from the centerline of the foundation.

For *N* reinforcing layers, the ratio of the load per unit area on the foundation supported by reinforced earth, q_R, to the load per unit on the foundation supported by unreinforced earth, q_0, is constant irrespective of the settlement level, *s*

The term τ_{xz} refers to the shear stress developed at a depth z below the foundation at a distance x measured from the centerline of the foundation.

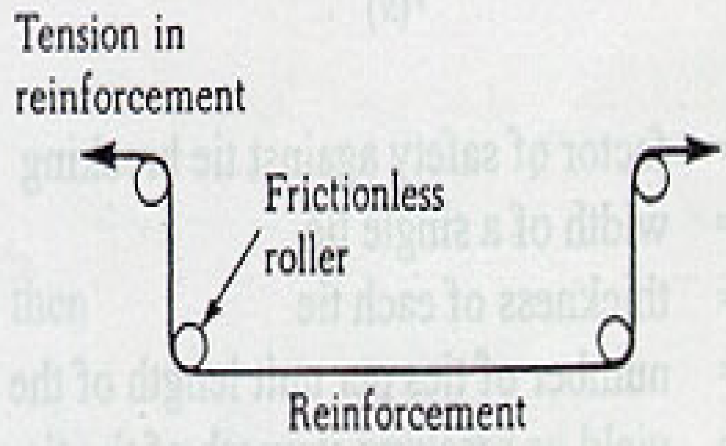
$$\tau_{xz} = \frac{4bq_R xz^2}{\pi\left[\left(x^2 + z^2 - b^2\right)^2 + 4b^2z^2\right]}$$

b= Half-width of the foundation = B/2, *B* = width of foundation, q_R = load per unit area on the foundation



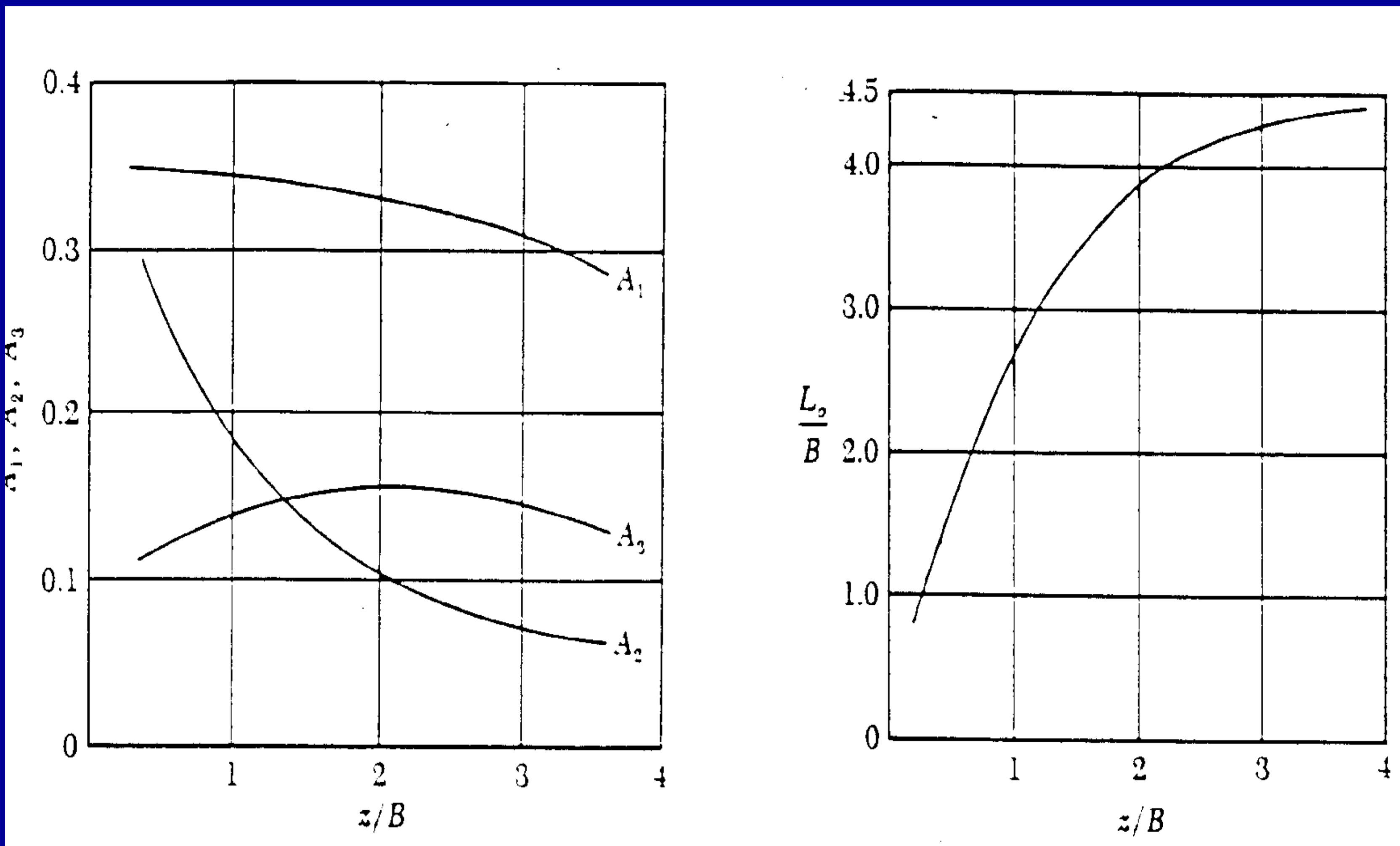
2.0
1.5
1.0
0.5
0
X_o/B
1
2
3
4
z/B

(b)



Tension in reinforcement
Frictionless roller
Reinforcement

(c)



0.4
0.3
0.2
0.1
0
A_1, A_2, A_3
A_1
A_3
A_2
1
2
3
4
z/B
4.5
4.0
3.0
2.0
1.0
0
$\frac{L_o}{B}$
1
2
3
4
z/B

Forces in reinforcement

Tension failure

$$T_{(N)} = \left(\frac{q_o}{N}\right)\left(\frac{q_R}{q_o} - 1\right)\left(A_1 B - A_2\, \Delta H\right)$$

Where $q_{0,}$ the bearing capacity of unreinforced soil, q_R, the bearing capacity of reinforced soil and A_1 and A_2, the functions of z

The factor of safety against tie breaking at any depth z below the foundation can be calculated as

$$FS_{(B)} = \frac{wtnf_y}{T_{(N)}}$$

$FS_{(B)}$ = FS against tie breaking,

w = width of a single tie

t = thickness of each tie

n = number of ties per unit length of the foundation

f_y = yield strength of the tie material

$T_{(N)}$ = Tension in the tie member

The term w_n (linear density ratio, LDR) is

$$FS_{(B)} = \left[\frac{tf_y}{T_{(N)}}\right](LDR)$$

The resistance against the tie being pulled

F_B = 2 tan ϕ_μ [normal force]

$$F_B = 2\tan\varphi t\left[(LDR)\left[A_3 B q_0\left(\frac{q_R}{q_0}\right) + (\gamma)\left(L_0 - X_0\right)\left(z + D_f\right)\right]\right]$$

Design Example

Design a continuous foundation that will carry a load of 480kN/m using the following parameters.

Soil: γ=16 kN/m^3; φ=30^o; E_s=3x10^{4}kN/m^2; μ_s=0.35

Reinforcement ties: f_y=2.5x10^5 kN/m^2 ;φ_μ=20^o

F.S against tie breaking=3.0; F.S against pullout=2.0

Foundation: D_f=1.0m; Allowable B.C=160 kPa;

Design life=50 Years;

Assume: B=1.0m; ΔH=0.5m;N=4; LDR=65%;

d=0.5m; width of reinforcement=75mm;

w_n=LDR

n= 0.65/0.075m=8.67/m

Hence each layer will consists of 8.67 strips per meter length of the foundation.

$q_0 = q_{all}$= 160kPa; q_R = 480kPa

BCR = 480/160 = 3

Calculation of tie force:

$$T_{(N)} = (\frac{q_0}{N})(\frac{q_R}{q_0} - 1)(A_1 B - A_2 \Delta H)$$

Table1(b): Calculation of tensile forces in reinforcement members

Calculation of tie force for each layer							
Layer No.	$(q_o/N)(q_R/q_o-1)$	z(m)	z/B	A_1B	$A_2\Delta H$	$A_1B-A_2\Delta H$	$T_{(N)}$(kN/m)
1	80	0.5	0.5	0.35	0.13	0.23	18.0
2	80	1	1	0.34	0.09	0.25	20.0
3	80	1.5	1.5	0.33	0.07	0.27	21.2
4	80	2	2	0.32	0.05	0.27	21.6

Calculation of Tie Resistance due to friction, F_B

$$F_B = 2\tan\phi_\mu \left[(LDR) \left[A_3 B_{qo} \left(\frac{q_R}{q_o} + (\gamma)(L_0 - X_0)(z + D_f) \right) \right] \right]$$

Calculation of Pull-out Lengths				
Quantity	Layer No			
	1	2	3	4
$2\tan\phi_\mu(LDR)$	0.473	0.473	0.473	0.473
A_3	0.125	0.14	0.15	0.15
$A_3Bq_o(q_R/q_0)$	60	67.2	72	72
z(m)	0.5	1	1.5	2
z/B	0.5	1	1.5	2
Lo(m)	1.55	2.6	3.4	3.85
X0(m)	0.55	0.8	1.1	1.4
(lo-X0)(m)	1	0.8	1.1	1.4
z+Df(m)	1.5	2	2.5	3
$\Upsilon(lo-X_0)(z+D_f)$	24	57.6	92	117.6
F_B(kN/m)	39.73	59.03	77.6	89.68
$FS_{(p)}=F_B/T_{(N)}$	**2.21**	**2.95**	**3.66**	**4.15**

Calculation of Tie-thickness to resist tie breaking:

$$FS_{(B)} = \frac{tf_y}{T_{(N)}}(LDR)$$

Layer 1, t=(1.846x10-5)(20)x1000=0.33mm.

Similarly for layer 2, t=0.33mm

layer 3, t=0.37mm

Layer 4, t=0.40mm

If galvanized steel is used rate of corrosion is 0.025mm/year.

t = 0.4+(0.025x50

= 1.65mm

Normally, a minimum thickness of 6.0mm thickness is available.

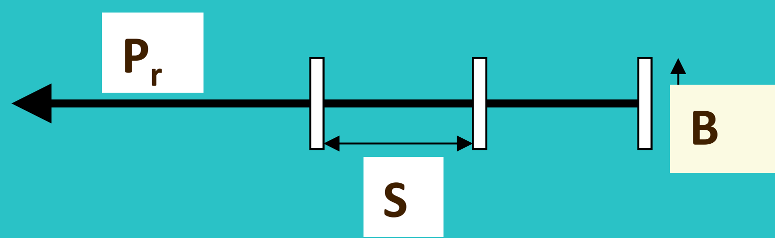
Hence strips of 75mm wide & 6mm thick are chosen for present case.

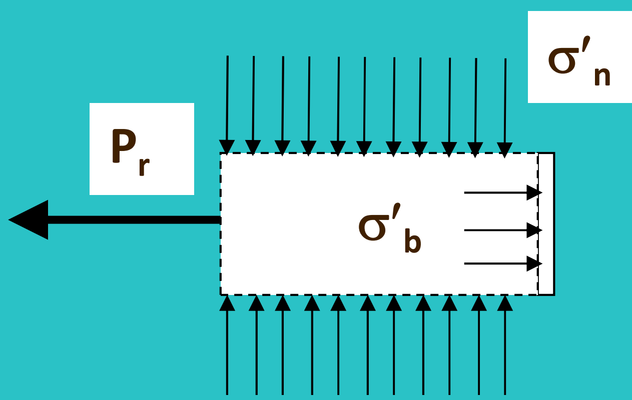
Curtailment of Pull-out length of reinforcement

- Total length of Reinforcement in 4th layer= $2xL_0$ = 2x3.85= 7.7m
- In order to reduce the reinforcement length & increase pullout or bond resistance, Anchors or strips are provided in transverse direction.

The mobilized Bond Resistance is given by

$$B\sigma'_b = 2S\sigma'_n \tan\phi'$$





To be fully rough:

$$2\ S\ \sigma'_n \tan \phi' = B\ \sigma'_b$$

σ'_b= Bond Stress

φ' = Friction angle of back fill

σ'_n = Effective normal stress

S = Spacing of anchors

- A Theoretical relationship between bond stress ratio & the back fill soil properties obtained from limit equilibrium considerations using bearing capacity theory is given by

$$\frac{\sigma'_b}{\sigma'_n} = \tan(\pi/4 + \phi'/2)e^{(\pi/2+\phi')\tan\phi'}$$

For $\varphi' = 30^0$

Ultimate Bond stress ratio= $\tan(45+15)\ e^{1.209} = 5.8$

Safe Bond ratio=5.8/1.5=3.867=2S/100Xtanφ'

S=334.86mm

≈ 300.0mm

The length of the reinforcement is taken as 3.0m,& 10 anchor plates at a spacing of 300 mm c/c are provided.

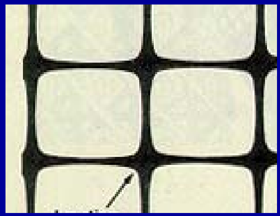
Calculation of Bond resistance of Anchor plates

Calculation of bond resistance of anchor plates

Calculation of Bond Resistance of Anchor Plates				
$(z+D_f)$(m)	Effective normal Stress(kPa)	Bond resistance per Anchor(kPa)	Tensile force per meter length(kN/m)	Factor of safety against pullout
1.5	24	92.8	18	5.16
2	32	123.7	20	6.19
2.5	40	154.7	22	7.03
3	48	185.6	22.4	8.29

Factor of safety against pull out is is >2.0; Hence O.K.

Guidelines for use of bi-axial geogrids for bearing capacity



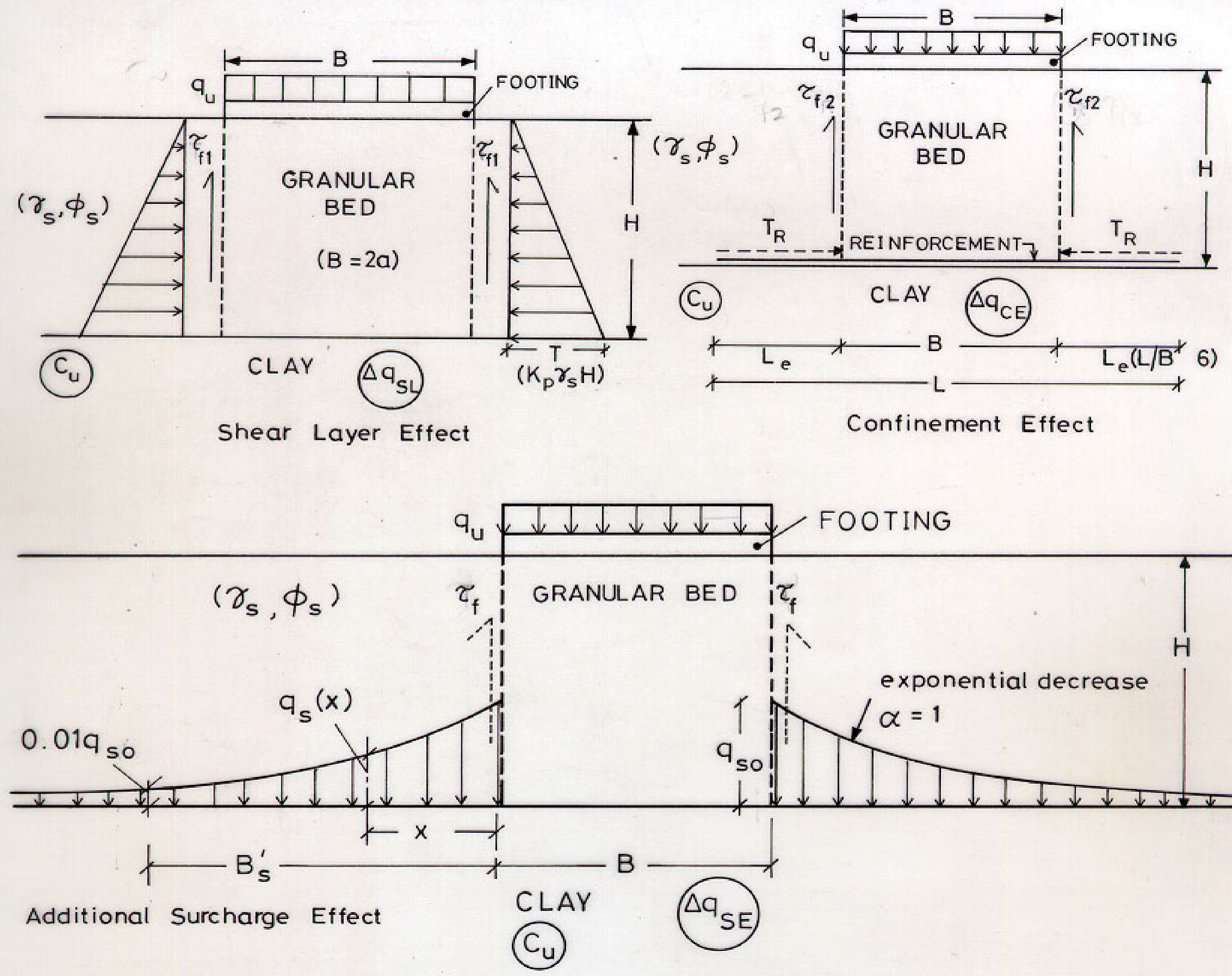
	Typical value	Recommended (not greater than)
u	0.15B to 0.3B	0.5B
s	0.15B to 0.3B	0.5B
z	0.5B to 1.0B	2.0B
b	2.0B to 3.0B	4.0B
a	0.1B to 0.2B	0.3B
Δl	0.5B to 1.0B	2.0B
N	2 to 4	5

Improvement of bearing capacity of soft soils

Use of sand and aggregates,
Use of geotextiles, geogrids and geocells

The improvement is attributed to three effects a) shear layer effect b) confinement effect due to the interaction between sand and reinforcement in the sand layer and c) additional surcharge effects.

The ultimate bearing capacity of a footing resting on soft soil $q_u = C_u N_c$


B
FOOTING
q_u
τ_{f1}
τ_{f1}
(γ_s, ϕ_s)
(γ_s, ϕ_s)
GRANULAR BED
(B = 2a)
H
T
$(K_p \gamma_s H)$
C_u
CLAY
Δq_{SL}
Shear Layer Effect
B
FOOTING
q_u
τ_{f2}
τ_{f2}
GRANULAR BED
H
T_R
REINFORCEMENT
T_R
C_u
CLAY
Δq_{CE}
L_e
B
L_e(L/B 6)
L
Confinement Effect
q_u
FOOTING
(γ_s, ϕ_s)
τ_f
GRANULAR BED
τ_f
H
exponential decrease
$\alpha = 1$
$q_s(x)$
$0.01 q_{so}$
q_{so}
x
B'_s
B
Additional Surcharge Effect
CLAY
C_u
Δq_{SE}

When a granular bed of thickness (H) of bulk density (γ_s) and friction angle (ϕ_s) with reinforcement is provided over soft soil, the bearing capacity of the footing resting on this foundation medium is increased. Frictional forces developed between the soil and the reinforcement induce tensile strains in the reinforcement. The tensile strains developed provide the confining effect. This will induce additional shearing resistance along the vertical plane at the edge and exponentially decrease with distance away from the edge of the footing. The three effects contribute to increase in bearing capacity, given as

$$q_u + \Delta q_R = C_u N_c + \Delta q_{SL} + \Delta q_{CE} + \Delta q_{SE}$$

$T_{f1} =$

$\tan\phi_s$ (4.14)

An example illustrating the application of the above formulation is given below.

Design a continuous foundation (B = 1m) that will carry a load of 480 kN/m that needs to be constructed on a soft soil and the undrained strength (C_u) of the soil is 10 kPa.

Solution

The ultimate bearing capacity of the soft soil

$$(q_u) = C_u N_c = 10 \times 5.14 = 51.4 \text{ kPa}$$

(i) A granular bed of 2m thick (design friction angle, φ_s = 30^o and unit weight = 18 kN/m^3) to introduce shear layer effect is considered. The effect of the granular bed is given by

$$T_{f1} = \frac{k_p \gamma H^2}{2} \tan \phi_s$$

$$= \left(\frac{1 - \sin \phi}{1 + \sin \phi} \right) * \frac{18 * 2^2}{2} * \tan 30 = 62.35 kN / m$$

Bearing capacity improvement due to shear layer effect

= (2*62.35)/1= 124.7 kN/m

ii) Consider that a geotextile layer is laid at the interface of the clay and sand bed.

The tensile forces are generated in the reinforcement s a result of friction between the granular soil and the reinforcement. If L_e is the effective length, ϕ_R is the friction angle between the reinforcement and granular soil, LDR is the linear density, ratio of reinforcement material (LDR = 1 for geosynthetics and 0.5 to 0.7 for metallic grids).

The reinforcement force $(T_R) = (\gamma_s H) \tan\phi_R . L_e (LDR)$

$$T_R = 18*2*\tan 30*3*1 = 62.36$$

Bearing capacity due to confinement effect = 2 x 62.36 =124.70 kN/m

The contribution due to surcharge effect

= 0.84 (124.7+124.7) = 209.5 kN/m

Total improvement = 124.7+124.7+209.5 = 458.9 kN/m

Hence, it can be noted that the original ultimate bearing capacity of the soft soil is likely to increase from 51.4 kN/m to 458.9 kN/m, owing to the contribution in improvement of bearing capacity from shear layer effect, confining effect and surcharge effect. However, it is desirable that the improvement needs to be examined in relation to results that can be obtained from testing of a trial foundation.

Thank You