Stability Analysis

5.3 Overall Shear Failure

Overall shear failure modes：

Push In

Basal Heave

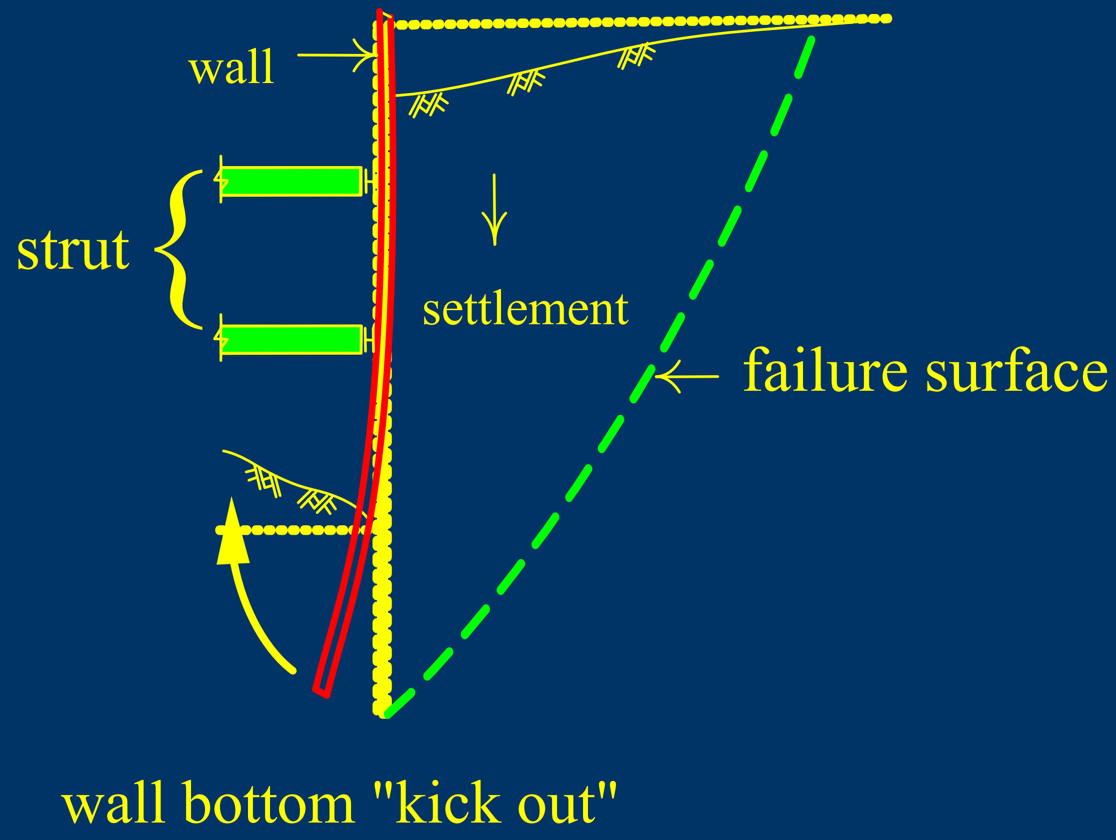


FIGURE 5.1 Overall shear failure modes (a) push-in

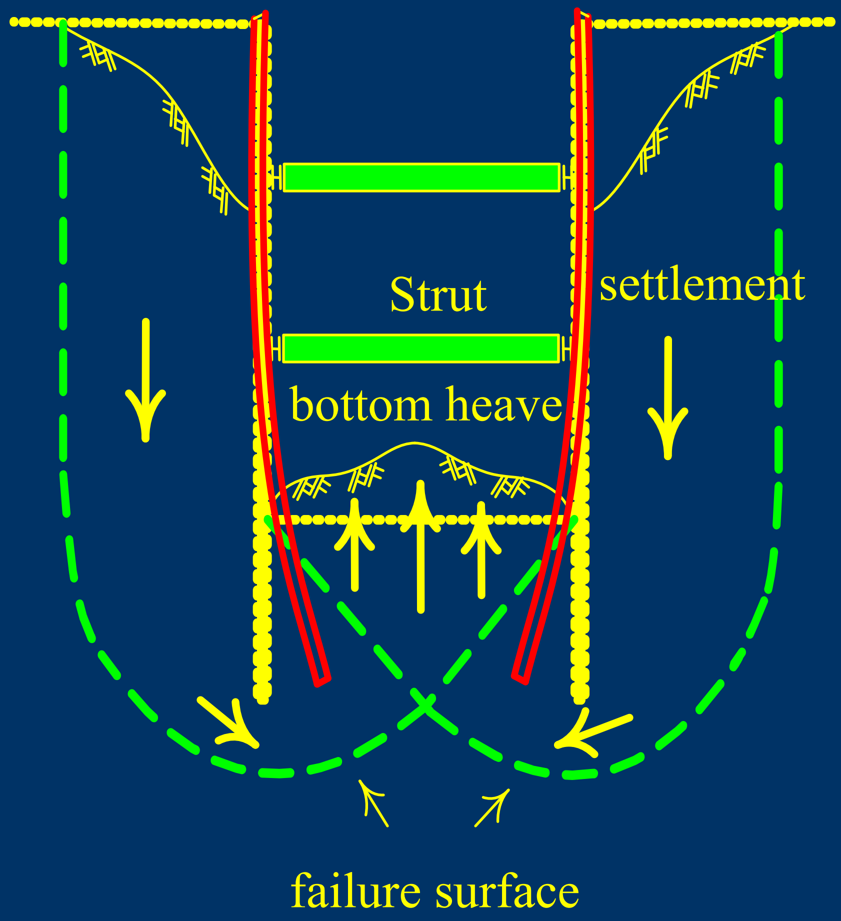


FIGURE 5.1 Overall shear failure modes (b) basal heave

5.5 Overall Shear Failure of Strutted Walls

5.5.1 Push-in

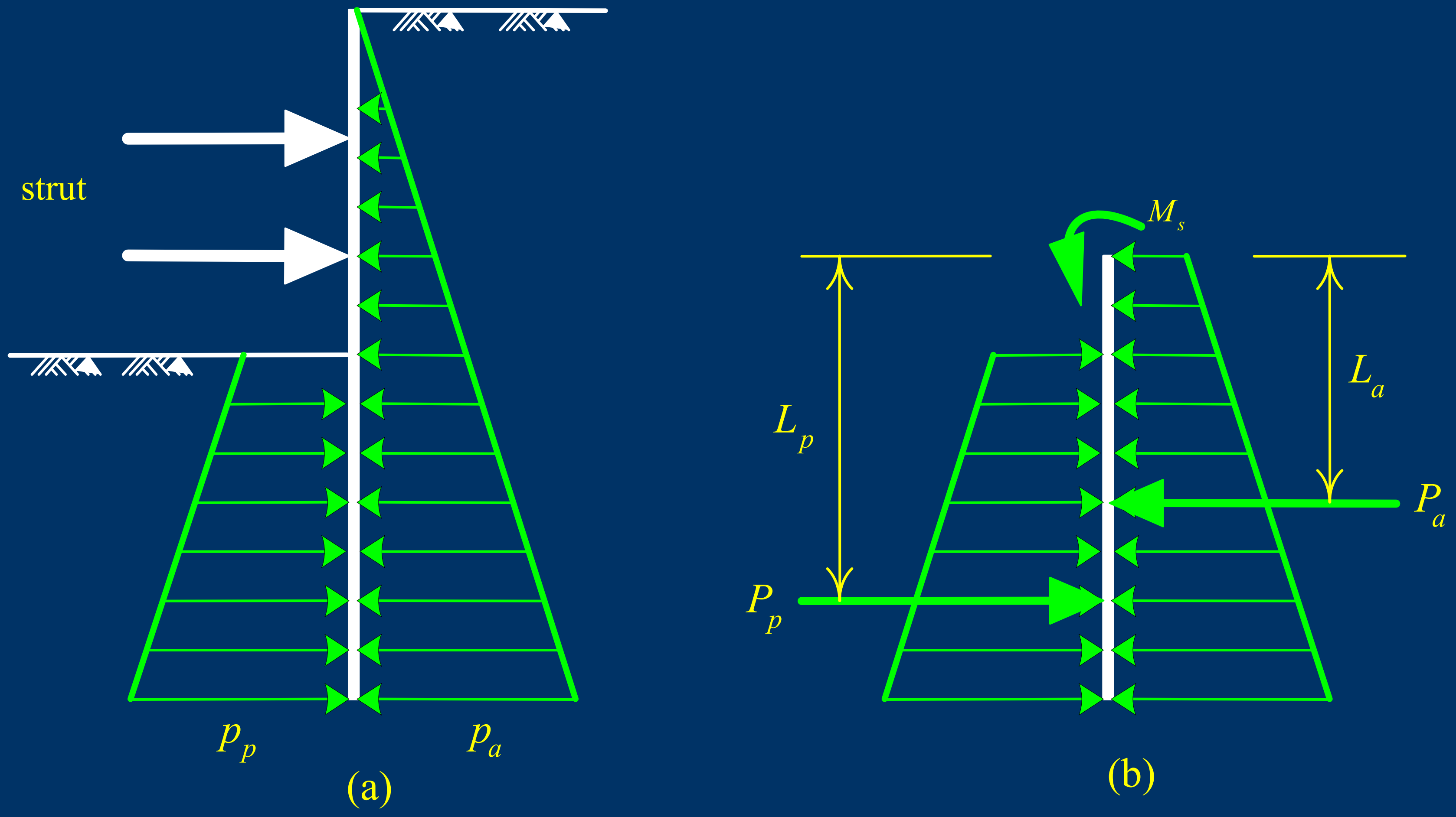


FIGURE 5.4 Analysis of push-in by gross pressure method
(a) distribution of gross earth pressure
(b) force equilibrium of the retaining wall as a free body

The factor of safety against push-in:

$$F_p = \frac{M_r}{M_d} = \frac{P_p L_p + M_s}{P_a L_a} \tag{5.5}$$

Distribution of earth pressures for cohesive soil：

$$\sigma_a = \sigma_v K_a - 2cK_{ac} \qquad (4.16)$$

$$K_{ac} = \sqrt{K_a \left(1 + \frac{c_w}{c}\right)} \qquad (4.17)$$

$$\sigma_p = \sigma_v K_p + 2cK_{pc} \qquad (4.18)$$

$$K_{pc} = \sqrt{K_p \left(1 + \frac{c_w}{c}\right)} \qquad (4.19)$$

$$c_w = \alpha s_u \qquad (5.6)$$

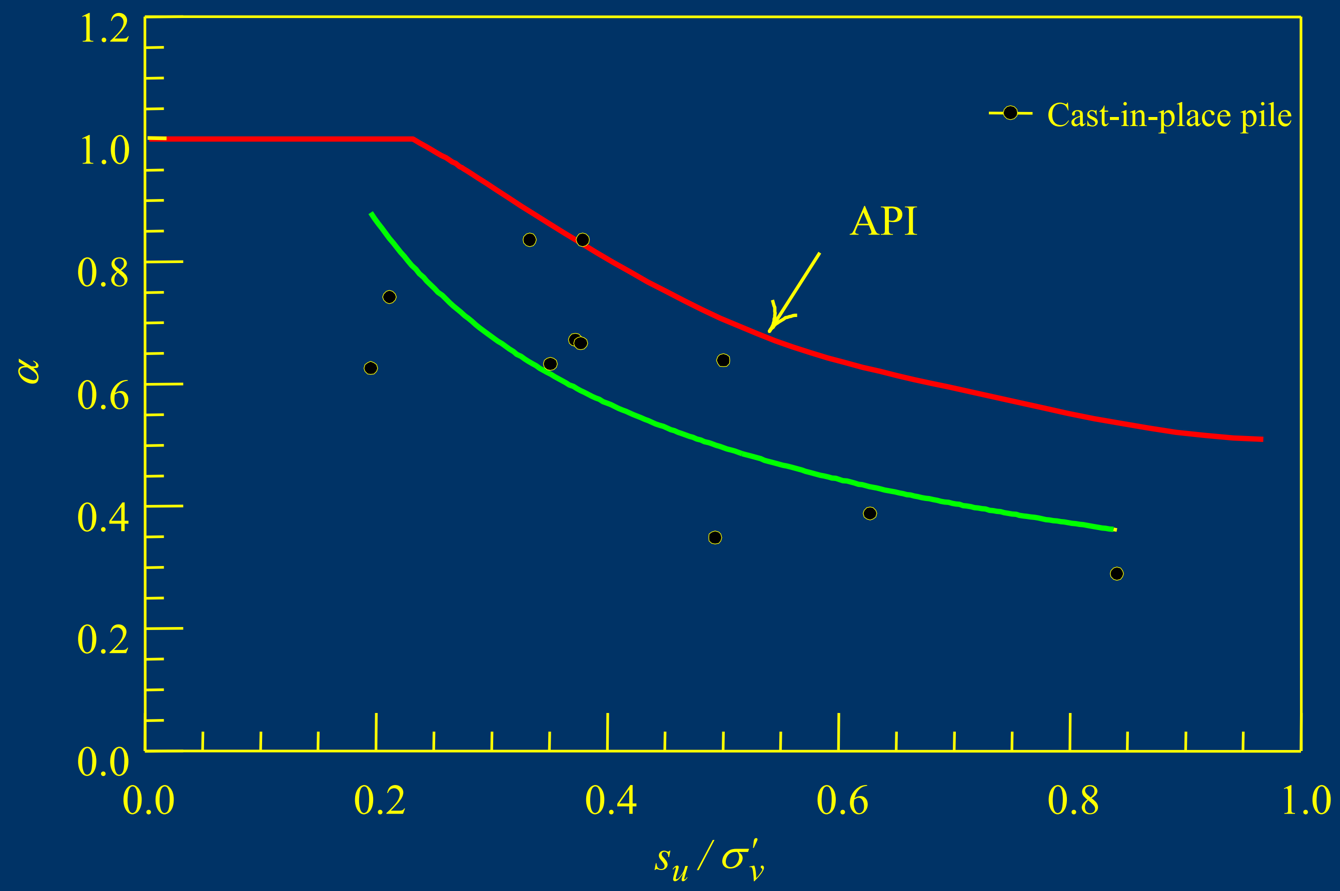


FIGURE 4.12 Relation between adhesion and undrained shear strength of clay

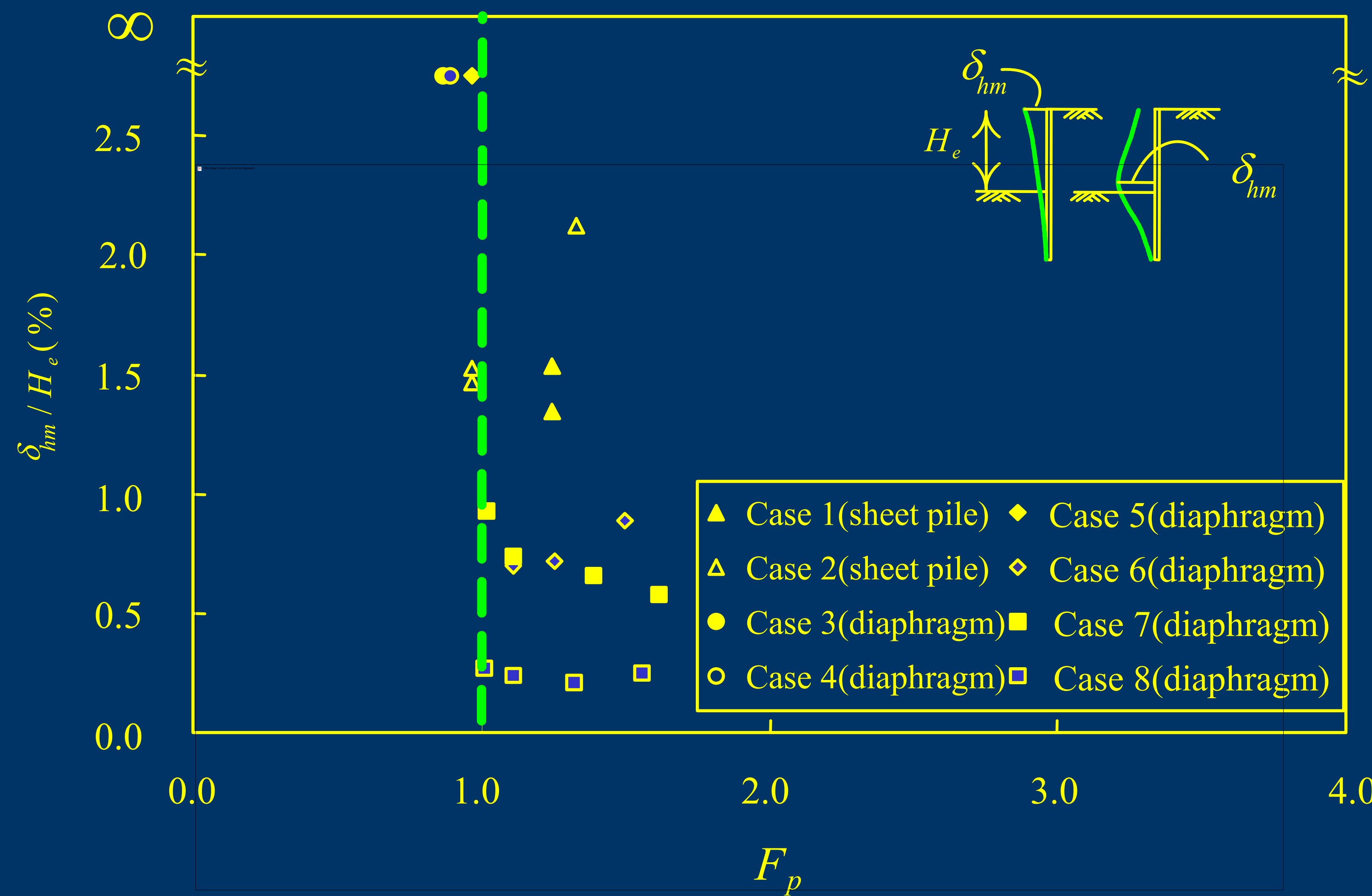


FIGURE 5.6 Factors of safety against push-in for excavations in clayey soils where Cases 3, 4, 5 are failure cases and the others are safe cases (assuming $c_w = 0.67 s_u$ for diaphragm walls, $c_w = 0.5 s_u$ for sheet piles)

The excavation depth of TNEC was 19.7 m and its retaining wall was 35 m deep diaphragm wall. The lowest level of struts was 3.2 m above the excavation surface. The soil at the site was mainly composed of normally consolidated clay.

TABLE 5.1 Relationship between depth of the diaphragm wall (or penetration depth) for the assumed excavation case and c_w

	$F_p=1.2$	$F_p=1.3$	$F_p=1.5$
$c_w=0.0$	50.4(30.7)	60.4(40.7)	96.6(76.9)
$c_w=0.33s_u$	39.5(19.8)	45.3(25.6)	63.1(43.4)
$c_w=0.50s_u$	35.7(16.0)	40.3(20.6)	53.7(34.0)
$c_w=0.67s_u$	32.7(13.0)	36.4(16.7)	46.8(27.1)
$c_w=1.00s_u$	28.4(8.7)	30.9(11.2)	37.7(18.0)

Numbers in ( ) represent the "penetration depth."

$c_w = 0.67 s_u$ (Diaphragm walls)

$c_w = 0.5 s_u$ (Steel sheet piles)

Factor of safety $F_p = 1.2 \sim 1.3$

Cohesionless soil（sandy, gravel）

Distribution of water pressures ：

Gross water pressure distribution ？

Net water pressure distribution ?

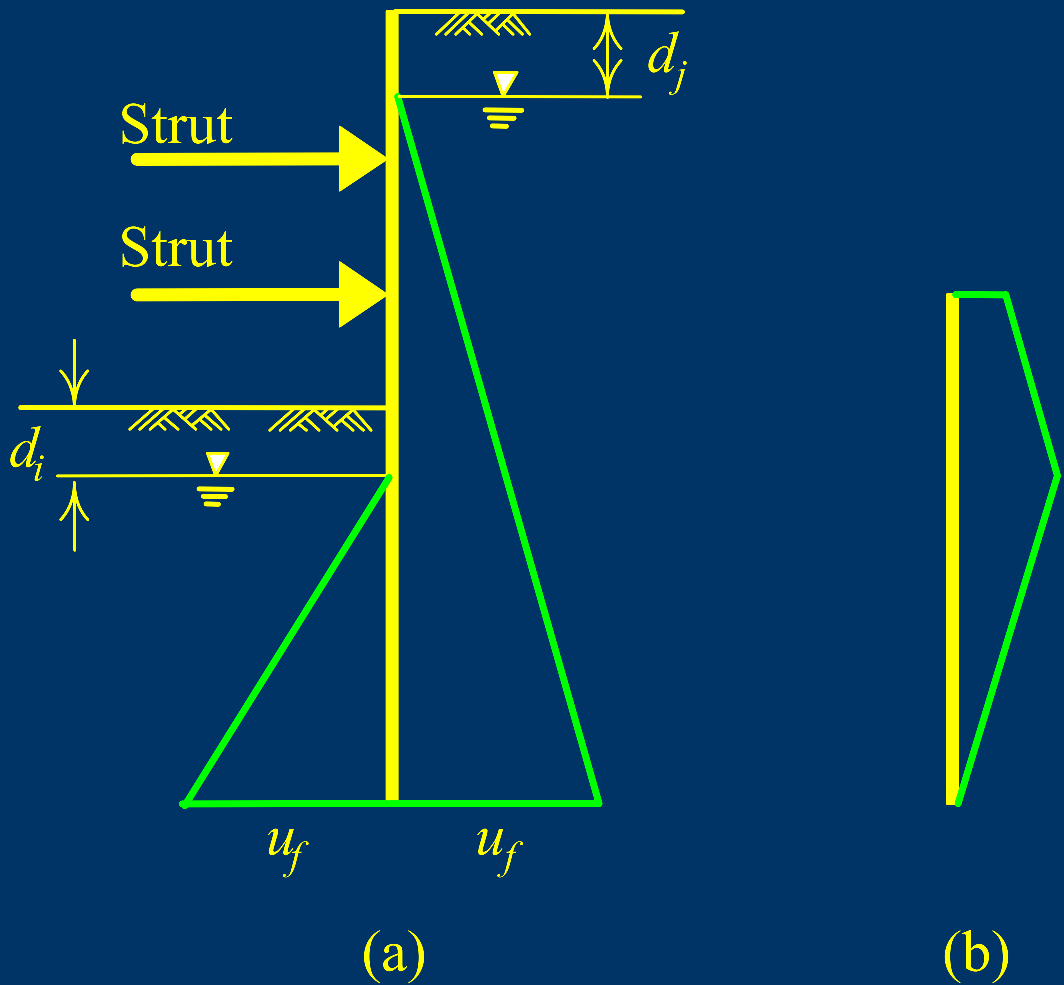


FIGURE 5.7 Distribution of water pressure due to seepage (a) distribution of water pressure (b) net water pressure (note: u_f = water pressure due to seepage)

Cohesionless soil（sandy, gravel）

Distribution of earth pressures：

Caquot-Kerisel's or Coulomb's active earth pressure should be adopted for the active earth pressure.

Caquot-Kerisel's passive earth pressure should be adopted for the passive earth pressure. When $\delta<\phi'/2$, Coulomb's passive earth pressure coefficient is quite close.

Caquot-Kerisel's earth pressure theory's K_a ,K_p and δ have some relationship. Section 4.5.3 has summarized some findings on values of δ .

Clough's research：concluded that between concrete (cast in steel mold) and sand, δ is about $0.8\phi'$.

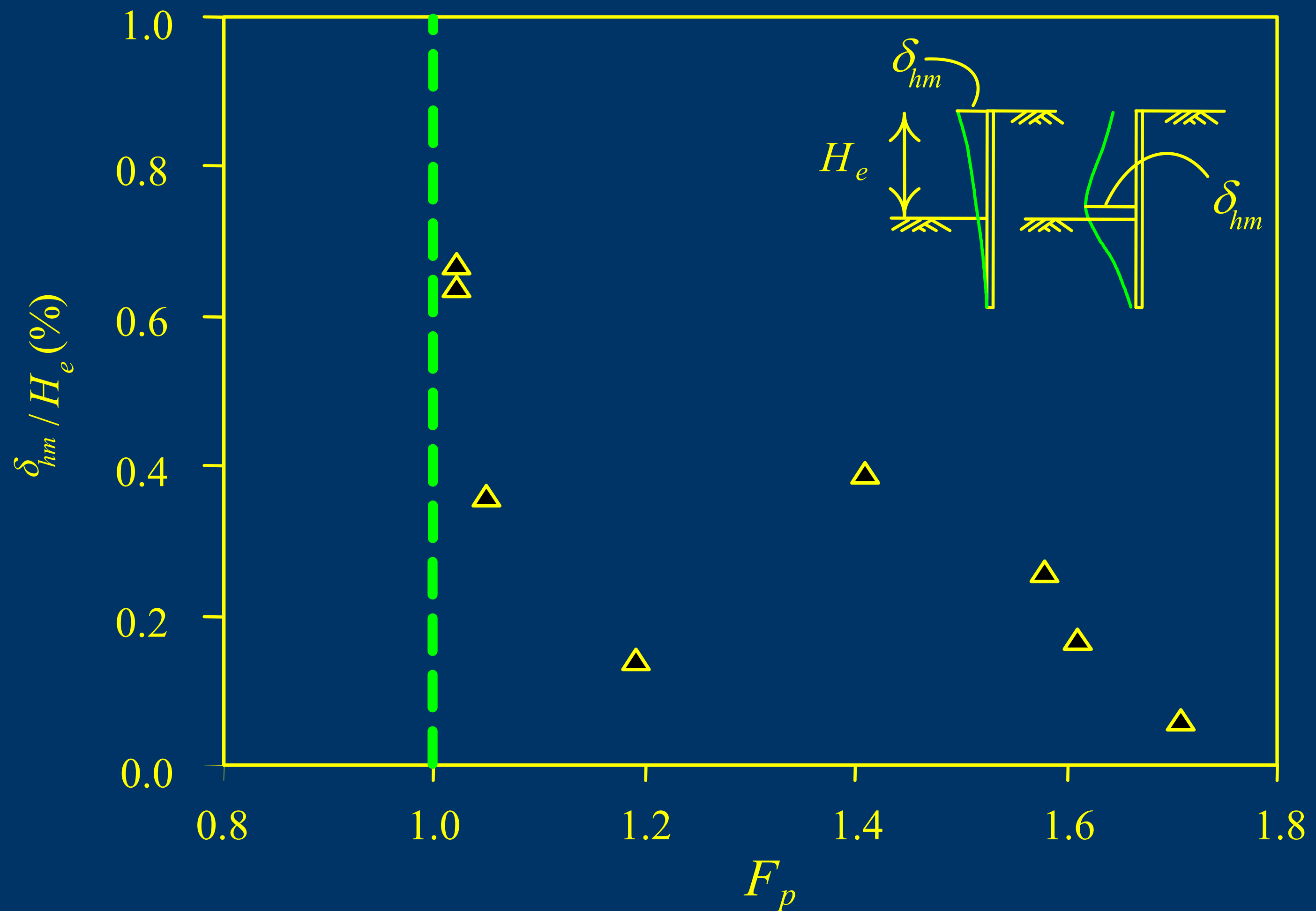


FIGURE 5.8 Factors of safety against push-in for excavations in sand (all cases are safe cases; $\delta = \phi'$ is assumed)

Conclusion：

Assumption that $\delta = \phi'$ seems to be reasonable.

To be conservative in analysis, we usually assume

$$\delta = (0.5\sim0.67)\ \phi'$$

$$F_p = 1.2\sim1.3$$

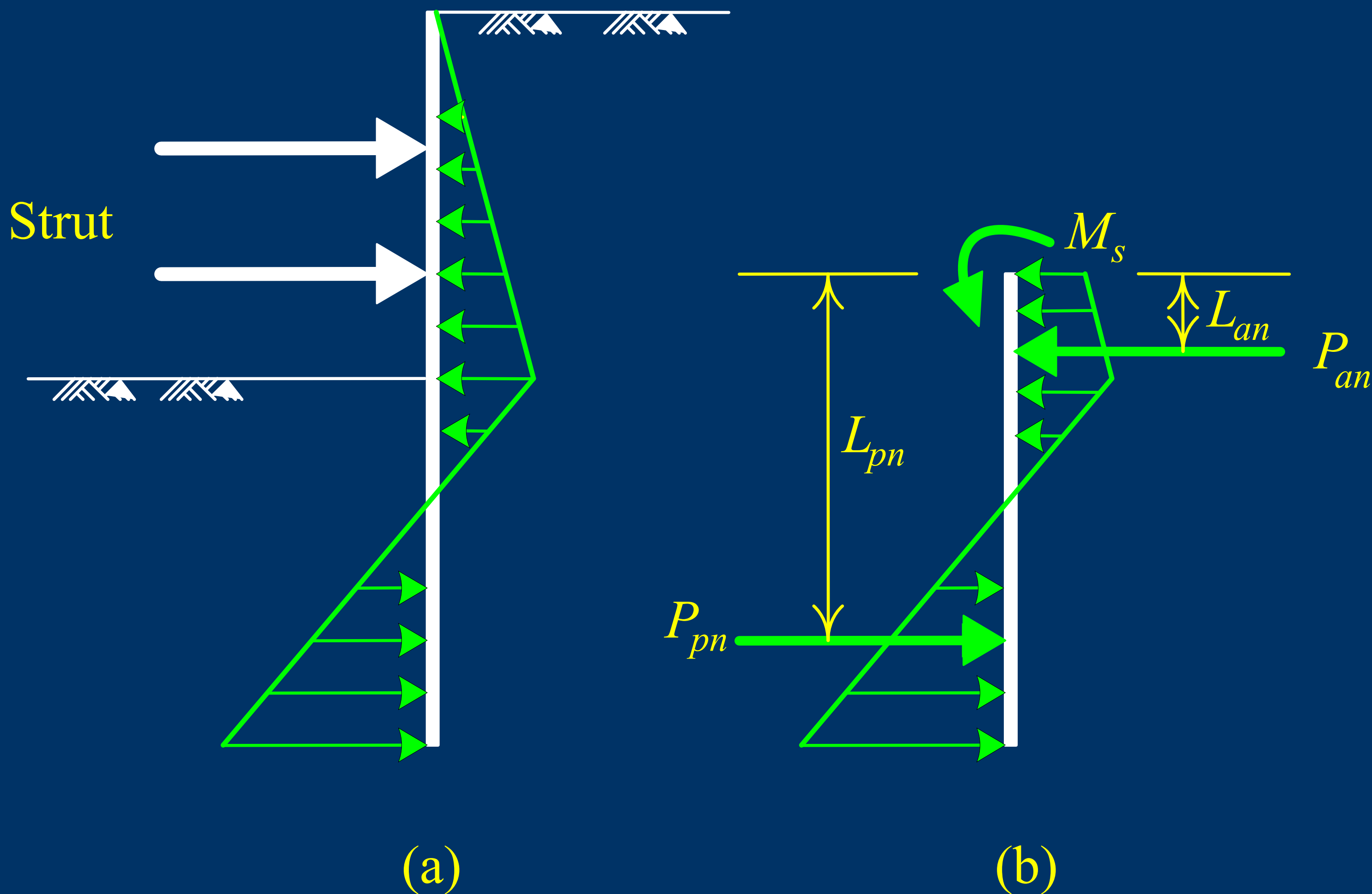


FIGURE 5.9 Analysis of- push in by the net pressure method
(a) distribution of net earth pressure
(b) force equilibrium of the retaining wall as a free body

5.2.2 Basal heave

The analyses of the basal heave failure are only applicable to clayey soils.

Like Terzaghi, Bjerrum and Eide,

Tschebotarioff, Terzaghi and Peck,

Clough and O'Rourke, etc.

But the most commonly applied of which are Terzaghi's method, Bjerrum and Eide, and the slip circle method.

(1) Bearing capacity method

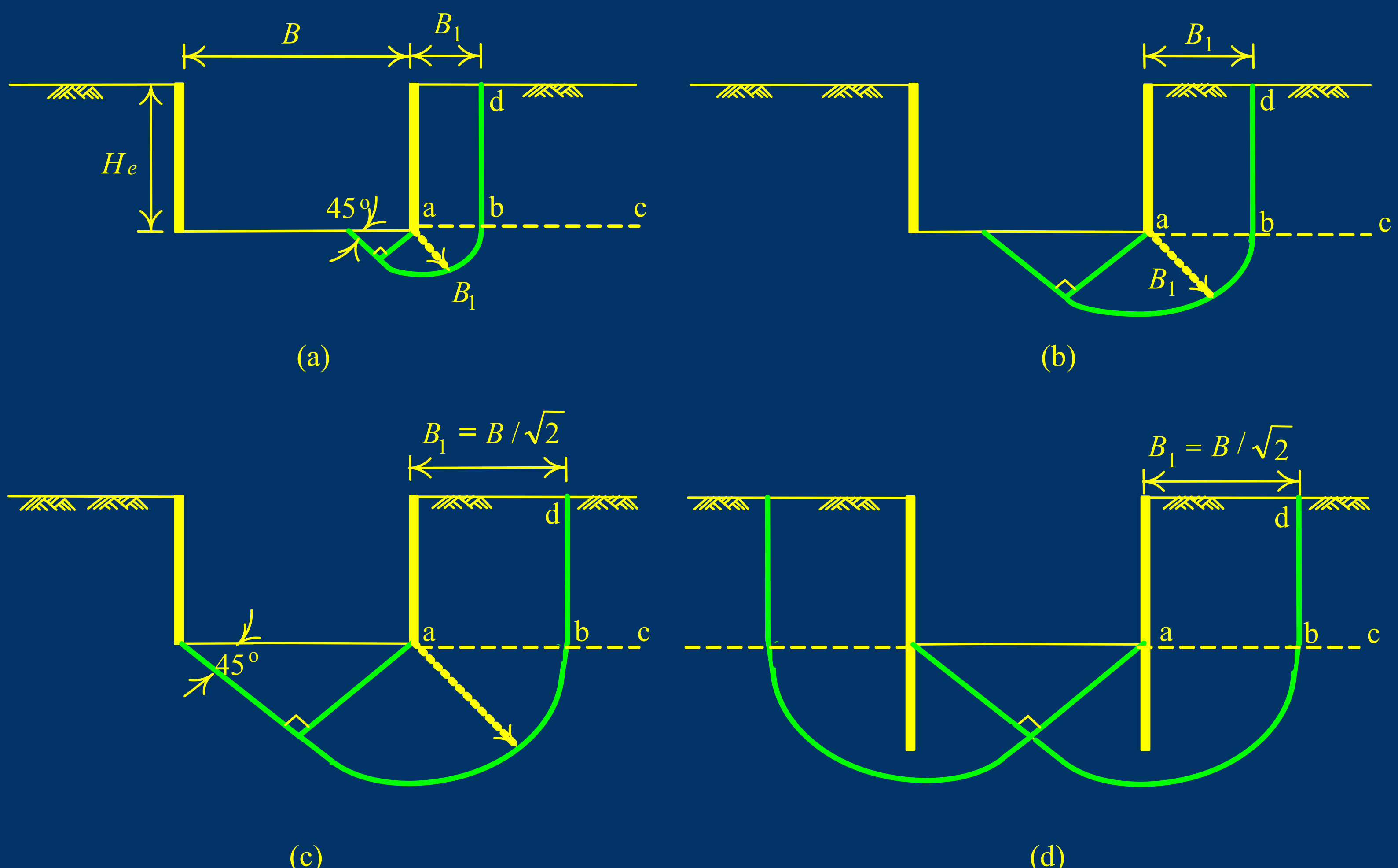


FIGURE 5.10 Analysis of push-in by bearing capacity method

(a) a B_1 wide trial failure surface

(b) a second B_1 wide trial failure surface

(c) a third B_1 wide trial failure surface

(d) both sides of the excavation produce failure surfaces

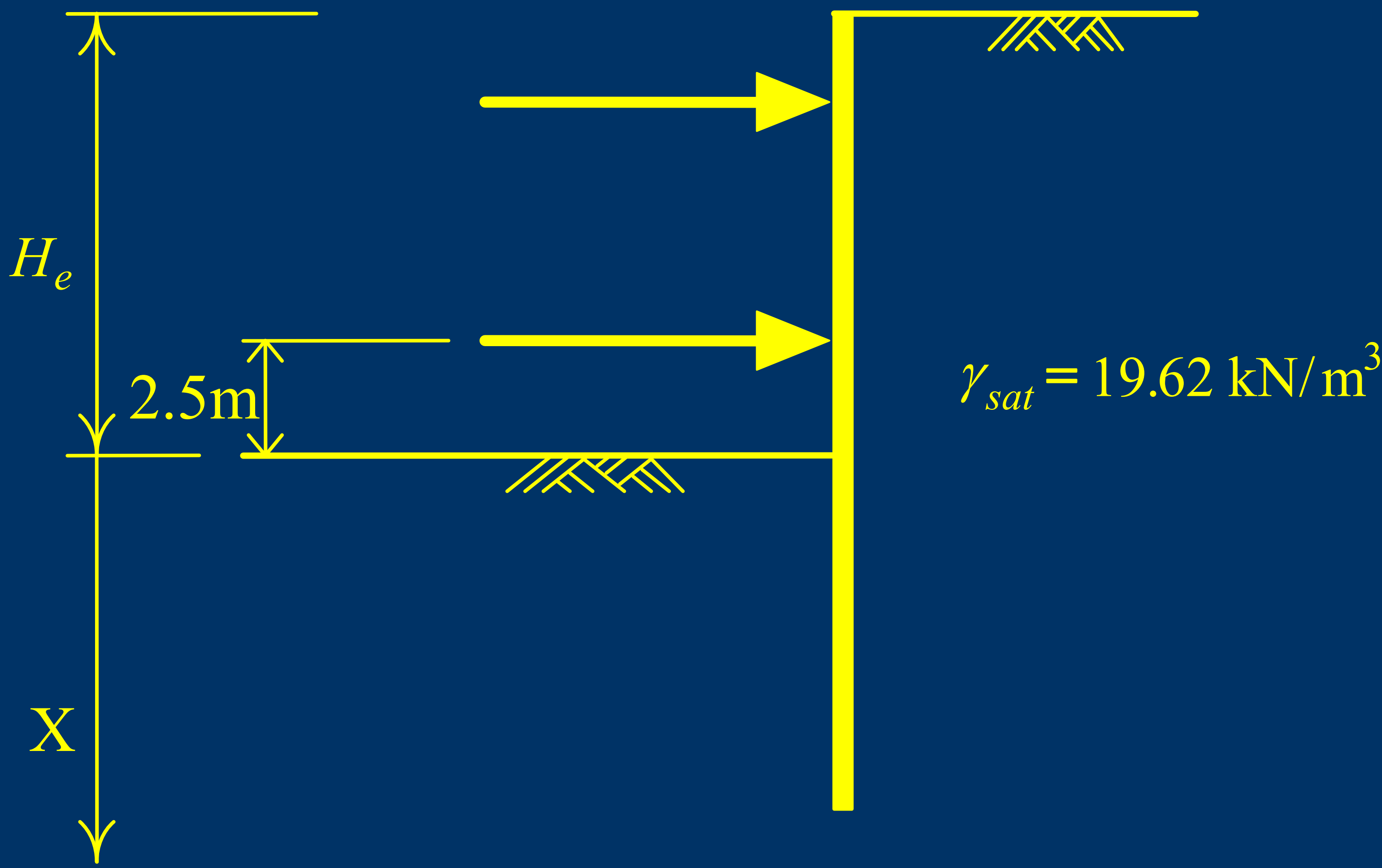


FIGURE 5.11 Excavation profile of the assumed excavation case

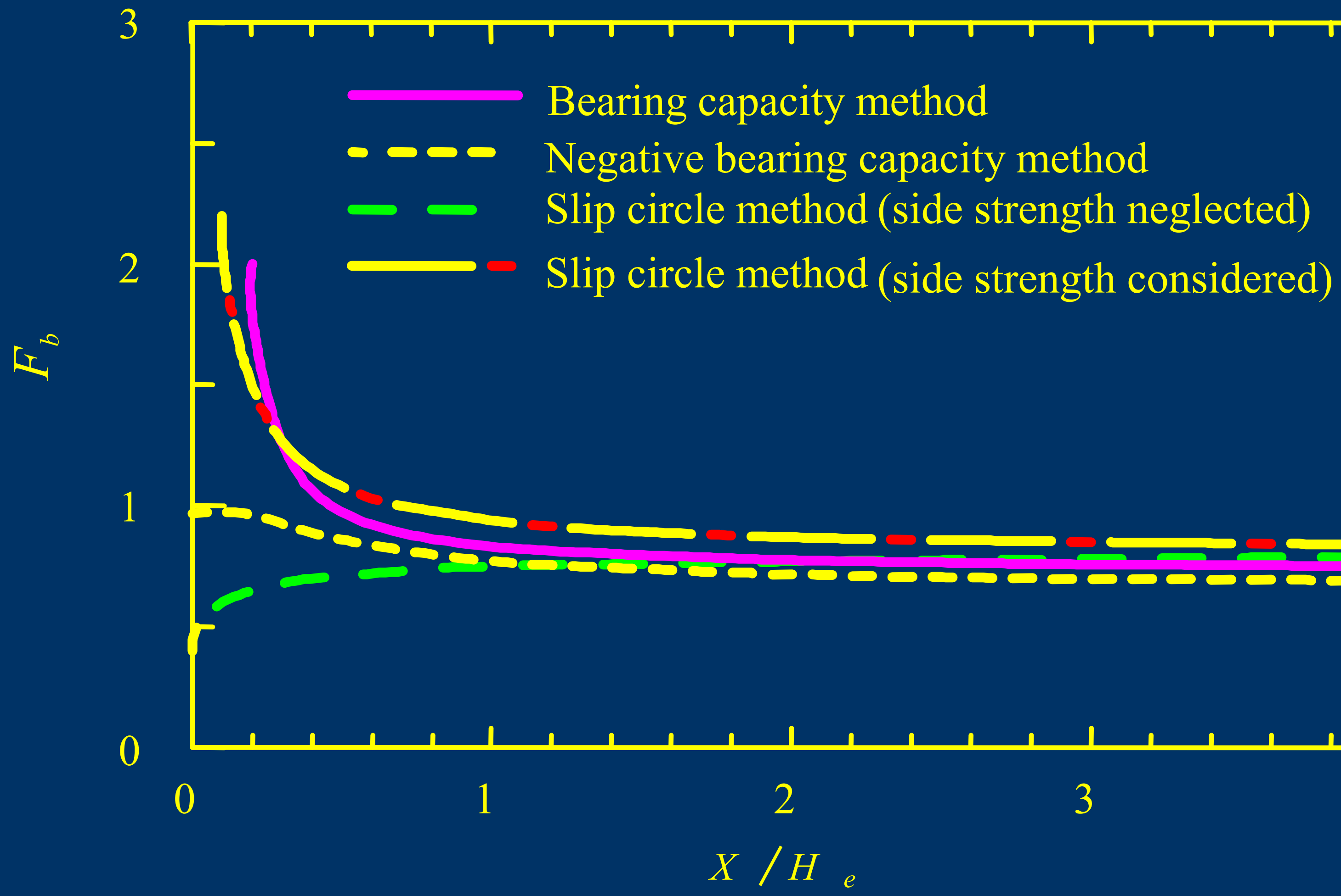


FIGURE 5.12 Relations between failure circle sizes and factors of safety against basal heave obtained by the bearing capacity method, negative bearing capacity method, and the slip circle method

$(S_u = 25 KN / m^2)$

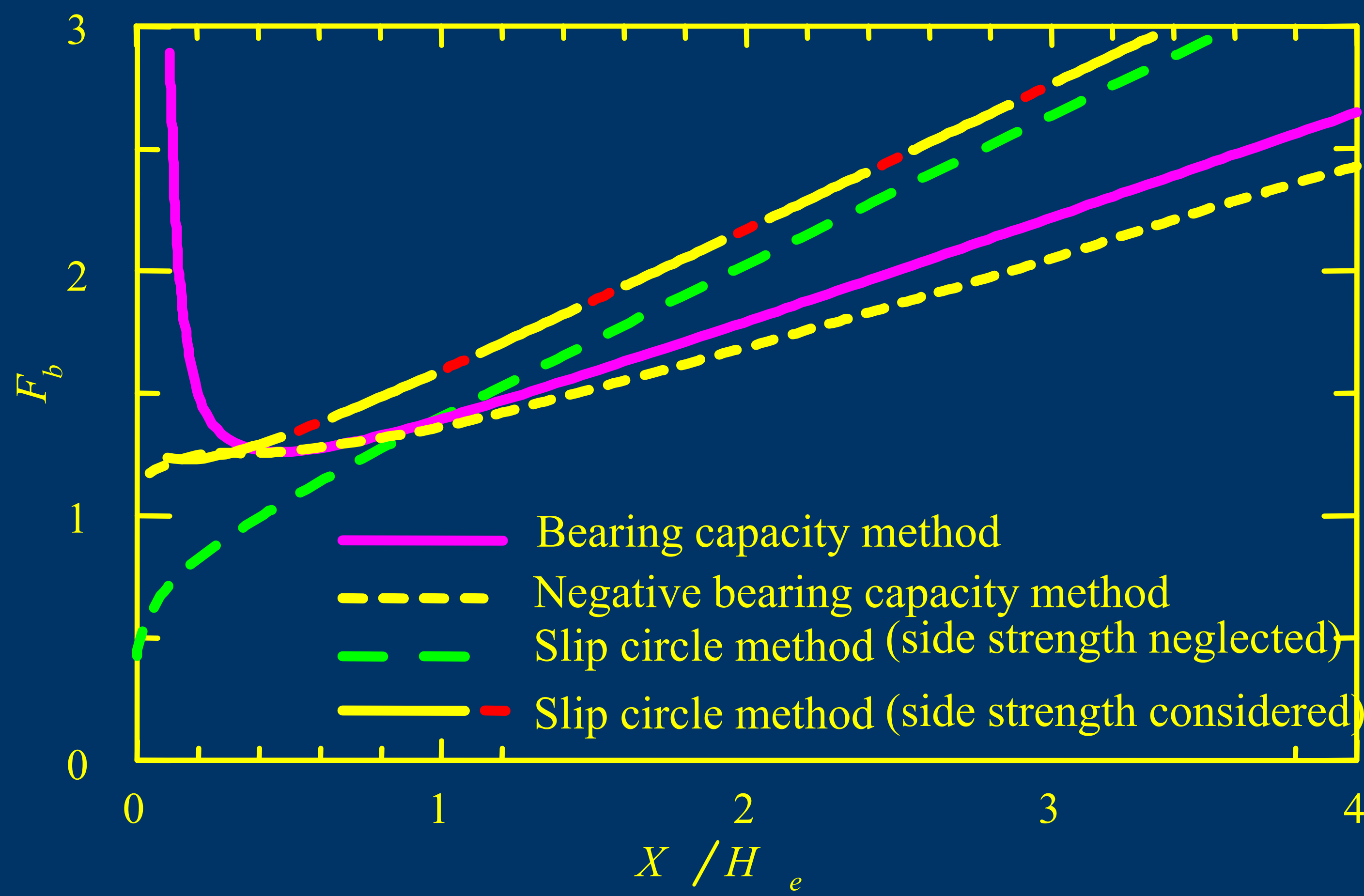


FIGURE 5.13 Relations between failure circle sizes and factors of safety against basal heave obtained by the bearing capacity method, negative bearing capacity method, and the slip circle method

$(S_u / \sigma_v' = 0.3)$

Terzaghi's method

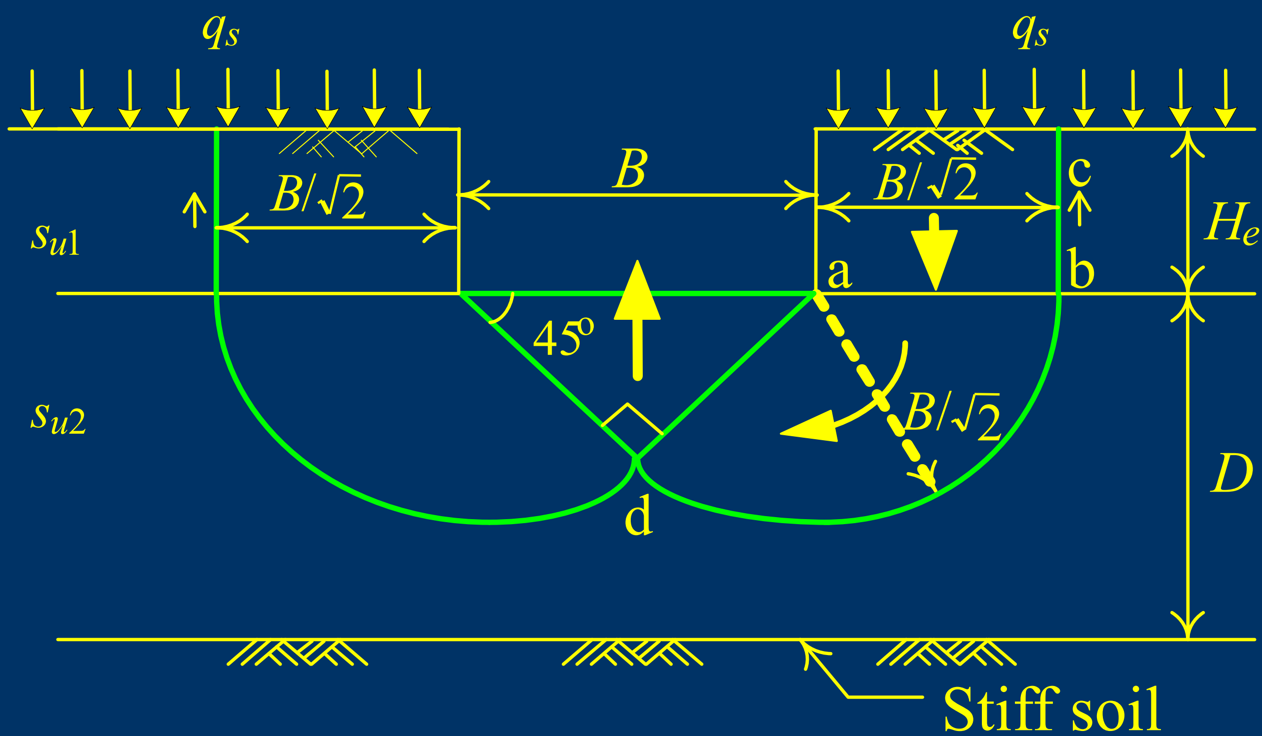


FIGURE 5.14 Analysis of basal heave using Terzaghi's method

$(a) D \geq B/\sqrt{2}$

When $D \geq B/\sqrt{2}$, the formation of a failure surface is not restrained by the stiff soil.

$$W = (\gamma H_e + q_s)(B_1 \times 1) = (\gamma H_e + q_s)\frac{B}{\sqrt{2}} \tag{5.7}$$

$$Q_u = 5.7 s_{u2}(B_1 \times 1) = (5.7 s_{u2})\frac{B}{\sqrt{2}} \tag{5.8}$$

Vertical plane **bc** can offer shear resistance $s_{u1} H_e$ and the factor of safety against basal heave will be：

$$F_b = \frac{Q_u}{W - s_{u1}H_e} = \frac{5.7 s_{u2} B/\sqrt{2}}{(\gamma H_e + q_s)B/\sqrt{2} - s_{u1}H_e} = \frac{1}{H_e} \times \frac{5.7 s_{u2}}{\gamma + \frac{q_s}{H_e} - \frac{s_{u1}}{0.7B}} \tag{5.9}$$

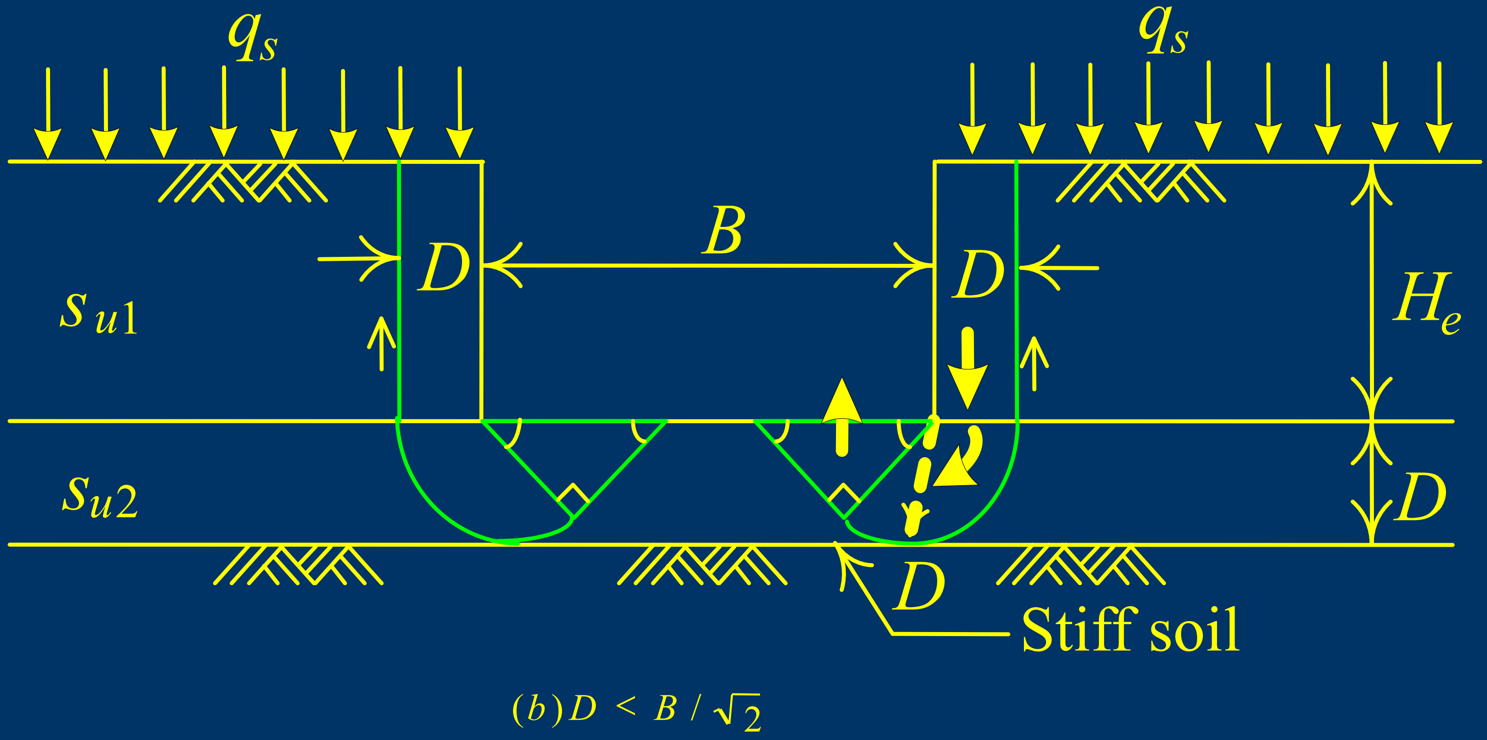


FIGURE 5.14 Analysis of basal heave using Terzaghi's method

$(b)\ D < B/\sqrt{2}$

When $D < B/\sqrt{2}$, the failure surface will be restrained by the stiff soil.

$$F_b = \frac{Q_u}{W - s_{u1}H_e} = \frac{5.7 s_{u2} D}{(\gamma H_e + q_s)D - s_{u1}H_e} = \frac{1}{H_e} \times \frac{5.7 s_{u2}}{\gamma + \frac{q_s}{H_e} - \frac{s_{u1}}{D}} \qquad (5.10)$$

Clough suggested that, Terzaghi's factor of safety (F_b) should be greater than or equal to 1.5.

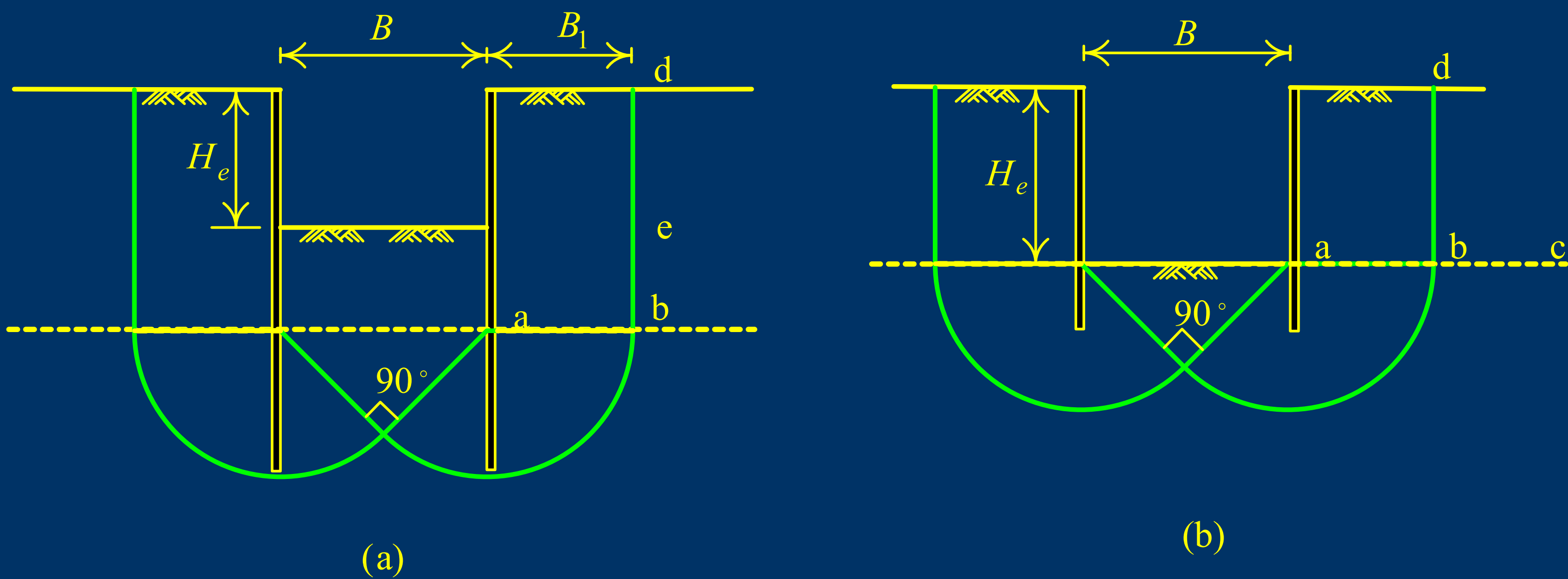


FIGURE 5.15 Relation between the embedded part of the retaining wall and the failure surface
(a) large penetration depth (b) small penetration depth

(2) Negative bearing capacity method

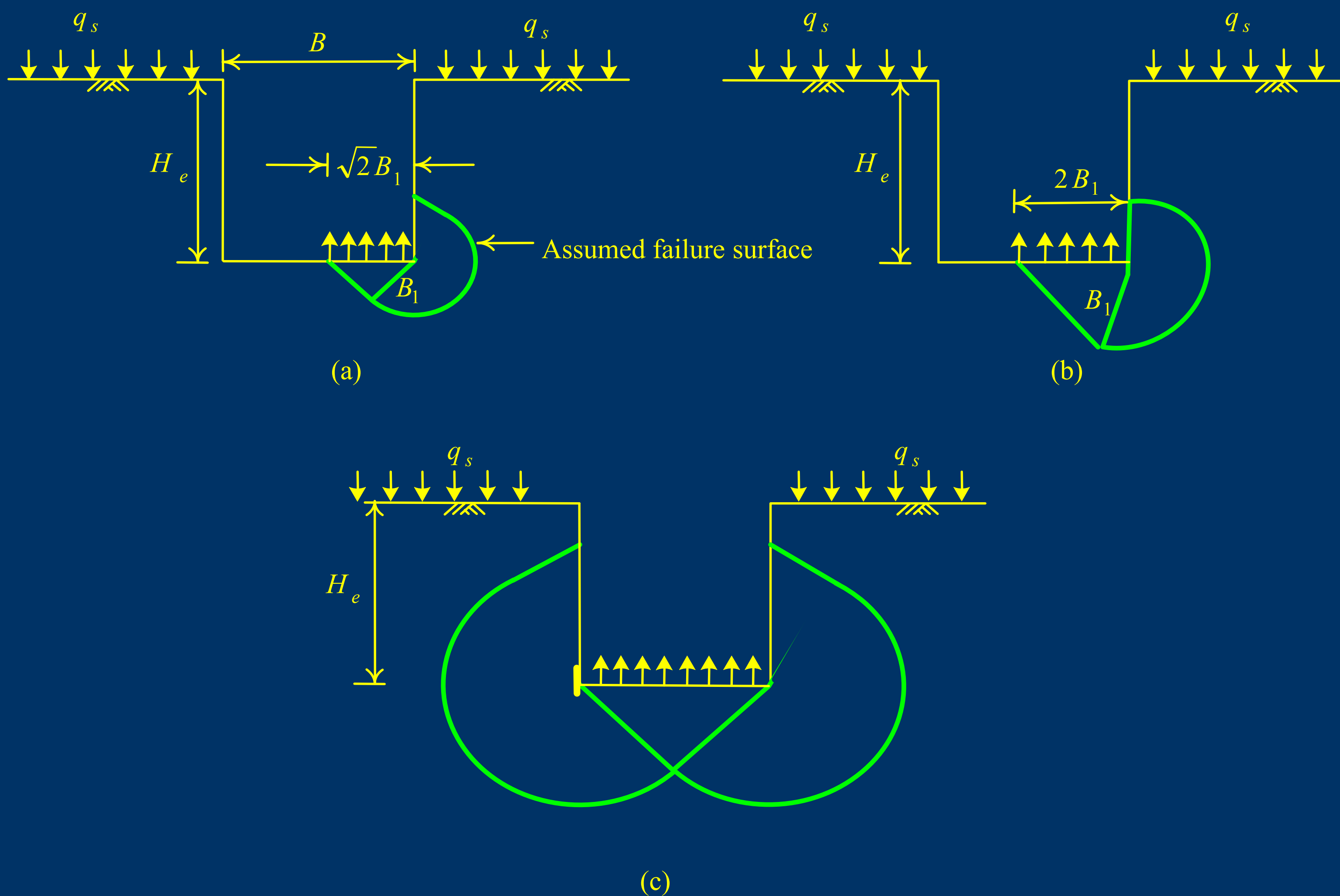


FIGURE 5.16 Analysis of basal heave by negative bearing capacity method (a) a $\sqrt{2}B_1$ wide failure surface (b) another $\sqrt{2}B_1$ wide failure surface (c) Failure surface covers the whole excavation bottom

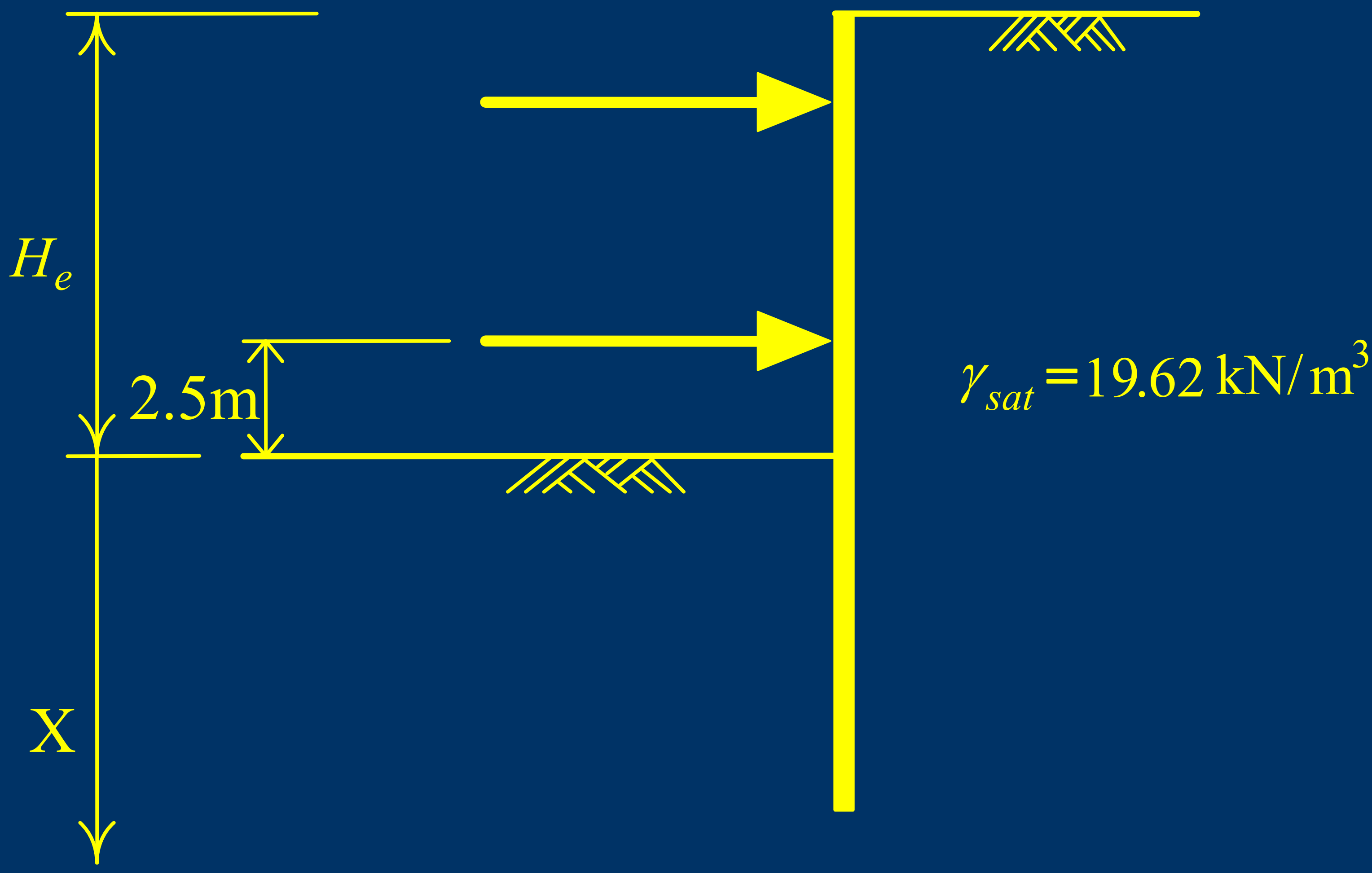


FIGURE 5.11 Excavation profile of the assumed excavation case

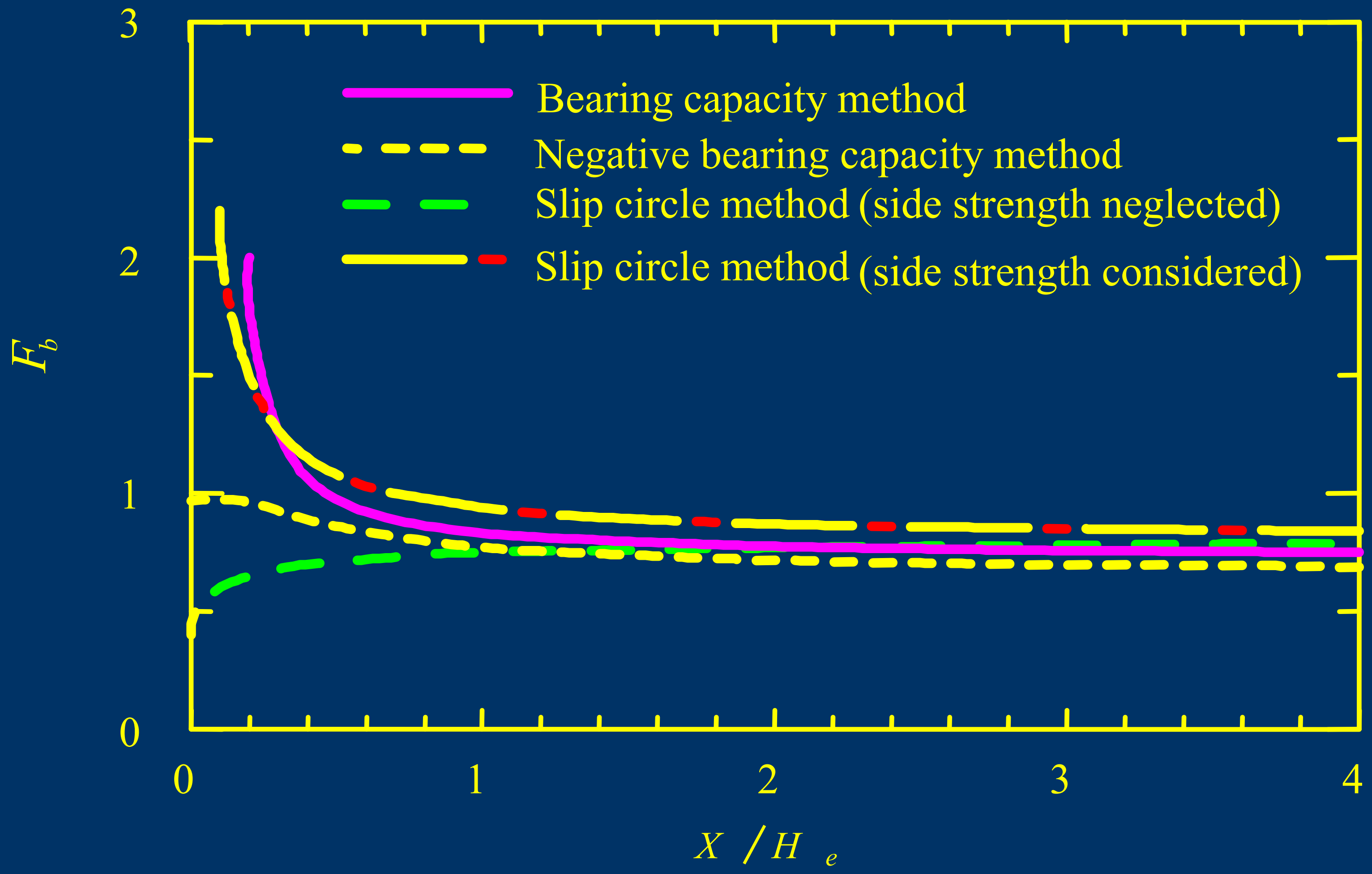


FIGURE 5.12 Relations between failure circle sizes and factors of safety against basal heave obtained by the bearing capacity method, negative bearing capacity method, and the slip circle method
($S_u = 25 KN/m^2$)

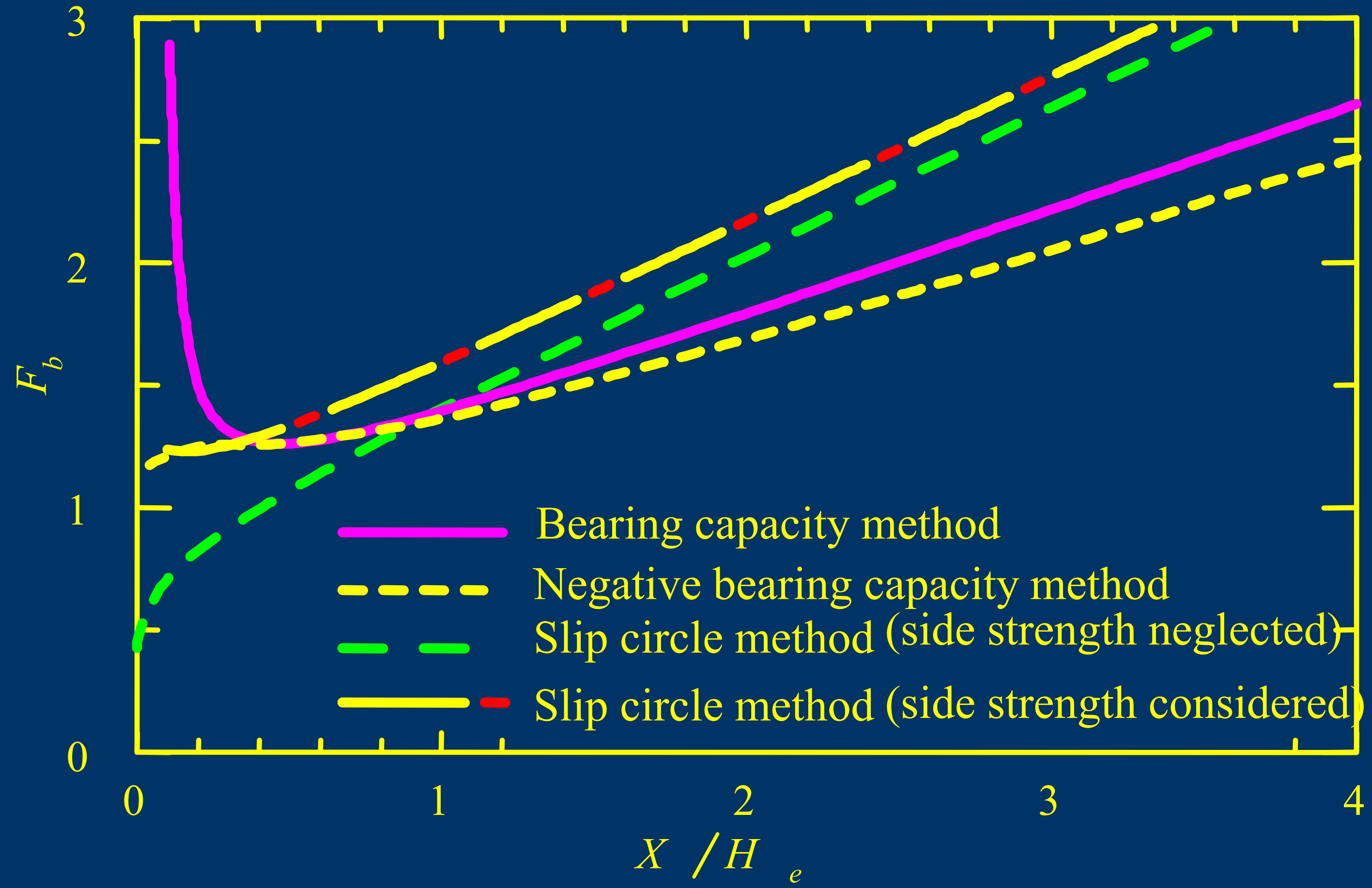


FIGURE 5.13 Relations between failure circle sizes and factors of safety against basal heave obtained by the bearing capacity method, negative bearing capacity method, and the slip circle method $(S_u / \sigma_v' = 0.3)$

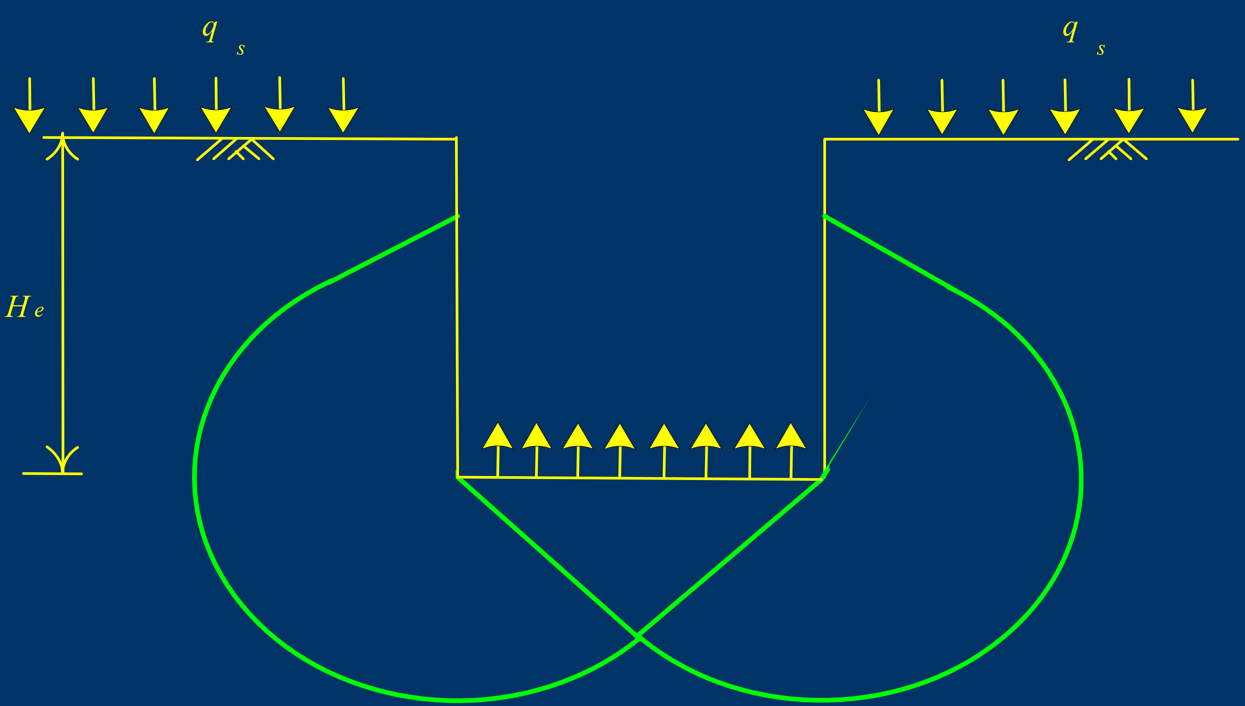


FIGURE 5.16c

Bjerrum and Eide's method

$$F_b = \frac{N_c \times s_u}{\gamma \times H_e + q_s} \qquad (5.12)$$

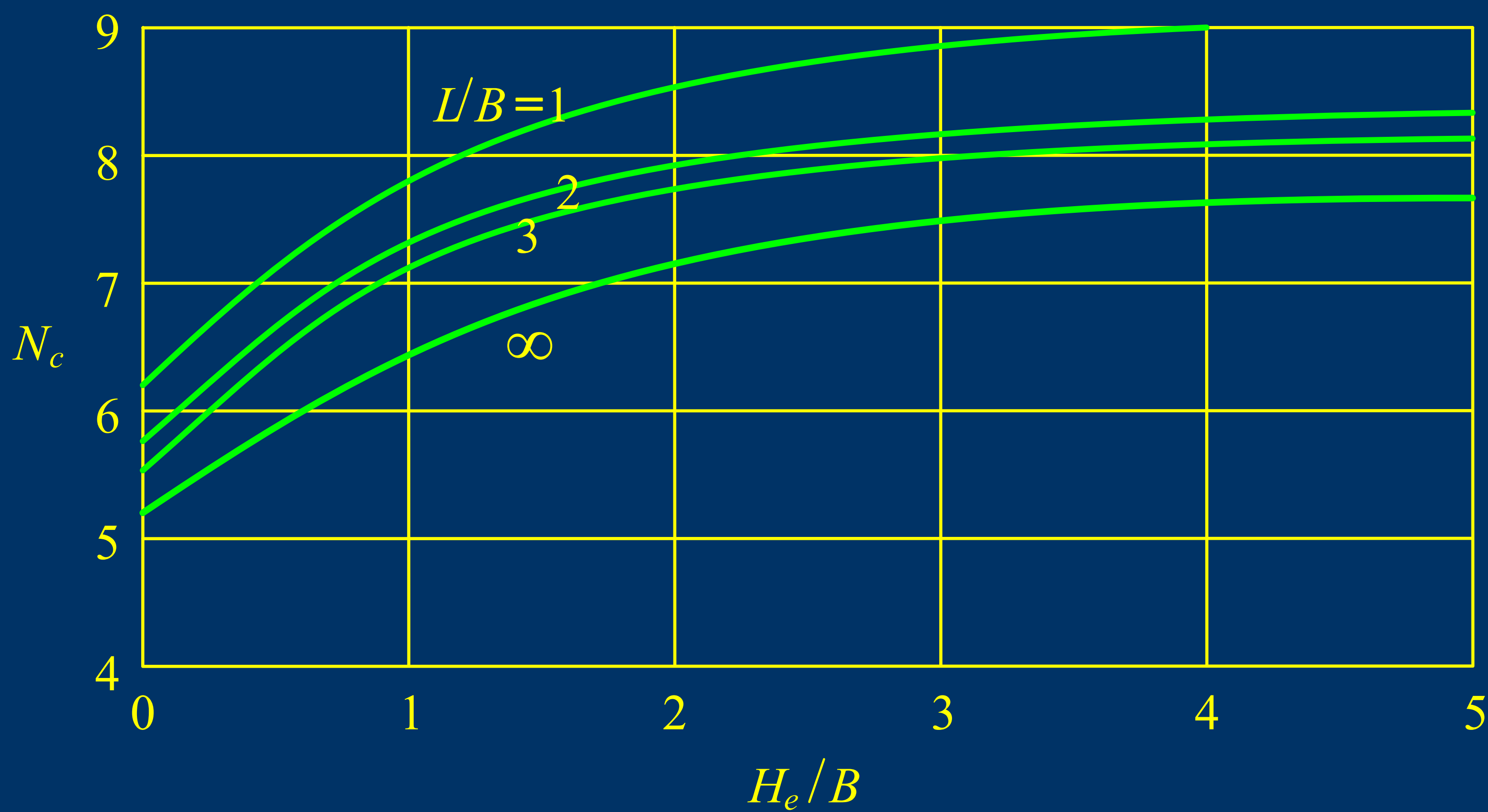


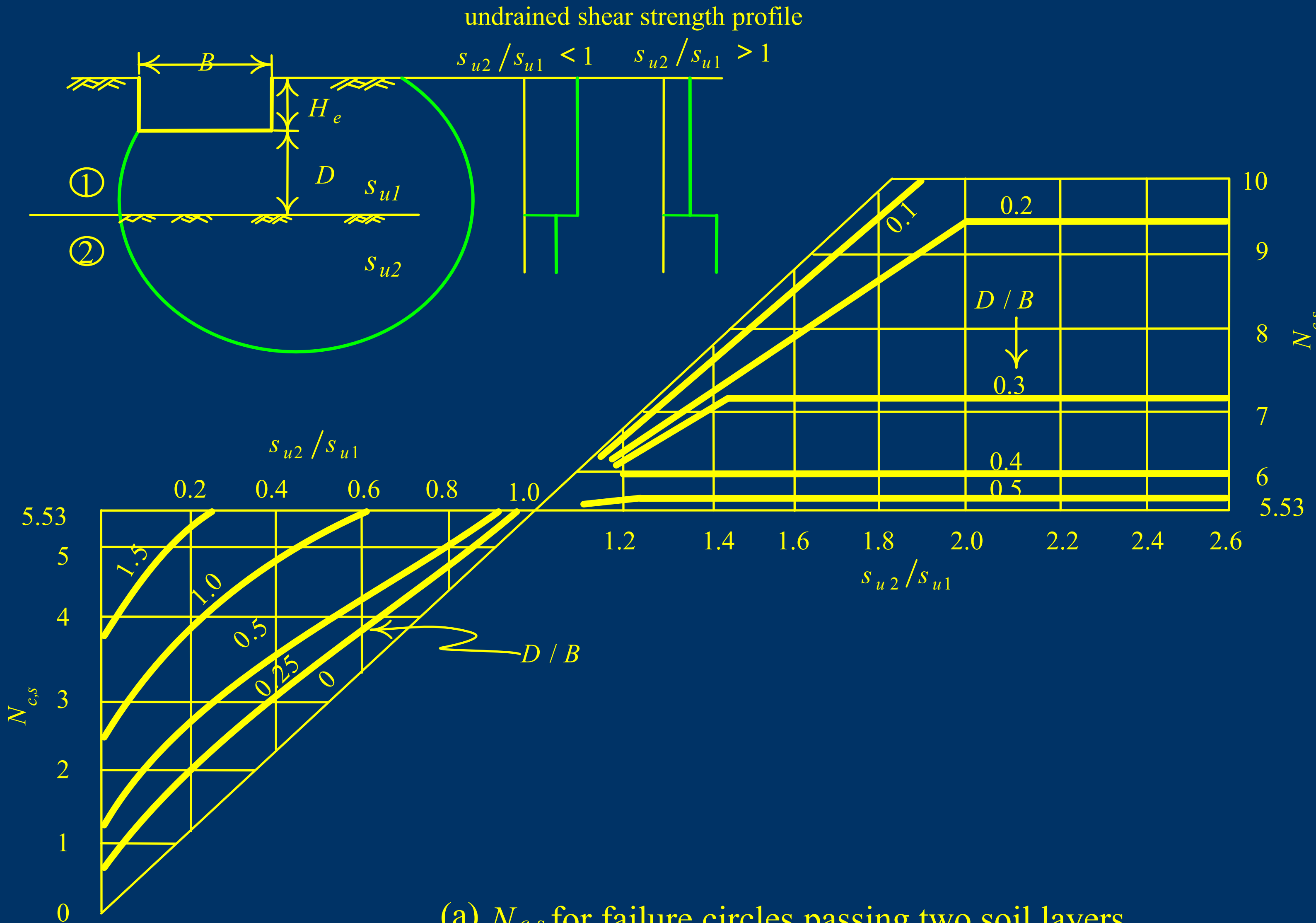
FIGURE 5.17 Skempton's bearing capacity factor

$$N_{c\,(\text{rectangular})} = N_{c\,(\text{square})} \quad (0.84 + 0.16\frac{B}{L}) \qquad (5.11)$$

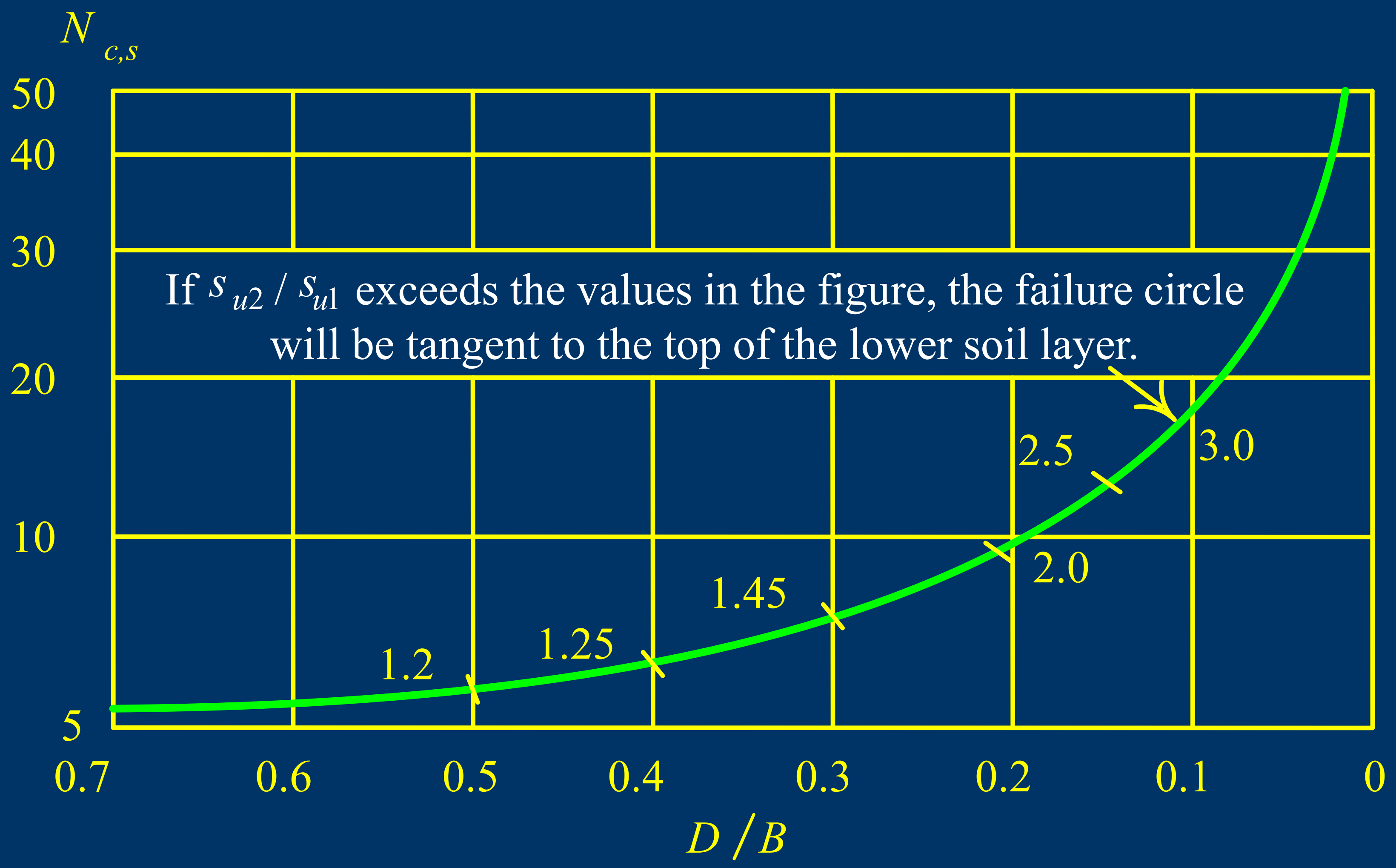
Modified Bjerrum and Eide's method：

$$F_b = \frac{s_{u1} N_{c,s} f_d f_s}{\gamma H_e} \qquad (5.13)$$

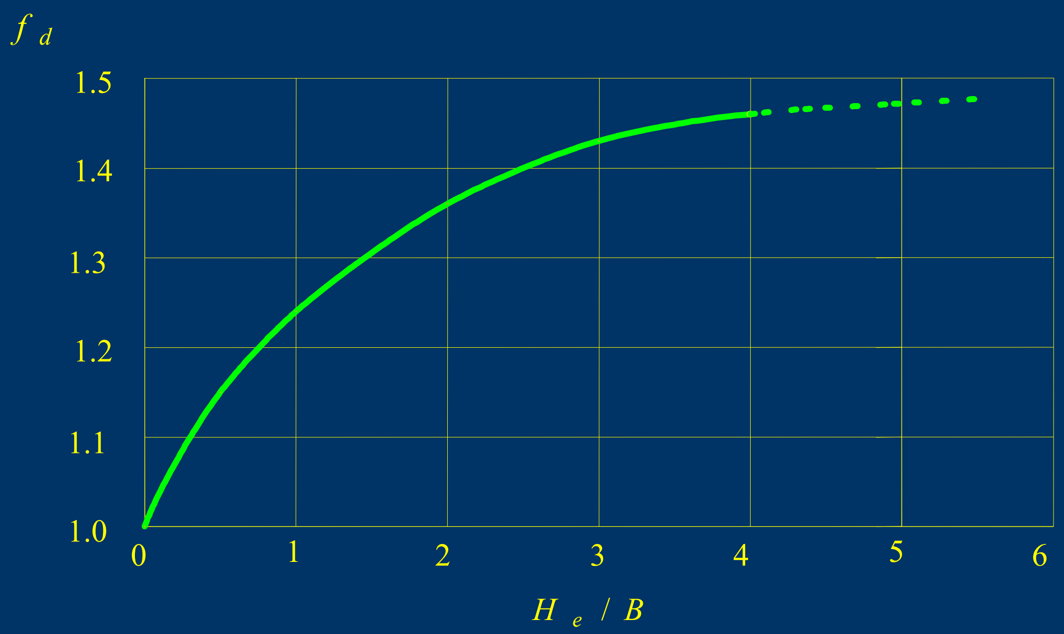
$$f_s = 1 + 0.2\frac{B}{L} \qquad (5.14)$$



(a) $N_{c,s}$ for failure circles passing two soil layers



(b) $N_{c,s}$ for failure circles tangent to the top of the lower soil layer



(c)

FIGURE 5.18 Extended Bjerrum and Eide's method

(a) $N_{c,s}$ for failure circles passing two soil layers

(b) $N_{c,s}$ for failure circles tangent to the top of the lower soil layer

(c) width modification factor f_d

DM7.2 suggested that Bjerrum and Eide's factor of safety F_b should be greater than or equal to 1.5.

(3) Slip circle method

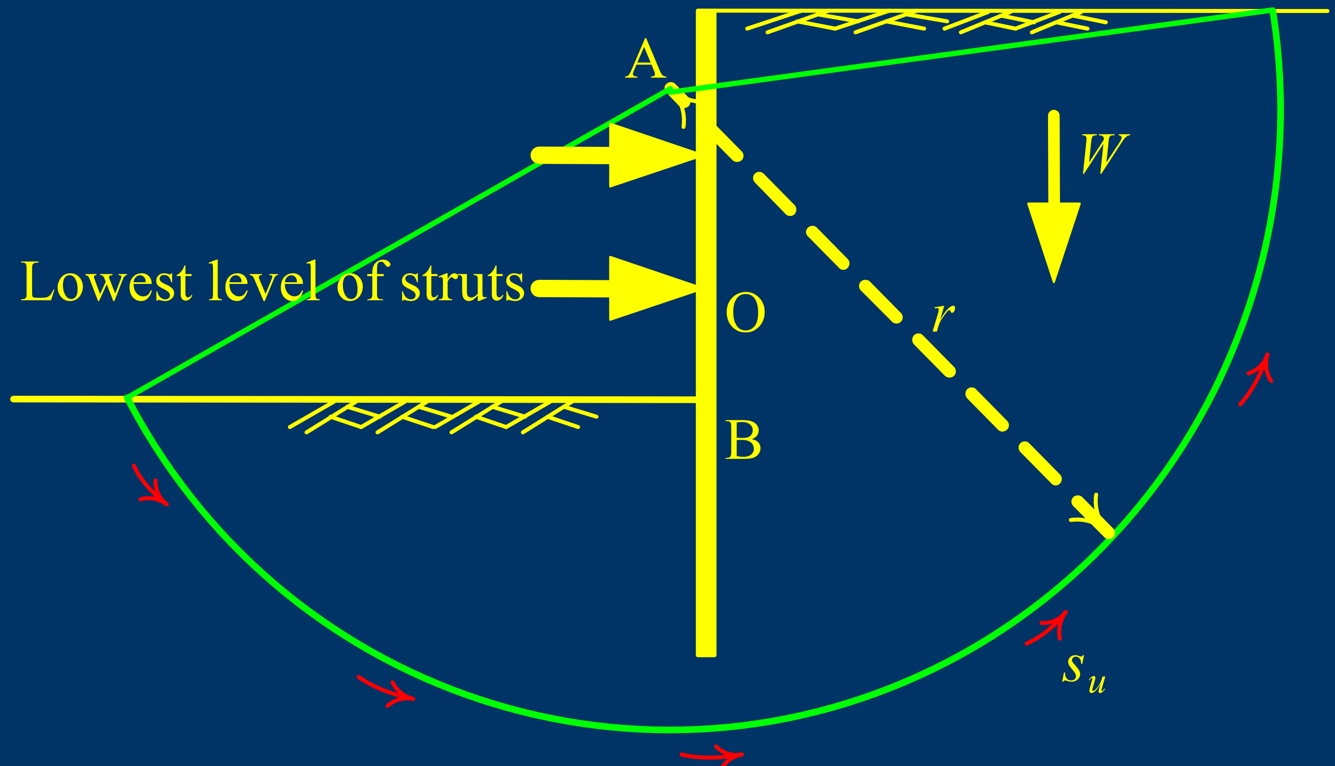


FIGURE 5.19 Location of the center of a failure circle for the slip circle method

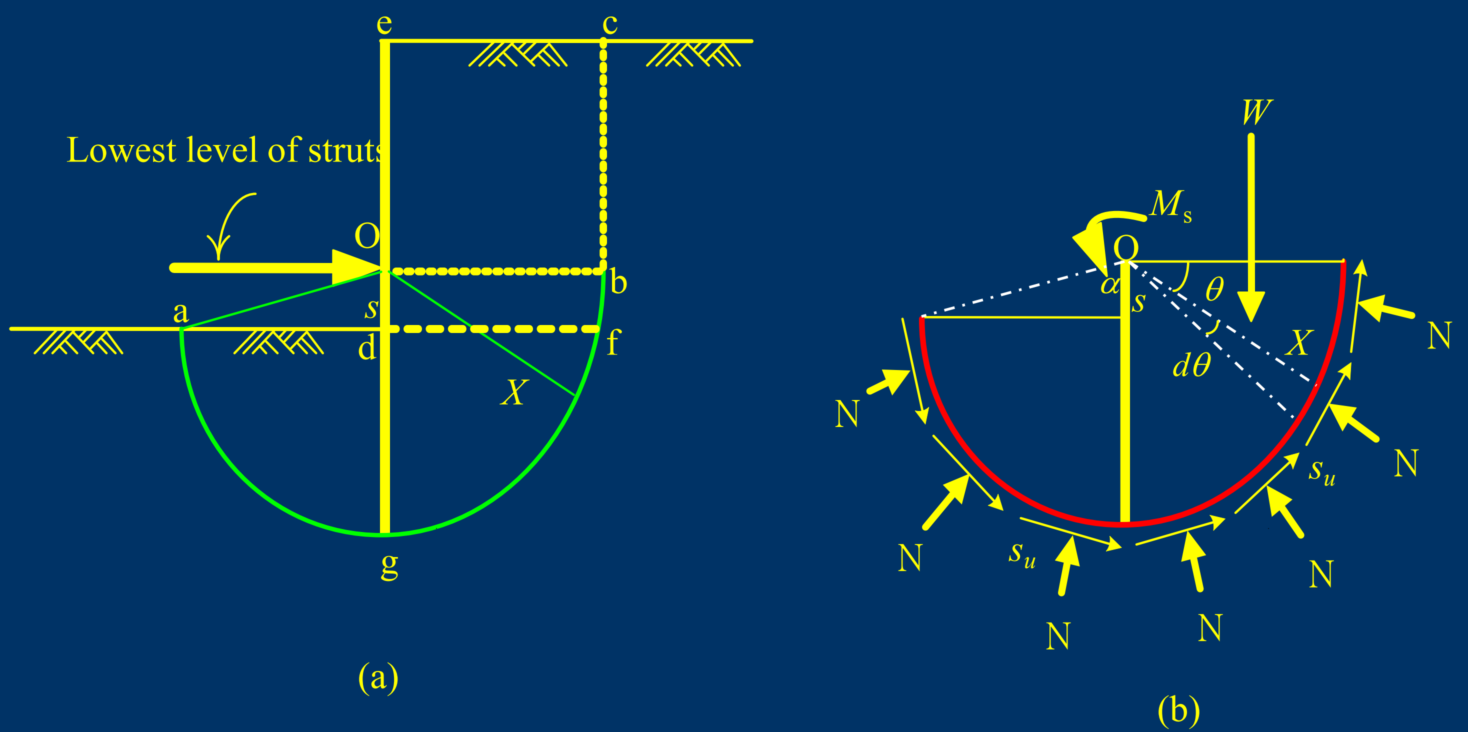


FIGURE 5.20 Analysis of basal heave by the slip circle method
(a) the failure surface (b) balance of the a free body

$$F_b = \frac{M_r}{M_d} = \frac{X \int_0^{\frac{\pi}{2}+\alpha} s_u \, (X \, d\theta) + M_s}{W \times \frac{X}{2}} \qquad (5.15)$$

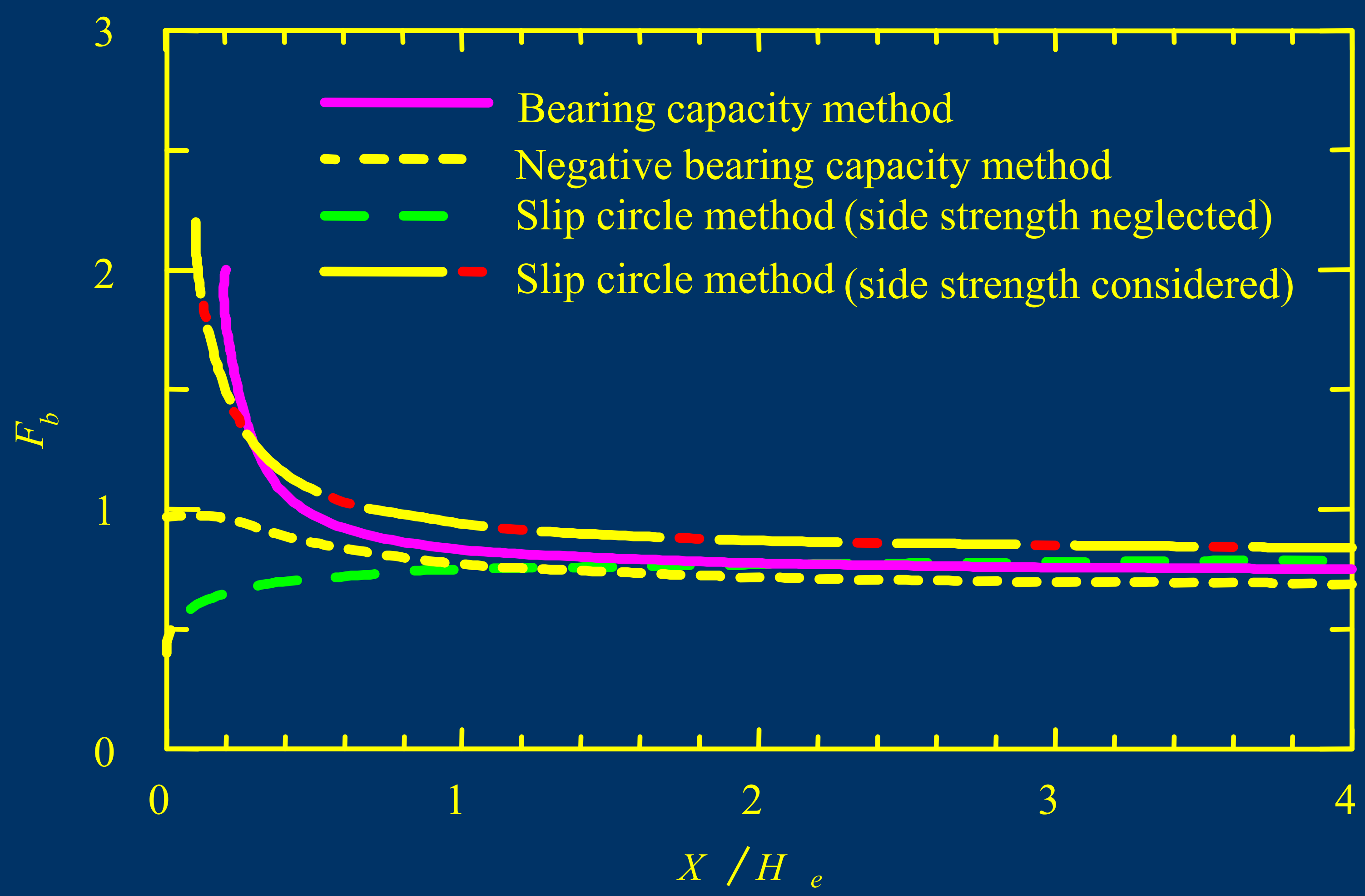


FIGURE 5.12 Relations between failure circle sizes and factors of safety against basal heave obtained by the bearing capacity method, negative bearing capacity method, and the slip circle method

$(S_u = 25KN/m^2)$

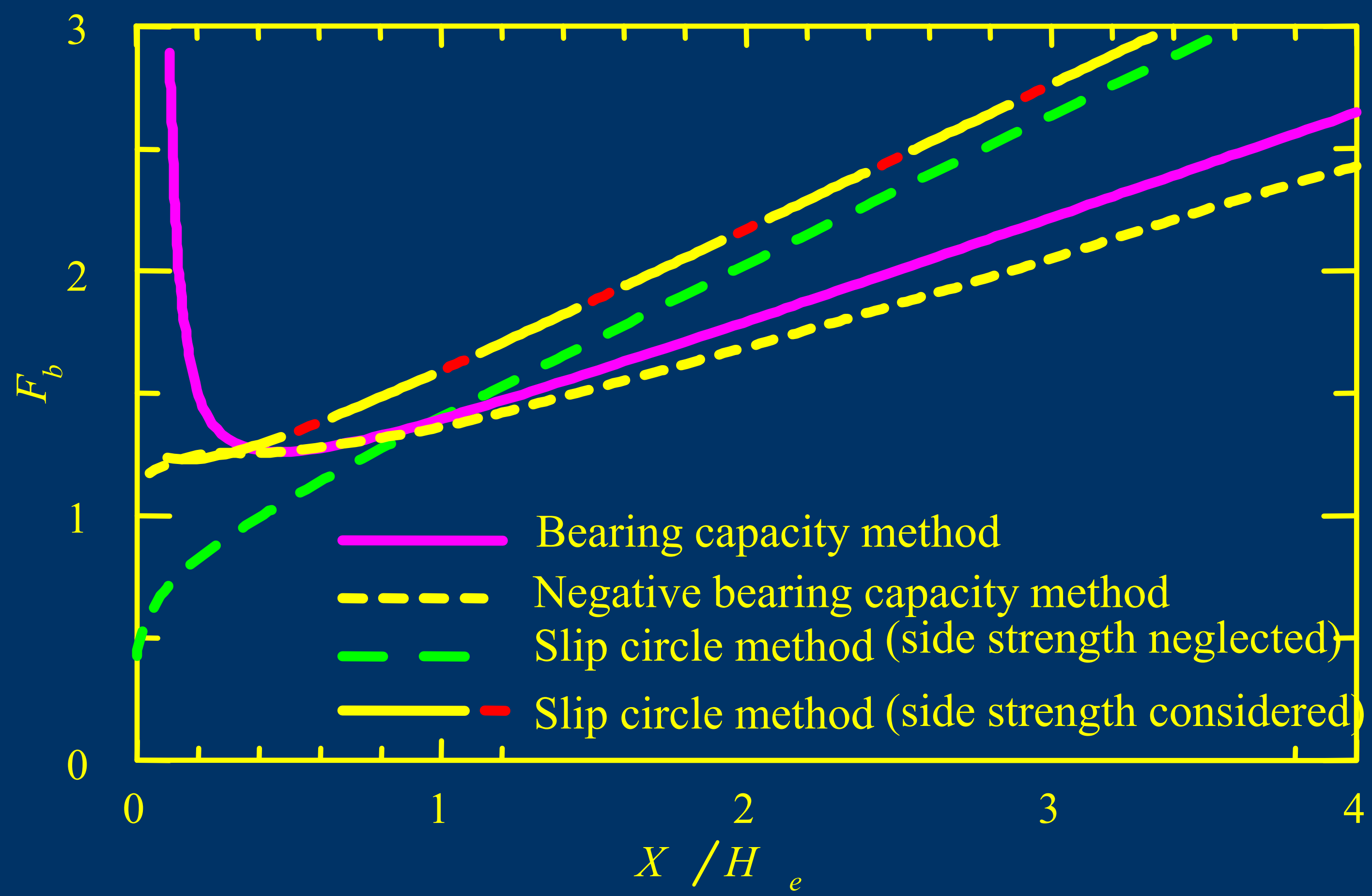


FIGURE 5.13 Relations between failure circle sizes and factors of safety against basal heave obtained by the bearing capacity method negative bearing capacity method, and the slip circle method

$(S_u / \sigma'_v = 0.3)$

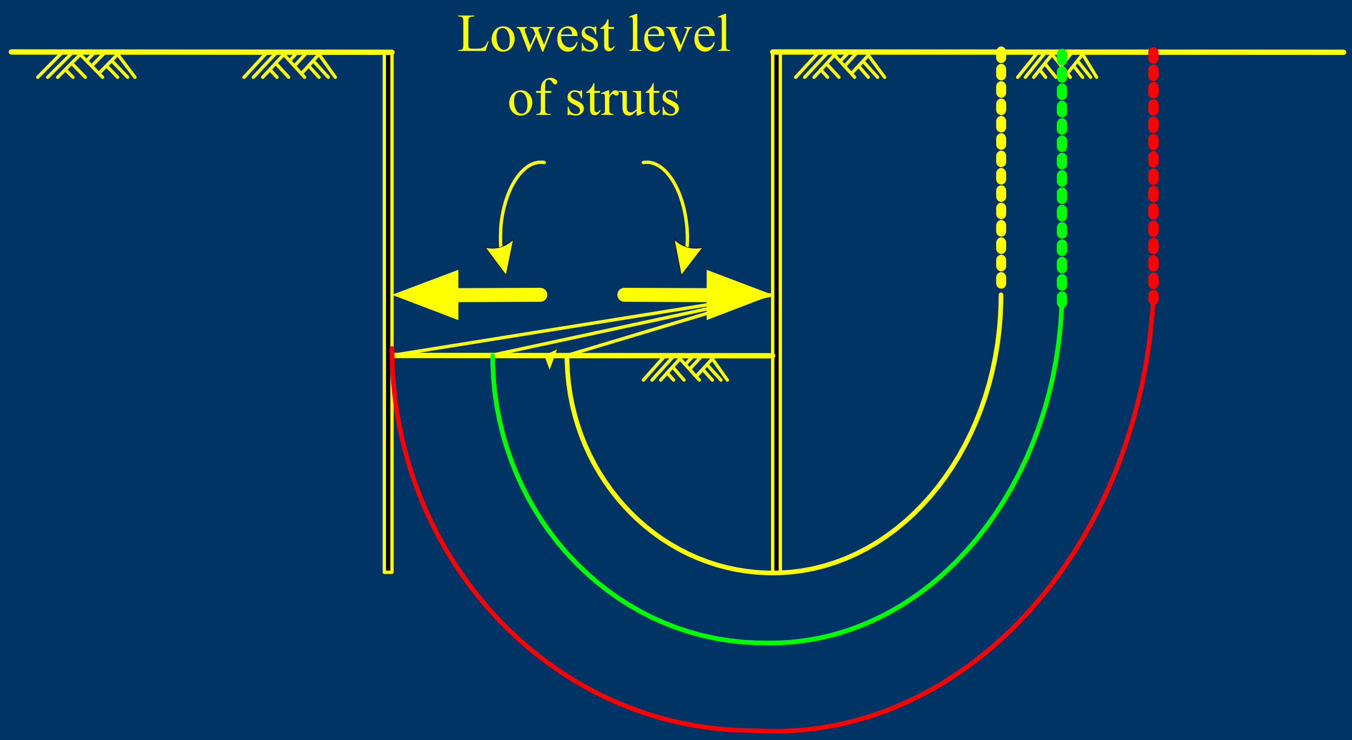


FIGURE 5.21 Factor of safety increasing due to the failure circle exceeding the excavation width

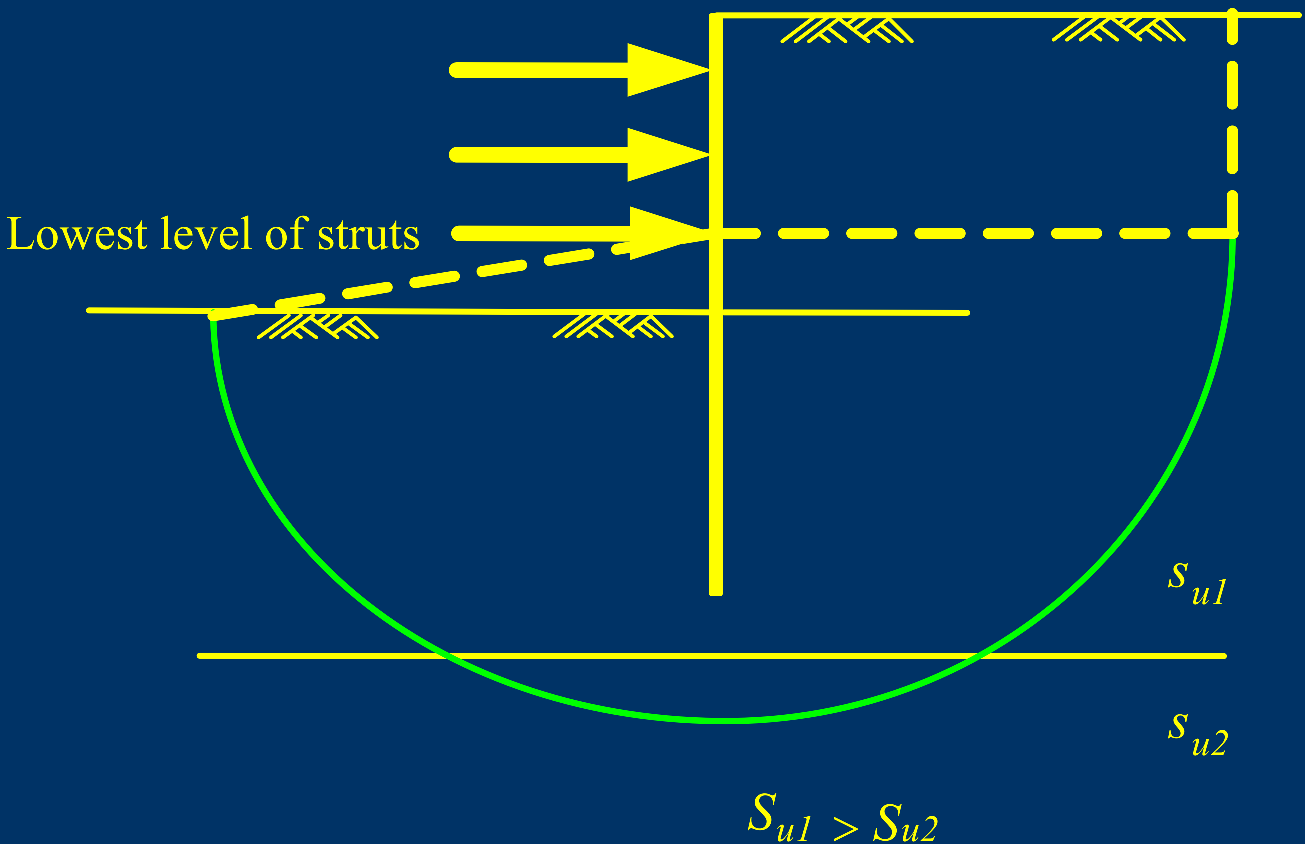


FIGURE 5.22 Analysis of basal heave in layered soft soils

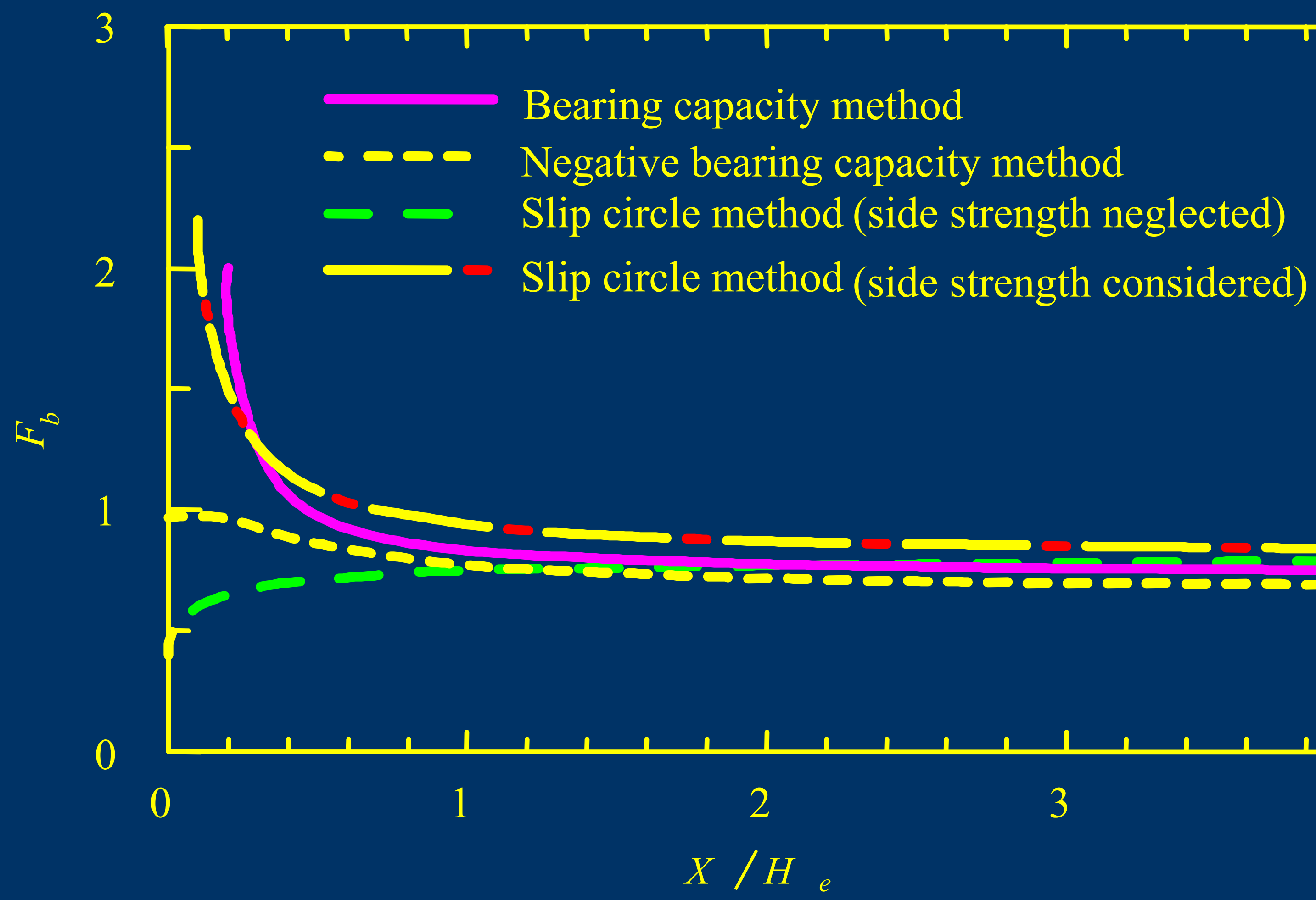


FIGURE 5.12 Relations between failure circle sizes and factors of safety against basal heave obtained by the bearing capacity method, negative bearing capacity method, and the slip circle method

$(S_u = 25KN/m^2)$

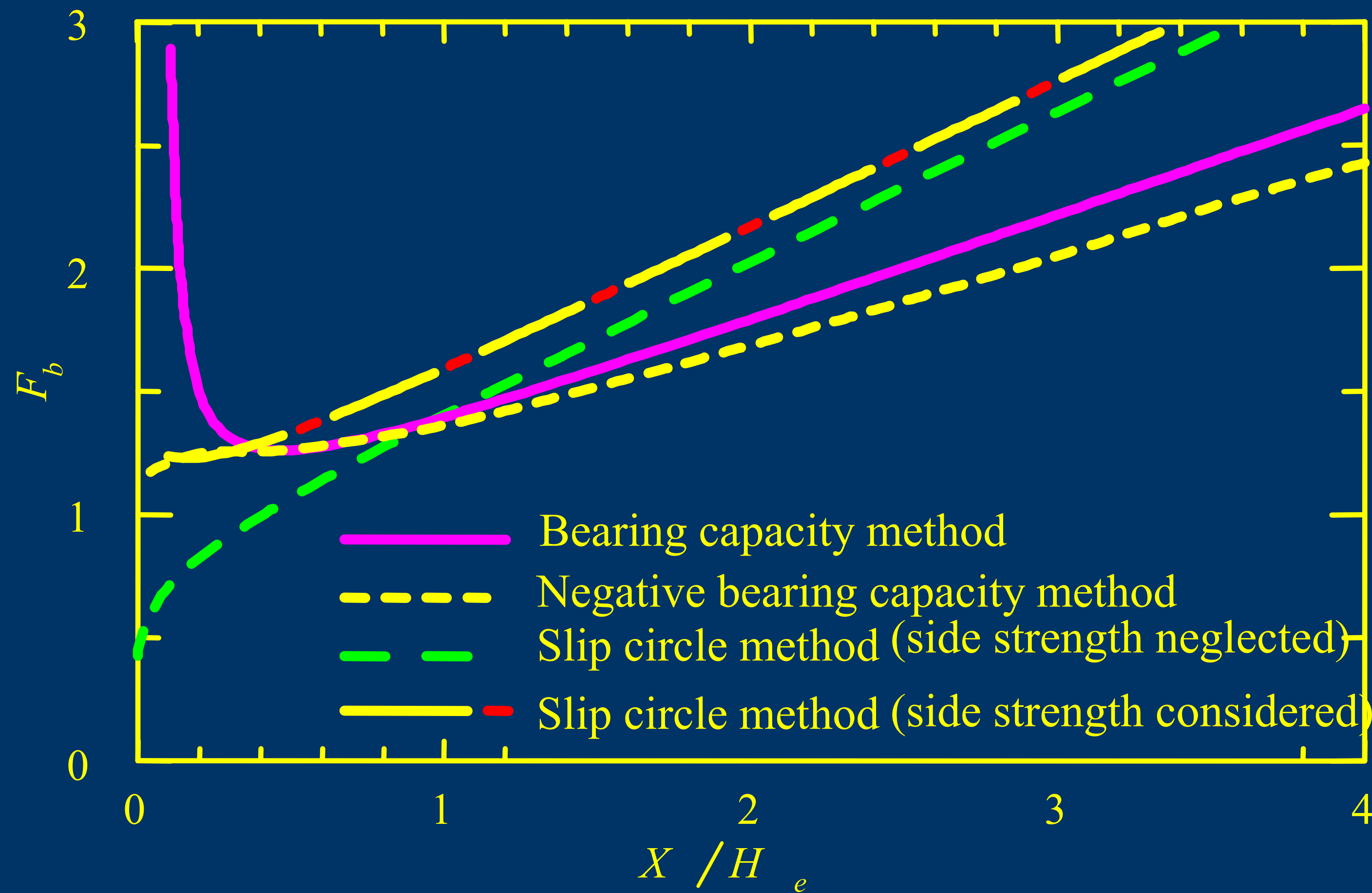
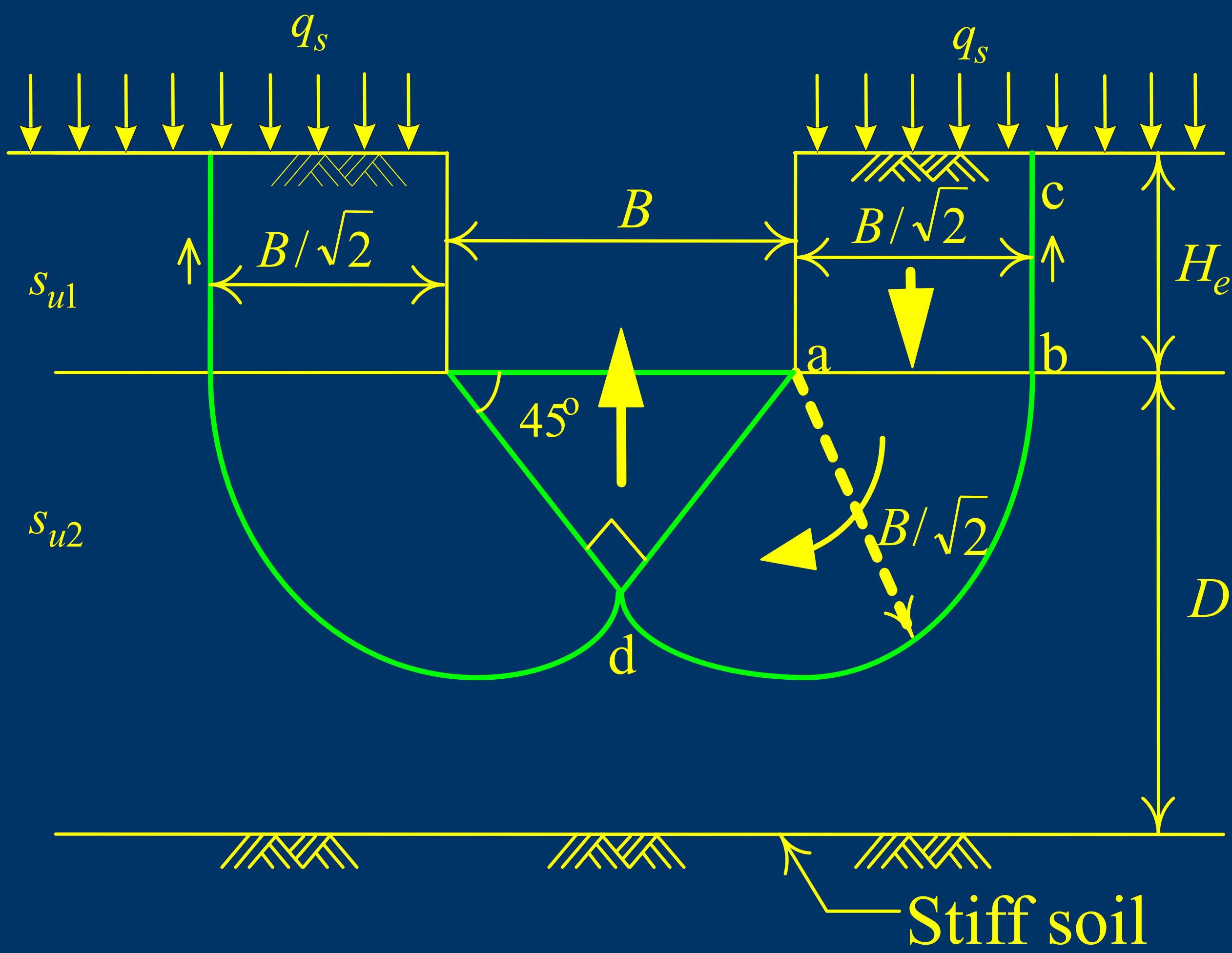


FIGURE 5.13 Relations between failure circle sizes and factors of safety against basal heave obtained by the bearing capacity method, negative bearing capacity method, and the slip circle method ($S_u/\sigma'_v = 0.3$)

(5) Applicability to sandy soils



$(a) D \geq B/\sqrt{2}$

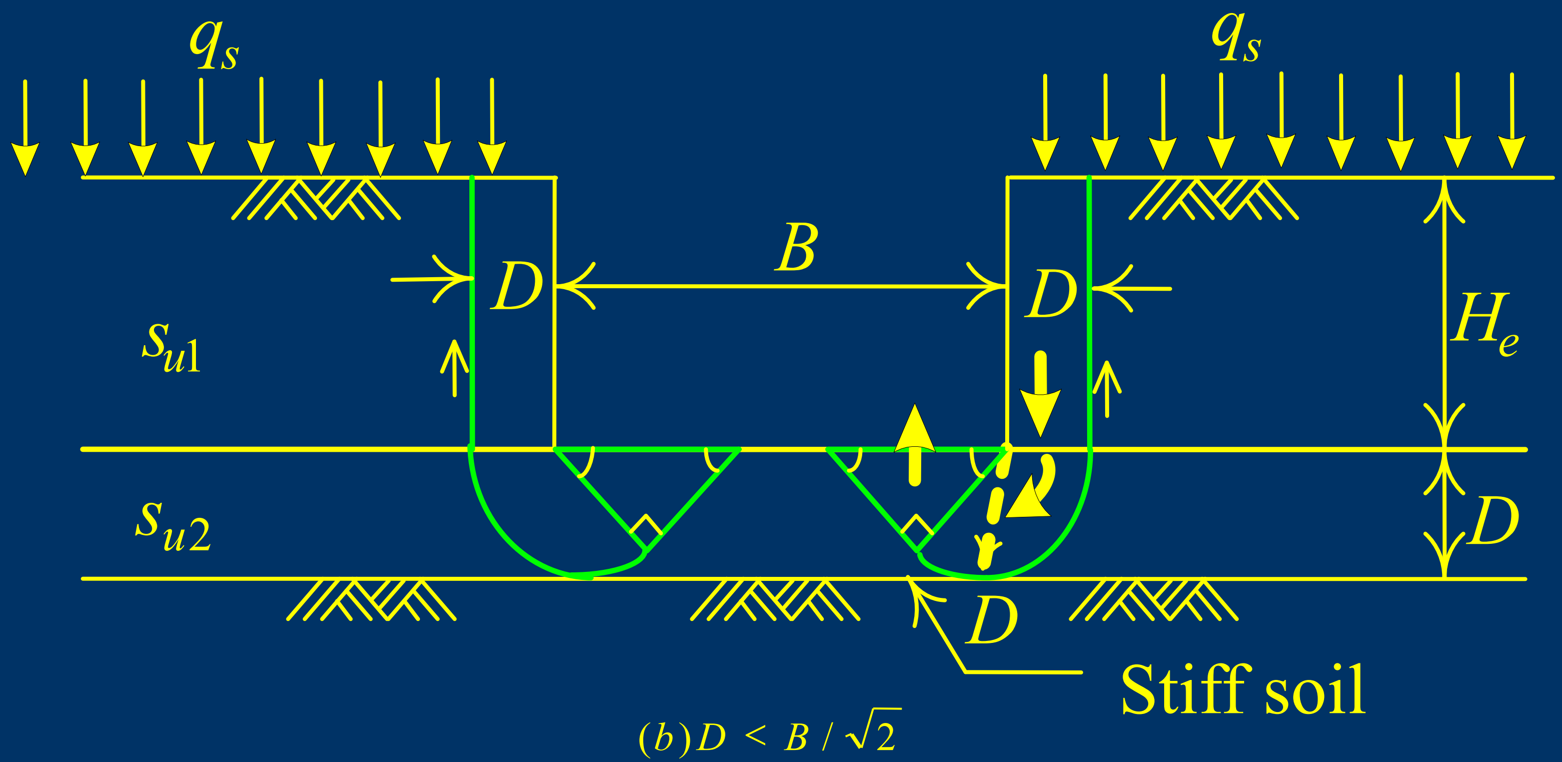


FIGURE 5.14 Analysis of basal heave using Terzaghi's method

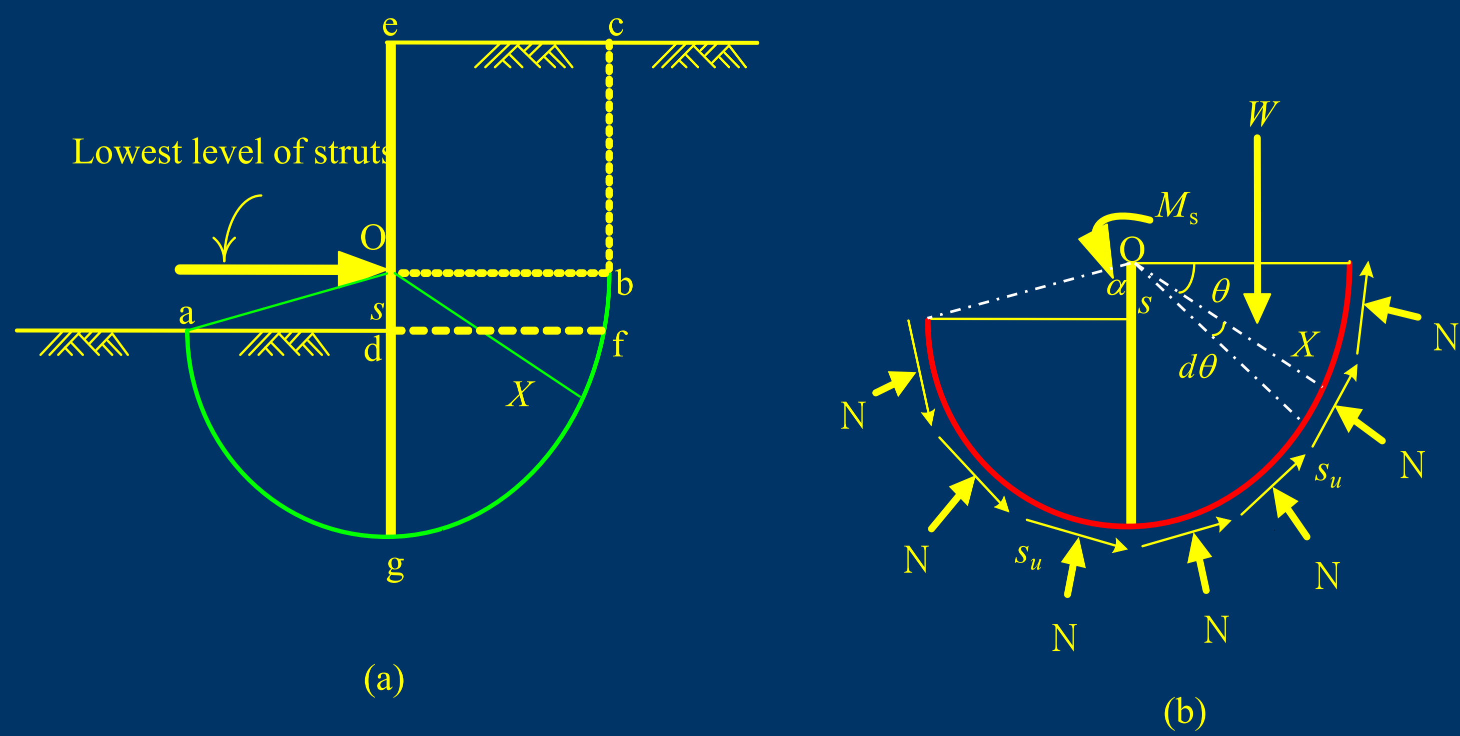


FIGURE 5.20 Analysis of basal heave by the slip circle method
(a) the failure surface (b) balance of the a free body

北投自強路 excavation failure case

5.5.3 Case Study of Overall Shear Failure

The excavation case was located in Taipei. The width of the excavation was 17.6m； the length was 100.1m ；the depth was 13.45m. The excavation adopted a 70cm thick , 34, deep diaphragm wall as the retaining wall. There four levels of struts and the excavation was carried out in 5 stages.

力霸百老匯
excavation failure

力霸百老匯 excavation failure case

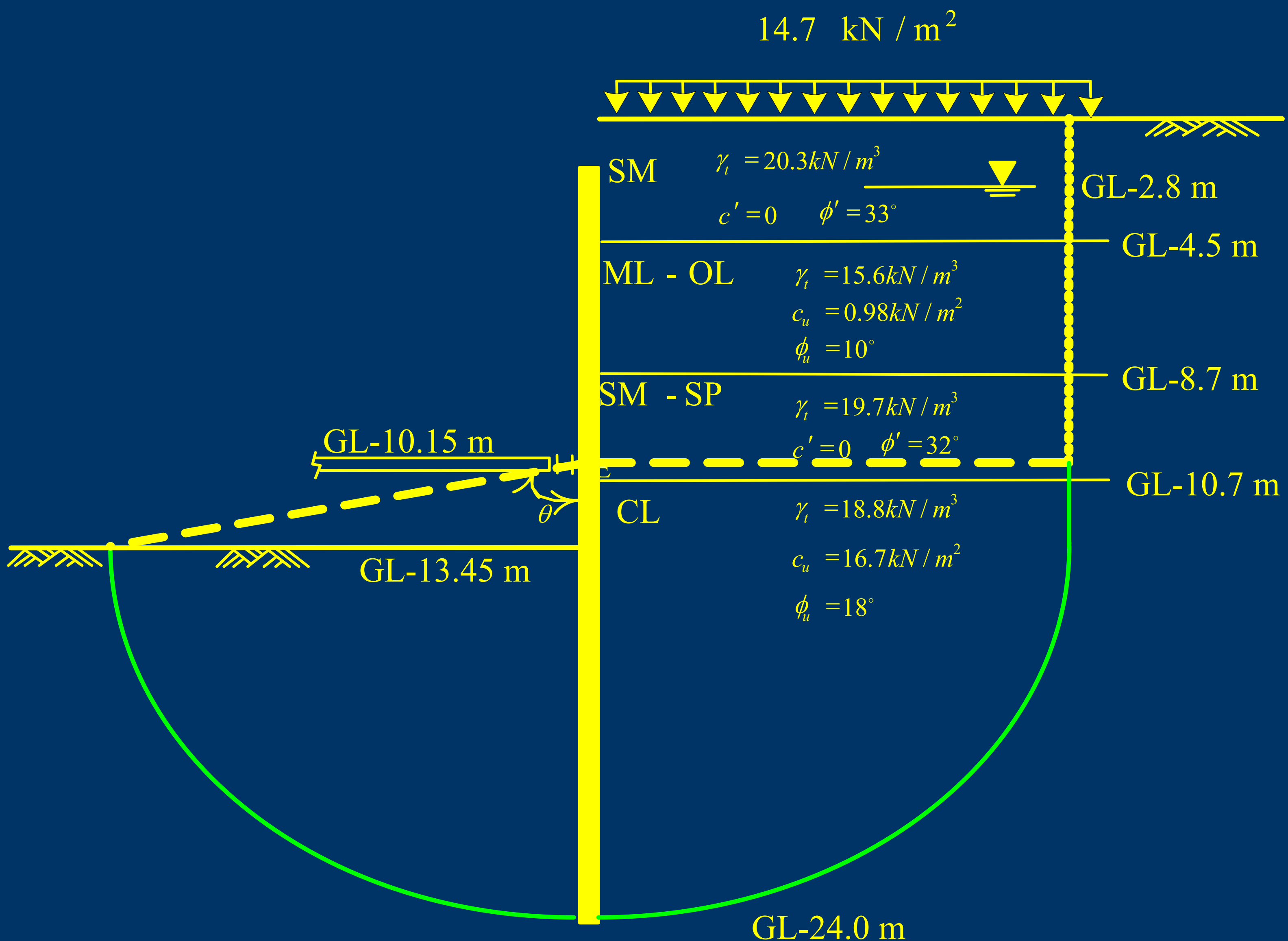


FIGURE 5.23 Stability analysis of an excavation case history
(a) excavation and geological profiles

c_u and ϕ_u were the total stress strength parameters of the clay soils, obtained from the triaxial CU test

adopt by the original designer F_p will be 1.5

F_b will be 2.3

We assume the soil below the lowest level of struts (GL-10.15 m) to be a clayey layer, the adhesion between the retraining wall and the soil $c_w = 2s_u / 3$
and the normalized undrained shear strength $s_u / \sigma_v' = 0.22$

$$\sigma_a = \sigma_v K_a - 2cK_{ac} \qquad (4.16)$$

$$\sigma_p = \sigma_v K_p + 2cK_{pc} \qquad (4.18)$$

At the depth of GL-10.15 m

$$\sigma_v = 20.3 \times 4.5 + 15.6 \times 4.2 + 19.7 \times 1.45 = 185.5 kN/m^2$$

$$u = (10.15 - 2.8) \times 9.81 = 72.1 kN/m^2$$

$$\sigma_v' = \sigma_v - u = 113.3 kN/m^2$$

$$s_u = 0.22\sigma_v' = 0.22 \times 113.3 = 24.9 kN/m^2$$

$$\sigma_{h,a} = \sigma_v K_a - 2s_u\sqrt{K_a\left(1+\frac{c_w}{s_u}\right)} = 185.5 - 2(24.9)\sqrt{1+\frac{2}{3}} = 121.2 kN/m^2$$

At the depth of GL-13.45 m

Before excavation—

$$\sigma_v = 185.5 + 19.7 \times 0.55 + 18.8 \times 2.75 = 248.0 kN/m^2$$

$$u = (13.45 - 2.8) \times 9.81 = 104.5 kN/m^2$$

$$\sigma_v' = \sigma_v - u = 143.5\ kN/m^2$$

$$s_u = 0.22\sigma_v' = 0.22 \times 143.5 = 31.6 kN/m^2$$

after excavation was started, $\sigma_v = 0$ on the passive side, but s_u value stayed unchanged.

Thus,

$$\sigma_{h,p} = \sigma_v K_p + 2s_u\sqrt{K_p(1+\frac{c_w}{s_u})} = 0 + 2(31.6)\sqrt{1+\frac{2}{3}} = 81.5 kN/m^2$$

At the depth of GL-24.0 m

The active side—

$$\sigma_v = 248.0 + 18.8 \times 10.55 = 446.3 kN/m^2$$

$$u = (24 - 2.8) \times 9.81 = 208.0 kN/m^2$$

$$\sigma_v' = \sigma_v - u = 238.3 kN/m^2$$

$$s_u = 0.22\sigma_v' = 0.22 \times 238.3 = 52.4 kN/m^2$$

$$\sigma_{h,a} = \sigma_v K_a - 2s_u\sqrt{K_a(1 + \frac{c_w}{s_u})} = 446.3 - 2(52.4)\sqrt{1 + \frac{2}{3}} = 311.0 kN/m^2$$

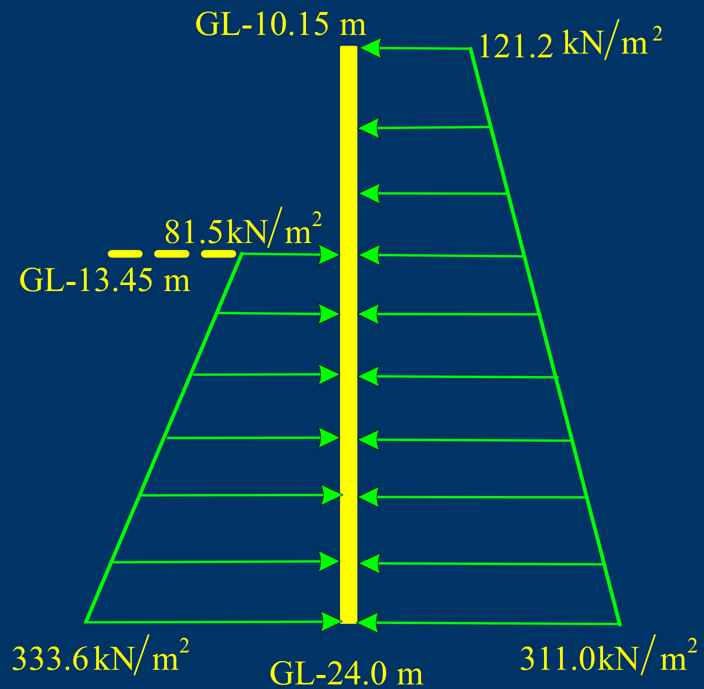
At the depth of GL-24.0 m

The passive side—

After excavation was start s_u stayed constant,

$$\sigma_v = 18.8 \times (24.0 - 13.45) = 198.3 kN/m^2$$

$$\sigma_{h,p} = \sigma_v K_p + 2s_u \sqrt{K_p (1 + \frac{c_w}{s_u})} = 198.3 + 2(52.4)\sqrt{1 + \frac{2}{3}} = 333.6 kN/m^2$$



(b)

FIGURE 5.23 Stability analysis of an excavation case history
(b) distribution of earth pressure for the push-in analysis

The factor of safety against push-in as

$$F_p = \frac{81.5\times10.55\times(10.55/2+3.3)+(333.6-81.5)\times10.55\times0.5\times(10.55\times2/3+3.3)}{121.2\times13.85\times13.85/2+(311.0-121.2)\times13.85\times0.5\times13.85\times2/3} = 0.89$$

Compute the factor of safety against basal heave according to Slip circle method ：

Similarly, assuming the soil below the lowest level of struts is clay, the average value of the undrained shear strengths (the active side) of the soil between GL-10.15 m and GL-24.0 m would be

$$s_{u,a} = \frac{24.9 + 52.4}{2} = 38.7 kN/m^2$$

The average value of the undrained shear strengths of the soil between GL-13.45 m and GL-24.0 m would be

$$s_{u,p} = \frac{31.6 + 52.4}{2} = 42.0 kN/m^2$$

The radius of the failure circular arc would be

$$24 - 10.15 = 13.85m$$

The central angle of the failure circular arc on the active side would be

$$\theta = \frac{\pi}{2} = 1.57$$

The central angle of the failure circular arc on the passive side would be

$$\theta = \cos^{-1}\left(\frac{3.3}{13.85}\right) = 1.33$$

The factor of safety against circular arc failure would be

$$F_b = \frac{13.85 \times 1.33 \times 42.0 \times 13.85 + 13.85 \times 1.57 \times 38.7 \times 13.85}{\sigma_{v(GL-13.45)} \times 13.85 \times 13.85/2} = \frac{22370}{23786} = 0.94$$

Computing the factor of safety against basal heave following Terzaghi's method：

The width of the excavation $B = 17.6m$, $B/\sqrt{2}$ was larger than the penetration depth (10.55 m). Assumed failure surface will pass below the bottom of the retaining wall.

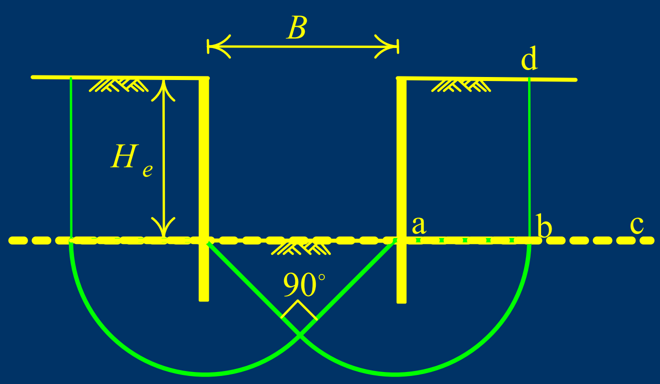


FIGURE 5.15 Relation between the embedded part of the retaining wall and the failure surface (b) small penetration depth

The average undrained shear strength of soil within the range of the failure circle can be calculated as follows：

of soil $13.45 + B/\sqrt{2} = 25.9m$ deep below the ground surface--

$$\sigma_v = 248.0 + 18.8 \times (25.90 - 13.45) = 482.1 kN/m^2$$

$$\sigma_v' = \sigma_v - u = 482.1 - (25.90 - 2.8) \times 9.81 = 255.5 kN/m^2$$

$$s_u = 0.22\sigma_v' = 0.22 \times 255.5 = 56.2 kN/m^2$$

The average undrained shear strength within the range of the failure circle would be

$$s_u = (31.6 + 56.2)/2 = 43.9 kN/m^2$$

As computed earlier, the total stress outside the excavation zone at the depth equaling the excavation surface would be

$$\sigma_v = 248.0 kN/m^2$$

To simplify the analysis and be conservative, we assume the soil above the excavation surface is clay and has soil shear strength expressed as $s_u / \sigma'_v = 0.22$. The average undrained shear strength of the soil outside the excavation zone and the excavation surface would be

$$s_u = \frac{0.22\sigma'_{v(GL-13.45)}}{2} = \frac{31.6}{2} = 15.8 kN/m^2$$

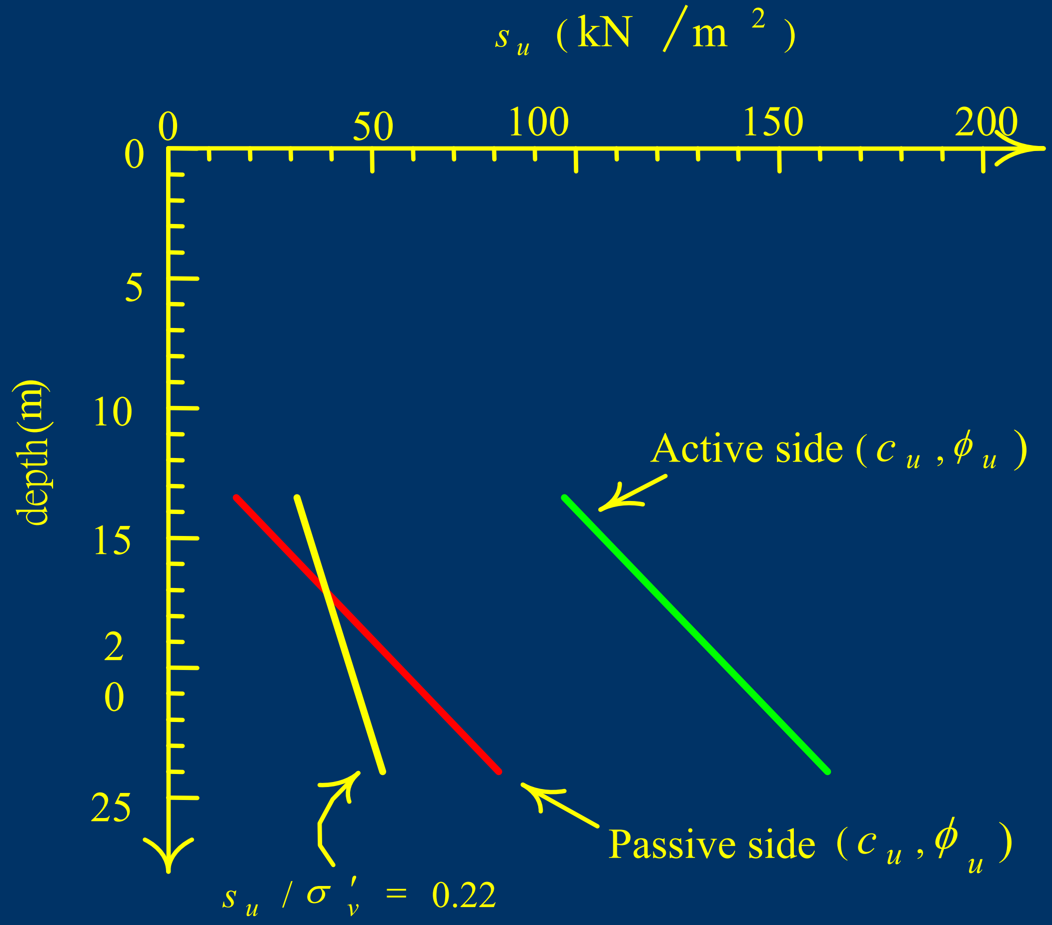
The factor of safety according to Terzaghi's method would be

$$F_b = \frac{Q_u}{W - s_{u1} H_e} = \frac{5.7 \times 43.9 \times 17.6/\sqrt{2}}{248.0 \times 17.6/\sqrt{2} - 15.8 \times 13.45} = \frac{3118}{2874} = 1.08$$

The factor of safety following Bjerrum and Eide's method would be

$$F_b = \frac{s_u N_c}{\gamma H_e + q_s} = \frac{43.9 \times 6.2}{248.0} = 1.10$$

Undrained shear strength and the depth：



(c)

FIGURE 5.23 Stability analysis of an excavation case history
(c) the undrained shear strength used in the analysis

【Example 5.1】Assume a 9.0 m deep excavation in a sandy ground and the lowest level of struts is 2.5m above the excavation surface. The level of groundwater outside the excavation zone is ground surface high while that within the excavation zone is as high as the excavation surface. The unit weight of saturated sandy soils $\gamma_{sat}=20\ \mathrm{kN/m^3}$, the effective cohesion $c'=0$ and the effective angle of friction $\phi'=30^\circ$. Because of the difference between the levels of groundwater , seepage will occur. Assume that the friction angles (δ) between the retaining wall and soil on both the active and passive sides are $0.5\phi'$ and the factor of safety against push-in, $F_p=1.5$. Compute the required penetration depth(H_p).

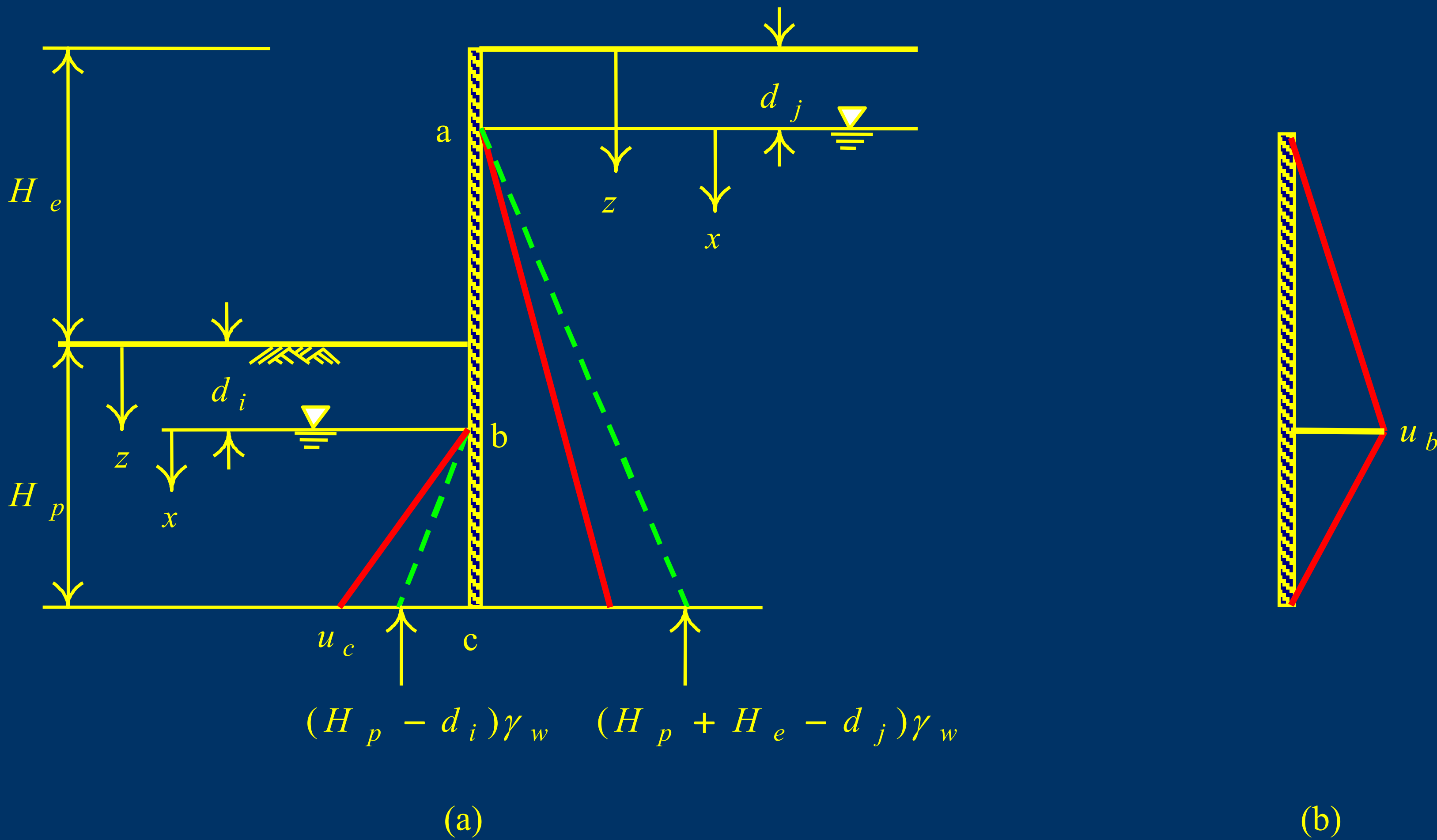


FIGURE 4.22 Simplified analysis method for seepage
(a) distribution of water pressure (b) net water pressure

【Solution】

1.determine the coefficient of the earth presure

Compute both the active and passive earth pressures following Caquot-Kerisel's earth pressure theory. When $\delta = 0.5\phi'$, the coefficients of active and passive earth pressure can be found from Figure 4.9 and Figure 4.10 to be 0.3 and 4.6 separately. Thus, the coefficients of the horizontal active and passive earth pressure would be

$$K_{a,h} = 0.3\cos\delta = 0.3\cos 0.5\phi' = 0.29$$

$$K_{p,h} = 4.6\cos\delta = 4.6\cos 0.5\phi' = 4.4$$

2. Compute the effective active active earth pressure on the wall

At the lowest level of strut (z=6.5m, x=6.5m) --

$\sigma'_{a,h} = \sigma'_v K_{a,h}$ $\quad\quad\quad$ $\sigma_v = 20 \times 6.5 = 130\,\text{kN/m}^2$

According to Eq. 4.51the porewater pressure at x away from upstream water level would be

$$u = \frac{2x(H_p - d_i)\gamma_w}{2H_p + H_e - d_i - d_j} = \frac{2 \times 6.5 \times H_p \times 9.81}{2H_p + 9} = \frac{63.77 H_p}{H_p + 4.5}$$

$$\sigma'_{a,h} = (\sigma - u) K_{a,h} = (130 - \frac{63.77 H_p}{H_P + 4.5}) \times 0.29 = 37.7 - \frac{18.49 H_p}{H_p + 4.5}$$

At the bottom of the retaining wall($z = 9 + H_p$, $x = 9 + H_p$)—

$$\sigma_v = 20 \times (9 + H_p) = 180 + 20H_p$$

$$u = \frac{2x(H_p - d_i)\gamma_w}{2H_p + H_e - d_i - d_j} = \frac{2 \times (9 + H_p) \times H_p \times 9.81}{2H_p + 9} = \frac{9.81H_p^2 + 88.29H_p}{H_p + 4.5}$$

$$\sigma'_{a,h} = (180 + 20H_p - \frac{9.81H_p^2 + 88.29H_p}{H_p + 4.5}) \times 0.29$$

$$= (52.2 + 5.8H_p - \frac{2.84H_p^2 + 25.60H_p}{H_p + 4.5})$$

3. Compute the lateral effective passive earth pressure on the wall

At the bottom of the retaining wall（ $z = H_p$ ）—

$$\sigma_v = 20 \times H_p = 20H_p$$

$$u = \frac{9.81H_p^2 + 88.29H_p}{H_p + 4.5}$$

$$\sigma'_{p,h} = (20H_p - \frac{9.81H_p^2 + 88.29H_p}{H_p + 4.5}) \times 4.4$$

$$= 88H_p - \frac{43.16H_p^2 + 388.48H_p}{H_p + 4.5}$$

4. Compute the maximum net water pressure (at the excavation surface)

According to Eq. 4.53, the maximum net water pressure would be

$$u_b = \frac{2(H_e + d_i - d_j)(H_p - d_i)\gamma_w}{2H_p + H_e - d_i - d_j} = \frac{2 \times 9 \times H_p \times 9.81}{2H_p + 9} = \frac{88.29 H_p}{H_p + 4.5}$$

5. The effective earth pressure on both sides of the wall and the distribution of the net water pressure are as shown in Figure 5.24

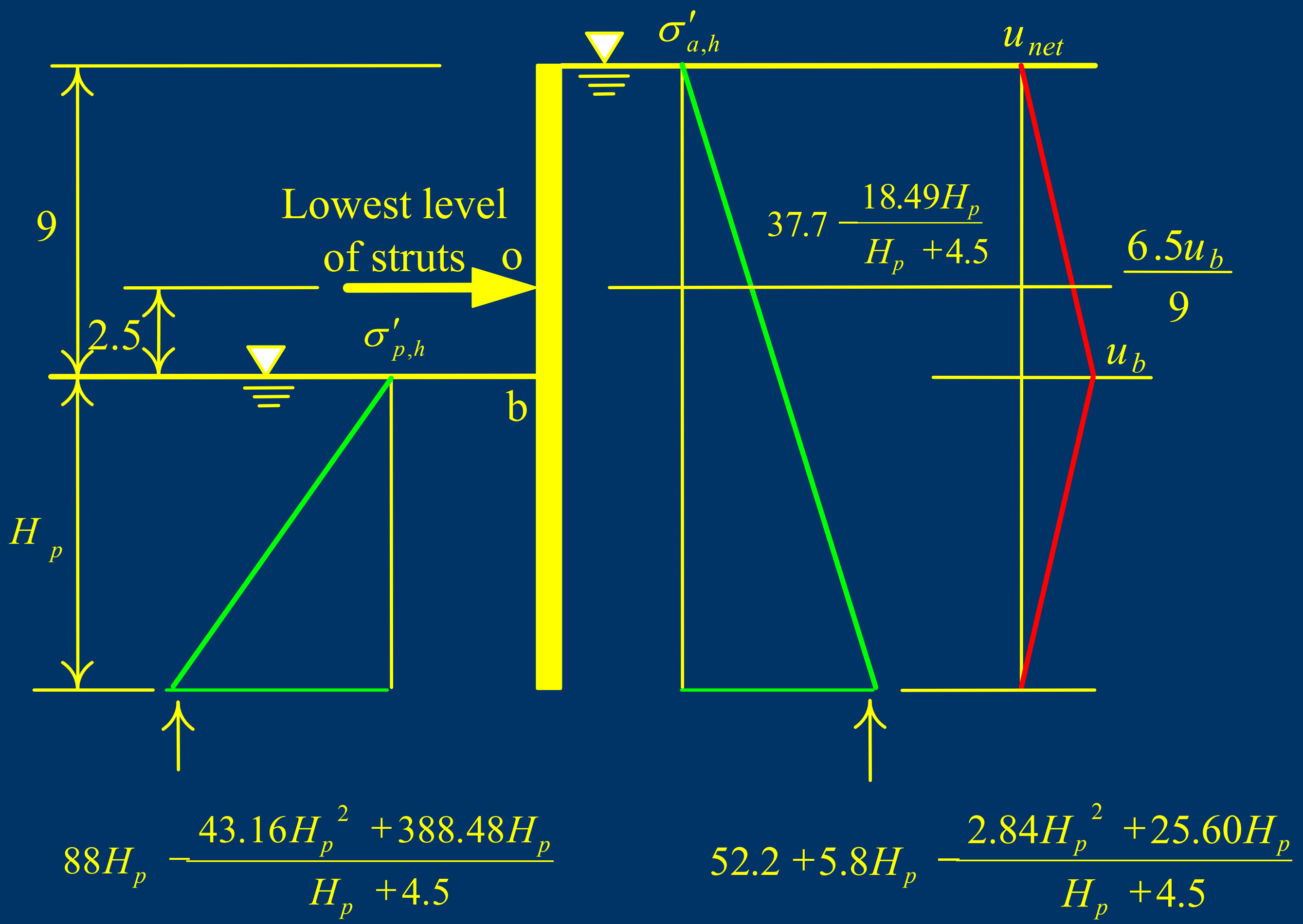


FIGURE 5.24 Distribution of lateral earth pressure

6. Compute the driving moment (M_d) and the resistant moment (M_r) for the free body below the lowest level of struts

$$
\begin{aligned}
M_d &= P_{a,h} L_a \\
&= (37.7 - \frac{18.49 H_p}{H_p + 4.5}) \times \frac{(H_p + 2.5)^2}{2} + (14.5 + 5.8 H_p - \frac{2.84 H_p^2 + 7.11 H_p}{H_p + 4.5}) \times \frac{2(H_p + 2.5)^2}{2 \times 3} \\
&+ \frac{u_b H_p}{2} \times (2.5 + \frac{H_p}{3}) + \frac{6.5 u_b}{9} \times \frac{2.5^2}{2} + \frac{2.5 u_b}{9} \times \frac{2 \times 2.5^2}{2 \times 3} \\
&= (23.68 + 1.93 H_p - \frac{0.95 H_p^2 + 11.62 H_p}{H_p + 4.5})(H_p + 2.5)^2 + [0.17 H_p^2 + 1.25 H_p + 2.84] u_b
\end{aligned}
$$

$$M_r = P_{p,h} L_p$$

$$= \left(88H_p - \frac{43.16H_p^{\ 2} + 388.48H_p}{H_p + 4.5}\right) \times \frac{H_p}{2} \times \left(2.5 + \frac{2H_p}{3}\right)$$

$$= \left(44H_p^{\ 2} - \frac{21.58H_p^{\ 3} + 194.24H_p^{\ 2}}{H_p + 4.5}\right)\left(2.5 + \frac{2H_p}{3}\right)$$

7. determine the penetration depth H_p

$$F_p = \frac{M_r}{M_d} = 1.5$$

Then we have $H_p = 7.25$ m

【Example 5.2】An excavation in clay goes 9.0m in to the ground (H_e =9.0m). The groundwater outside the excavation zone is at the ground surface level while that within the excavation zone is at the level of the excavation surface. γ_{sat}=17.0 $\mathrm{kN/m^3}$. The undrained shear strength s_u=45 $\mathrm{kN/m^2}$.
Suppose the excavation width B =10m and the excavation length L =30m. Compute the factor of safety against basal heave according to Terzaghi's method and Bjerrum and Eide's method, respectively.

【Solution】

In this example, the surcharge $q_s = 0$

According to Terzahi's method,

$$F_b = \frac{1}{H_e} \times \frac{5.7 \times s_u}{\gamma - \frac{s_u}{0.7B}} = \frac{1}{9} \frac{5.7 \times 45}{17 - \frac{45}{0.7 \times 10}} = 2.7$$

According to Bjerrum and Eide's method,

$$\frac{L}{B} = \frac{30}{10} = 3.0$$

$$\frac{H_e}{B} = \frac{9}{10} = 0.9$$

According to Figure 5.17, we have

$$N_c = 7.1$$

$$F_b = \frac{s_u N_c}{\gamma_t H_e} = \frac{45 \times 7.1}{17 \times 9} = 2.09$$

5.7 Upheaval

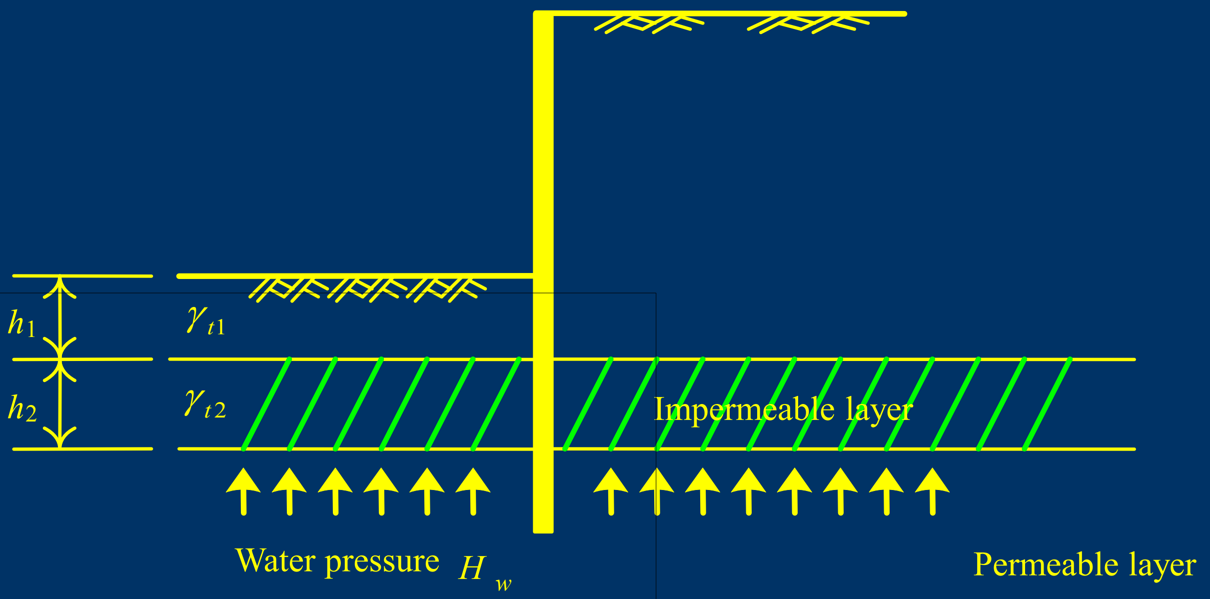


FIGURE 5.31 Analysis of upheaval

$$F_{up} = \frac{\sum_i \gamma_{ti} \times h_i}{H_w \times \gamma_w} \qquad (5.17)$$

The factor of safety against upheaval F_{up} should be larger than or equal to 1.2

5.8 Sand Boiling

5.8.1 Mechanism and Factors of Safety

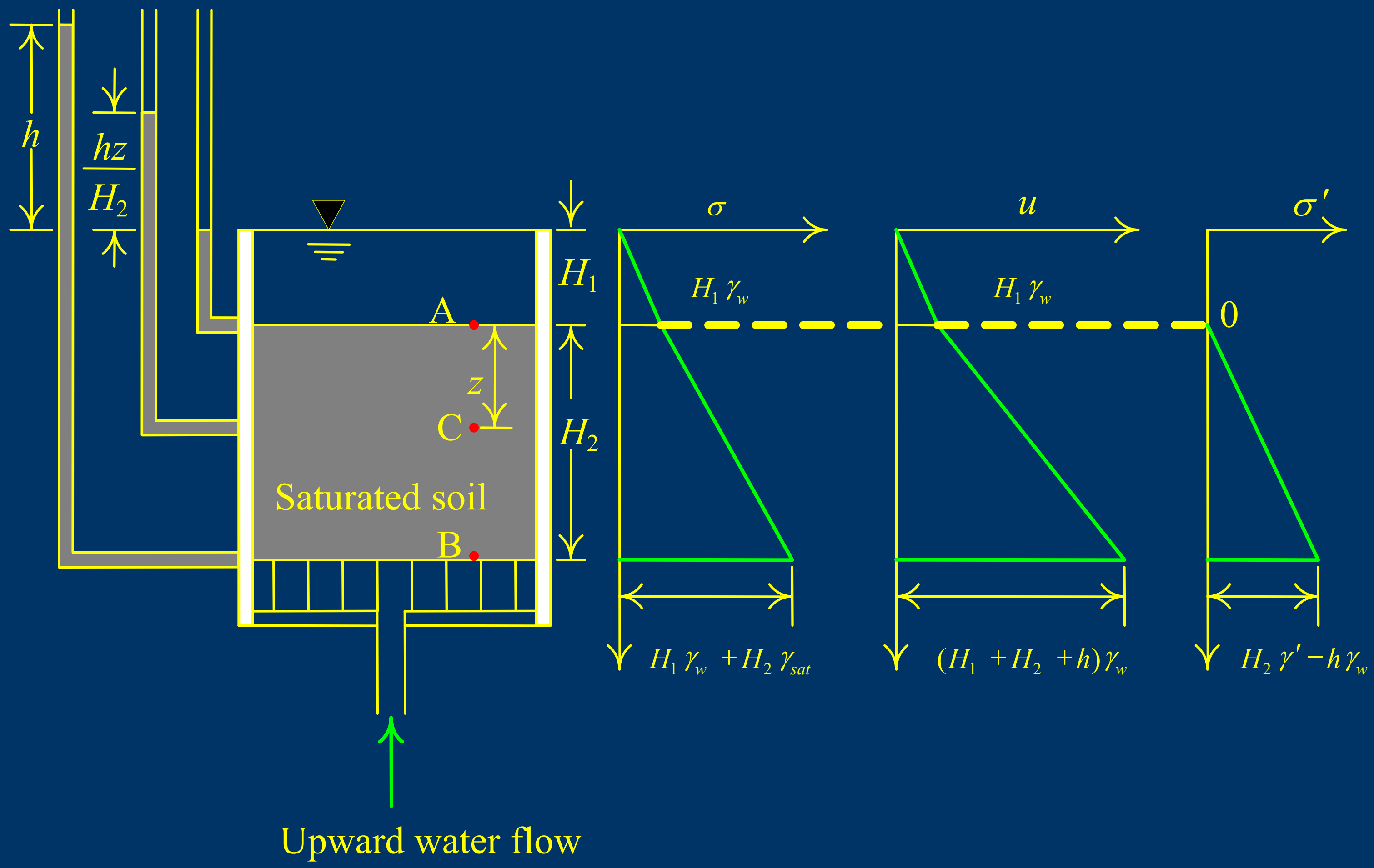


FIGURE 5.32 Total stresses, effective stresses, and change of porewater pressure in sandy soils acted on by an upward water flow

Besides, according to the phase relationship of soil, the submerged unit weight is

$$\gamma' = \left(\frac{G_s - 1}{1 + e}\right)\gamma_w \qquad (5.24)$$

The critical hydraulic gradient is then

$$i_{cr} = \frac{G_s - 1}{1 + e} \qquad (5.25)$$

$$F_s = \frac{i_{cr}}{i_{\max(exit)}} \qquad (5.26)$$

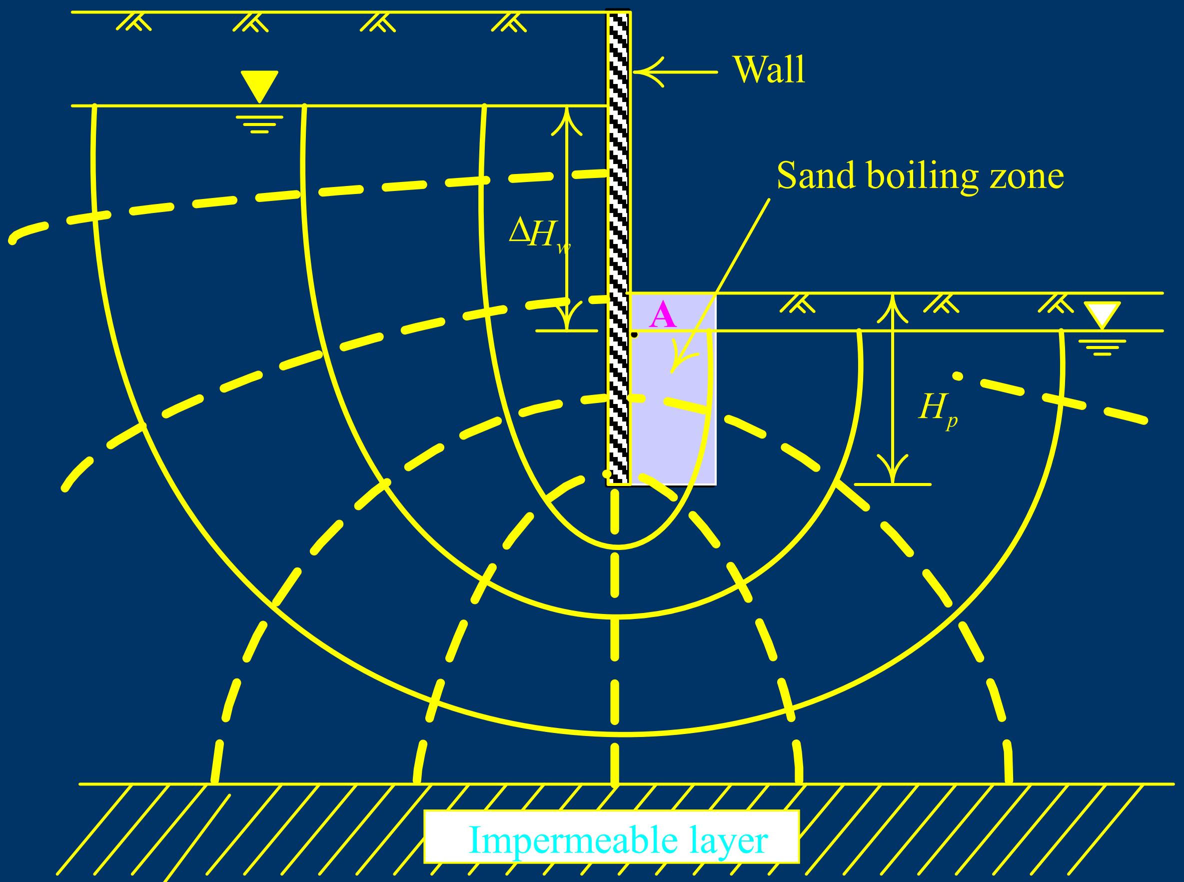


FIGURE 5.33 Seepage in soil below sheetpiles

$$F_s = \frac{i_{cr}}{i_{\max(exit)}} \qquad (5.26)$$

Terzaghi's method：

$$U = (\text{the volume of the soil column}) \times (i_{av}\,\gamma_w) = \frac{1}{2} H_p^{\,2}\, i_{avg}\, \gamma_w \tag{5.27}$$

$$W' = \frac{1}{2} H_p^2 (\gamma_{sat} - \gamma_w) = \frac{1}{2} H_p^2 \gamma' \tag{5.28}$$

The factor of safety is

$$F_s = \frac{W'}{U} = \frac{\gamma'}{i_{avg}\,\gamma_w} \tag{5.29}$$

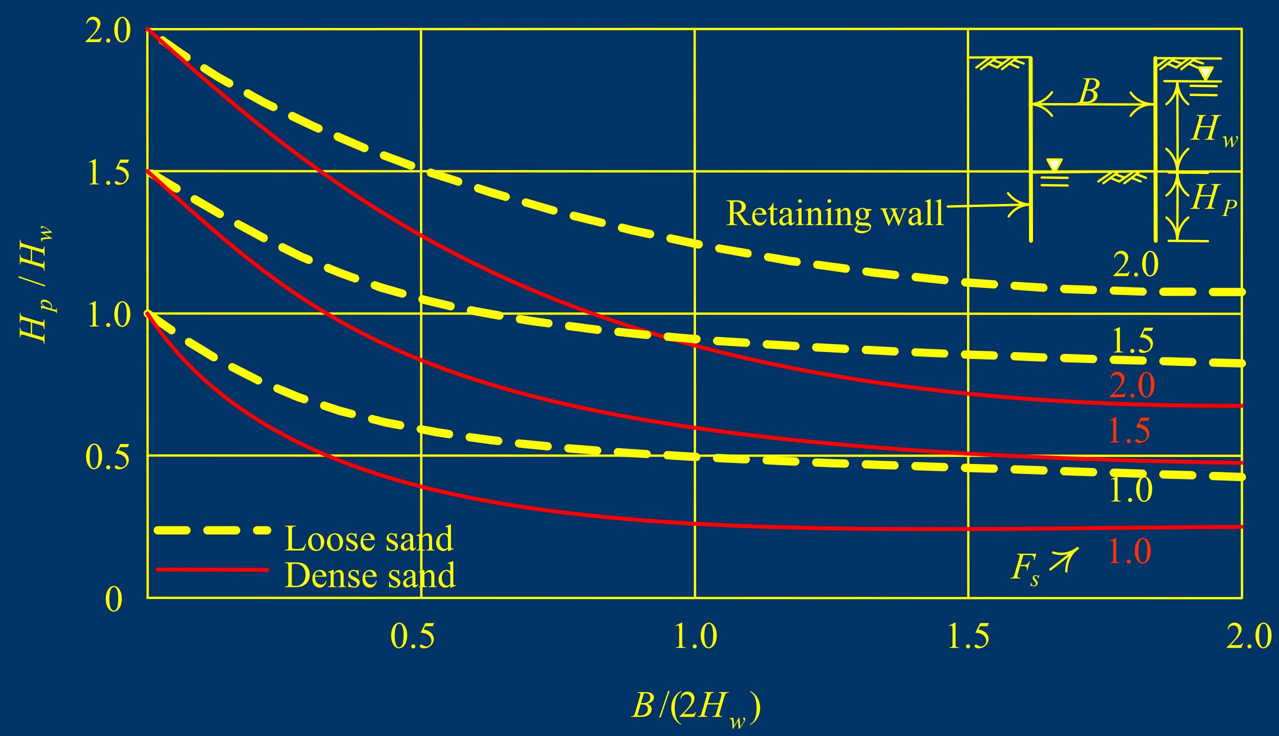
Provided the computed factor of safety is too small, we can consider placing filters at the exits of seepage. Assuming the weight of filters is Q, the factor of safety will be

$$F_s = \frac{W' + Q}{U} \tag{5.30}$$

In general, the required F_s for the above equation should be greater than of equal to 1.5

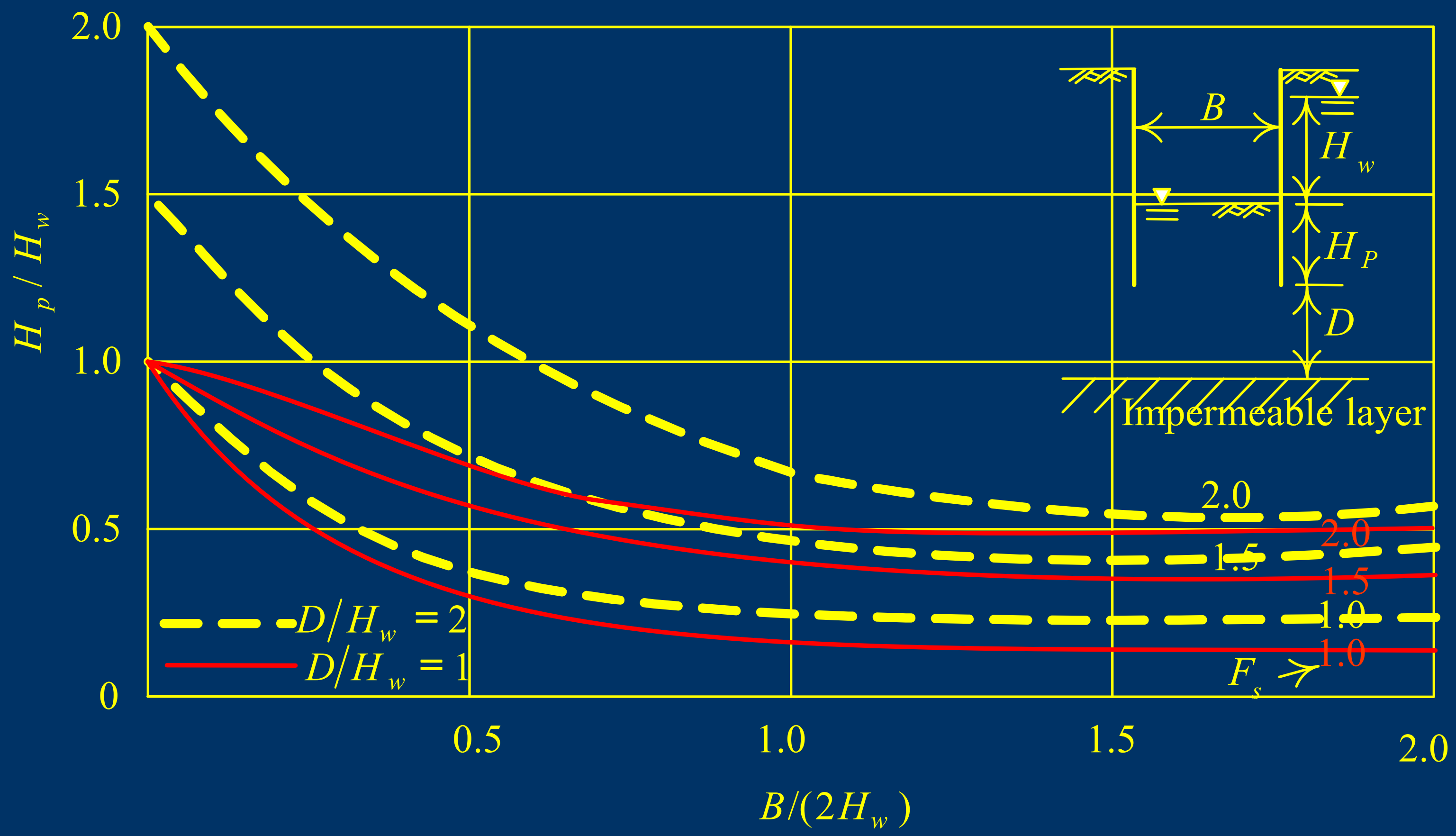
Marsland's method：

DM7.1 suggested that the reasonable factor of safety against piping in an excavation be around 1.5 to 2.0.



(a)

FIGURE 5.34 Relations between wall penetration depths and factors of safety against sand boiling (a) dense and loose sands with the impermeable layer located at the infinite depth



(b)

FIGURE 5.34 Relations between wall penetration depths and factors of safety against sand boiling (b) dense sand with the impermeable layer located at a finite depth

One dimension seepage method：

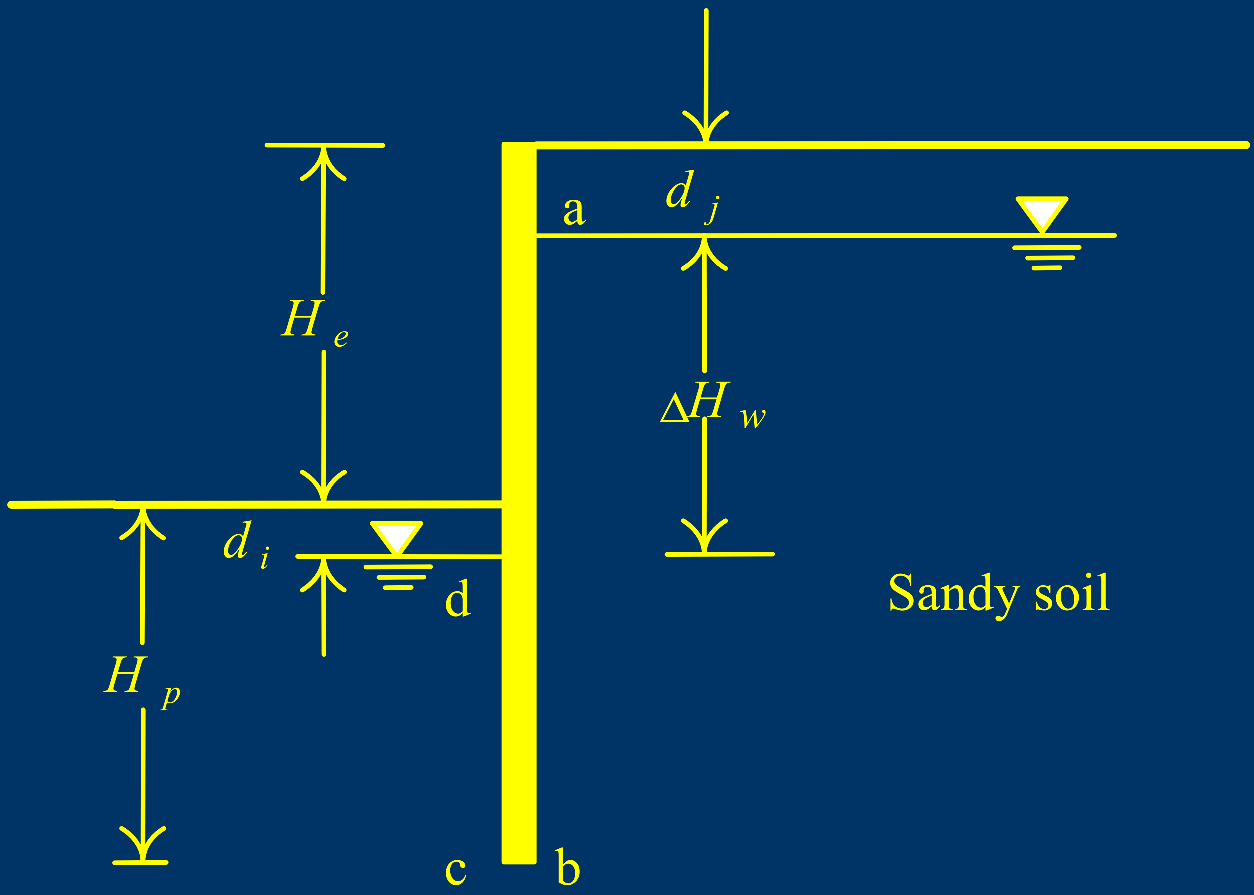


FIGURE 5.35 Analysis of sand boiling

If we assume the datum is at the downstream level, the total head at the elevation of downstream (point **d**) will be

$$h_{t,d} = h_e + h_p = 0 + 0 = 0 \qquad (5.31)$$

The total head at the upstream elevation (point **a**) will be

$$h_{t,a} = h_e + h_p = H_e + d_i - d_j + 0 = H_e + d_i - d_j \qquad (5.32)$$

The difference of the total heads between upstream and downstream levels will be

$$\Delta H_w = h_{t,a} - h_{t,d} = H_e + d_i - d_j \qquad (5.33)$$

Suppose the seepage is one dimensional and the hydraulic gradients for each depth along the flow path **abcd** are equal. the hydraulic gradient will be

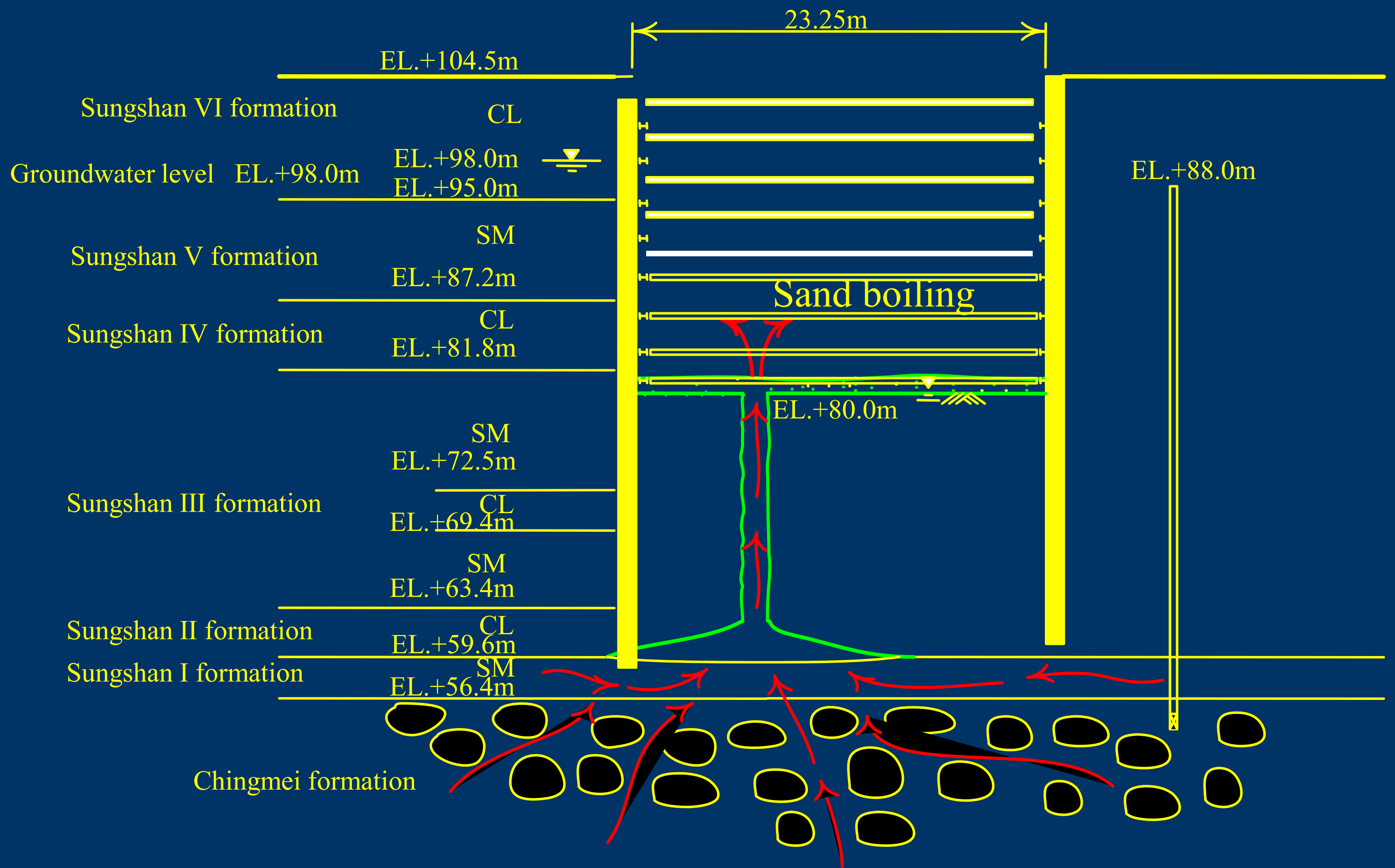
$$i_{avg} = \frac{\Delta H_w}{H_e - d_j + d_i + 2(H_p - d_i)} = \frac{\Delta H_w}{H_e + 2H_p - d_i - d_j}$$

The factor of safety against boiling will be

$$F_s = \frac{i_c}{i_{avg}} = \frac{\gamma'(H_e + 2H_p - d_i - d_j)}{\gamma_w \Delta H_w}$$

The required F_s for the above equation should be greater than or equal to 1.5.

5.8.2 Case Study



(a)

FIGURE 5.36 Excavation of Siemen Station of Taipei Rapid Transit System (a) excavation and geological profiles

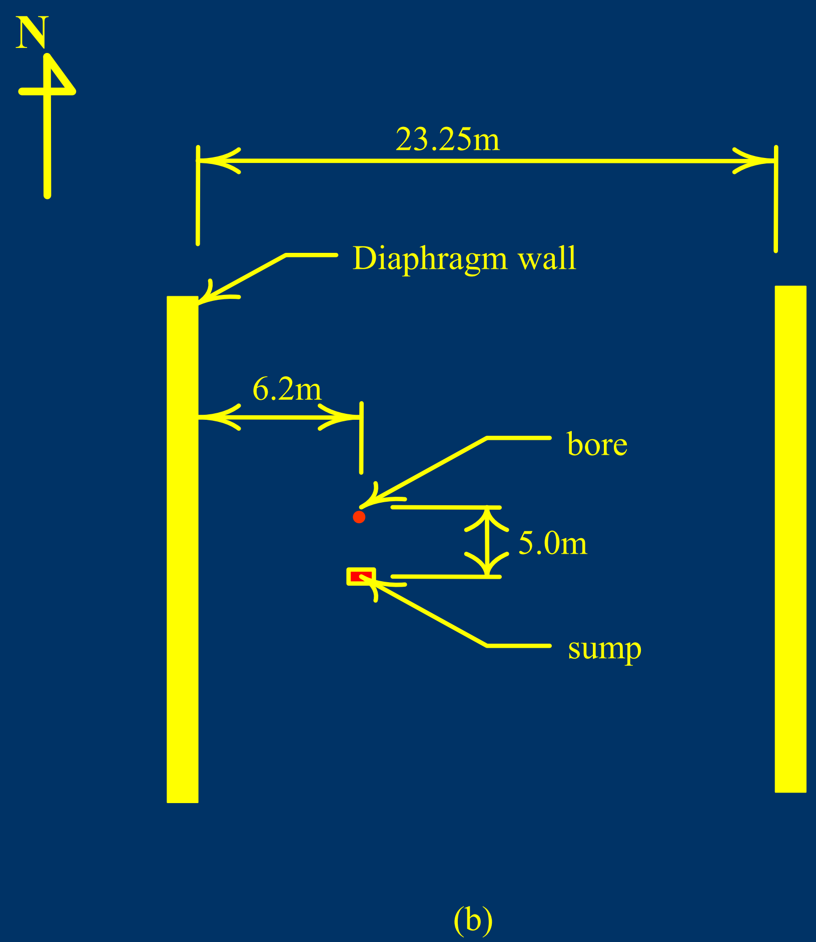
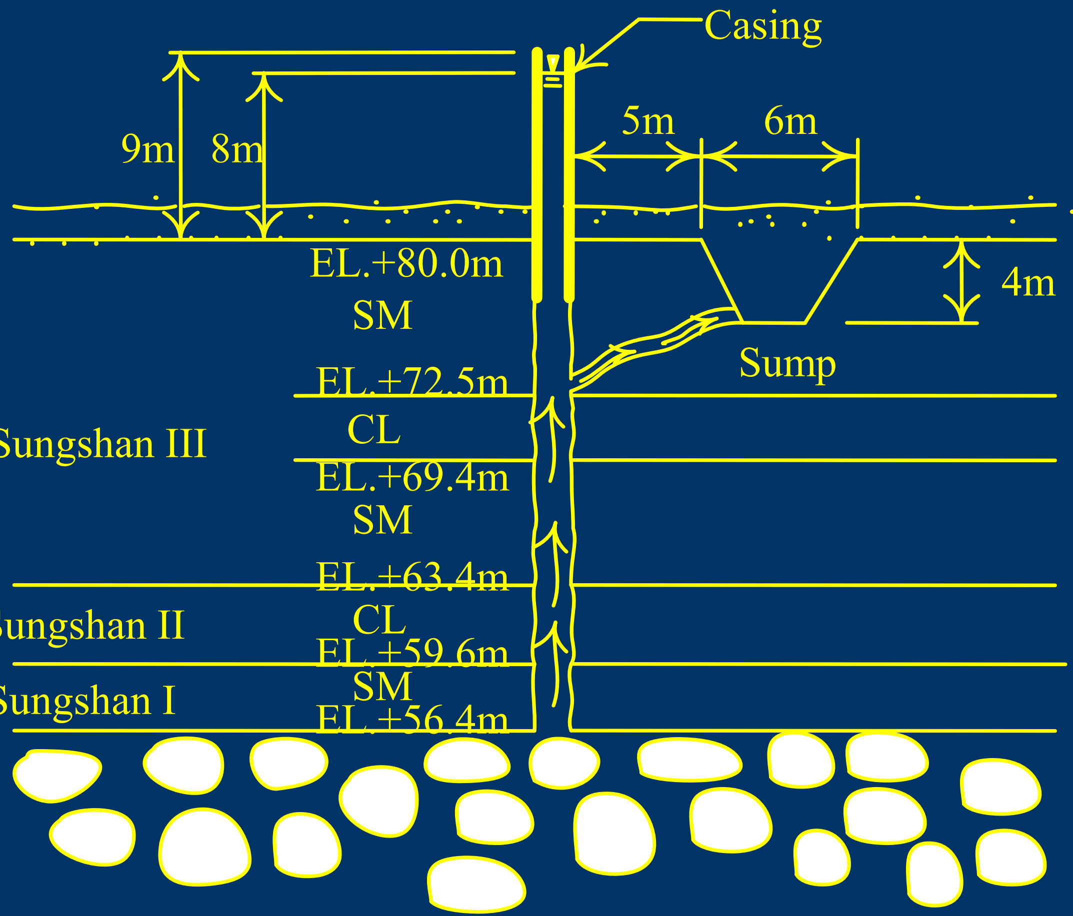


FIGURE 5.36 Excavation of Siemen Station of Taipei Rapid Transit System
(b) plan view



(c)

FIGURE 5.36 Excavation of Siemen Station of Taipei Rapid Transit System
(c) process of sand boiling

TABLE 5.2 Stability analysis methods for strutted walls and the required minimum factors of safety

	Overall shear failure		Sand boiling	Upheaval
	Push-in	Basal heave		
Sand or grave	Gross pressure method {Eq.5.5, $F_p \geq 1.2$, assuming M_s=0}(1)	--	Harza's method {Eq. 5.26, $F_s \geq 2.0$}(4) Terzaghi's method {Eq. 5.30 $F_s \geq 1.5$}(1, 5) Marsland's method {Fig. 5.34, $F_s \geq$ 1.5~2.0}(4) Simplified 1-D seepage method {Eq. 5.35, $F_s \geq 2.0$}(1, 5)	--
Clay	Gross pressure method {Eq. 5.5, $F_p \geq 1.2$} (1) assuming M_s=0}(1)	Terzaghi's method Bjerrum and Eide's Slip circle method {Eq. 5.15, $F_b \geq 1.2$}(1)	--	--
Alternated layers of sand (or gravel) and clay	Gross pressure method {Eq. 5.5, $F_p \geq 1.2$ } (1) assuming M_s=0}(1)	Terzaghi's method {Eq. 5.9 or 5.10, $F_b \geq 1.5$}(1,2,3) Bjerrum and Eide's method{Eq. 5.12 or 5.13, $F_b \geq 1.2$}(4,3) Slip circle method {Eq. 5.15, $F_b \geq 1.2$}(1,3)	Short term behaviors can be ignored while long term behaviors may need consideration. The analysis methods are the same as those for sand and gravel.	{Eq. 5.17, $F_{up} \geqq 1.2$}(1)

NOTE:

(1) The methods and factors of safety are suggested by TGS (2001) and JSA (1988)

(2) The factor of safety is suggested by Mana and Clough (1981)

(3) It is only when clay is the dominant soil layer that the analysis of basal heave is required

(4) The factor of safety is suggested by NAVFAC DM 7.1 (1982)

(5) TGS (2001) and JSA (1988) suggest the conservative value obtained by Terzaghi's method or the simplified 1-D seepage be adopted for design.