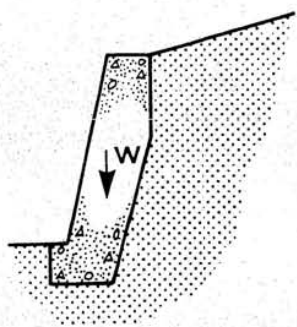
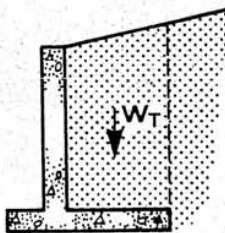


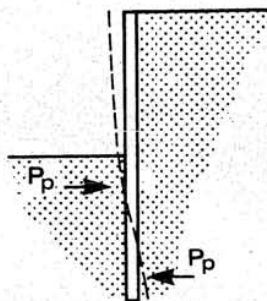
PROGETTO OPERE DI SOSTEGNO



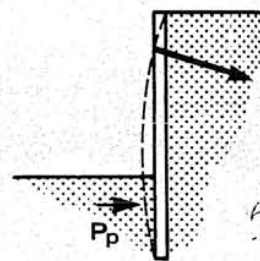
a)



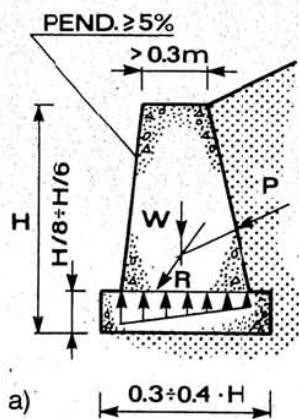
b)



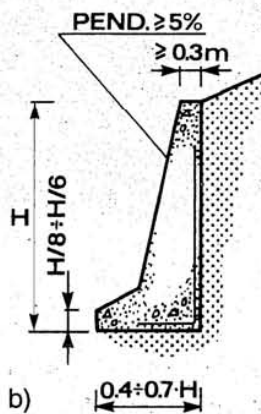
c)



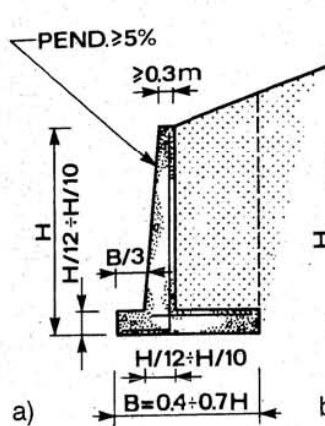
d)



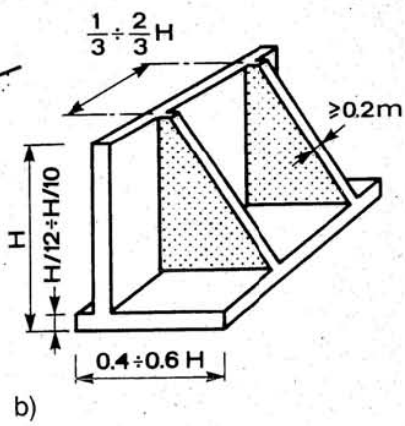
a)



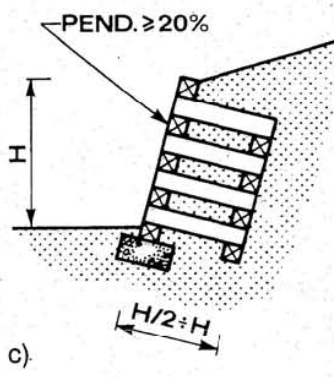
b)



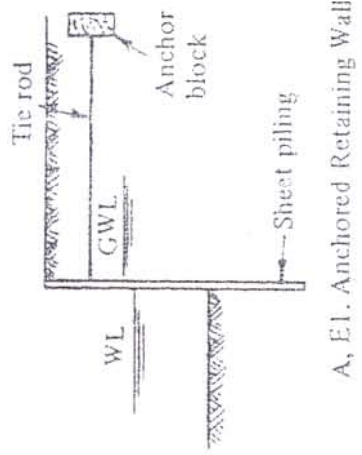
a)



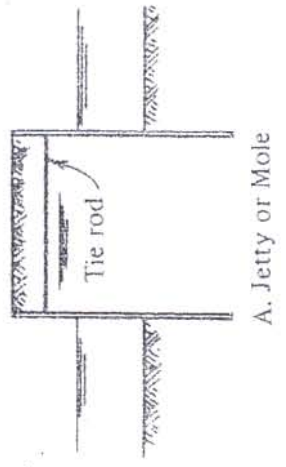
b)



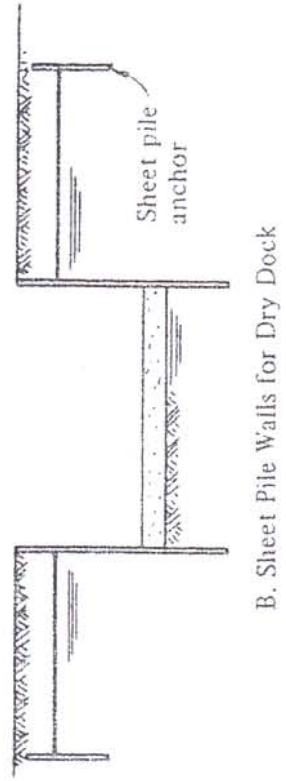
c)



A, E.L. Anchored Retaining Wall



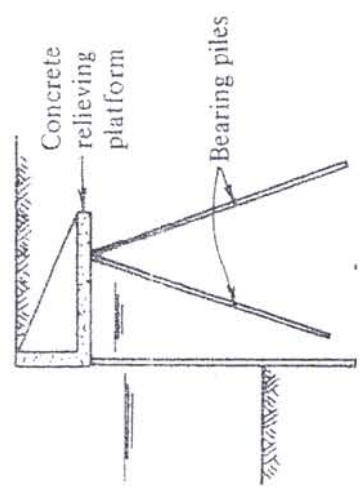
A. Jetty or Mole



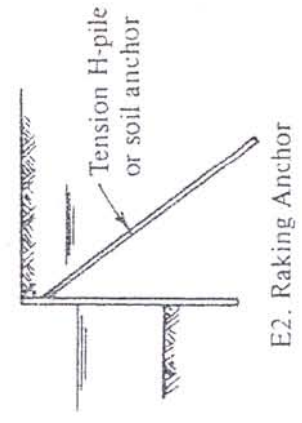
B. Sheet Pile Walls for Dry Dock



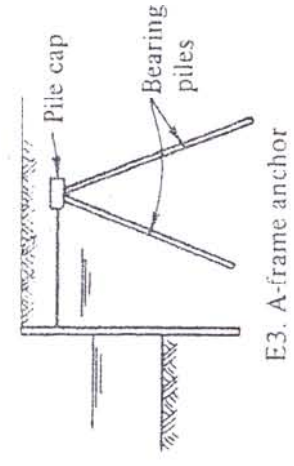
D. Cantilever Retaining Wall



F. Relieving-Platform Wall



E2. Raking Anchor



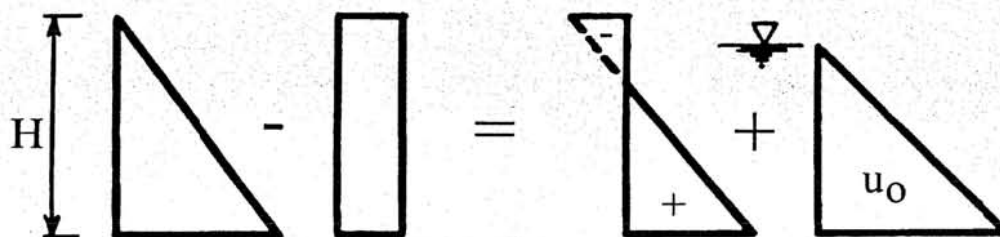
E3. A-frame anchor

Fig. 13.6 Types of permanent sheet pile structures (letters A, E1, etc., refer to Fig. 13.5).

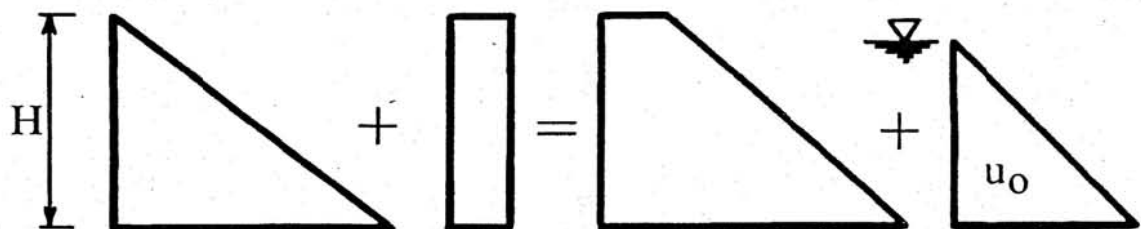
FORMULE DI RANKINE-BELL IN CONDIZIONI DRENATE

$$\tau_{ff} = \sigma'_{ff} \tan \varphi' + c'$$

$$\sigma'_a = \sigma'_{vo} \cdot K_A - 2c' \sqrt{K_A} \quad \text{SPINTA ATTIVA}$$



$$\sigma'_p = \sigma'_{vo} \cdot K_P + 2c' \sqrt{K_P} \quad \text{SPINTA PASSIVA}$$



FORMULE RIFERITE ALLE TENSIONI EFFICACI

ALLE SPINTE DEL TERRENO VA AGGIUNTA QUELLA DELL'ACQUA.

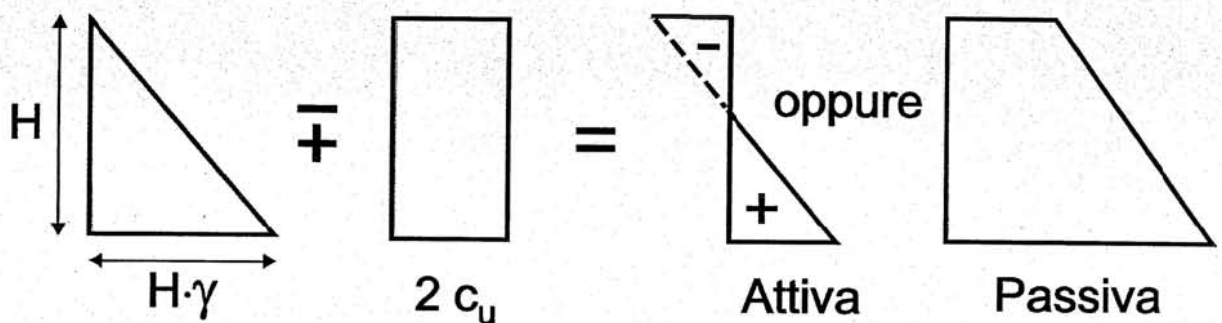
SPINTA ATTIVA E PASSIVA IN CONDIZIONI NON DRENATE

$$\varphi_u = 0^\circ \rightarrow \tau_f = c_u$$

$$\sigma_a = \sigma_{v0} - 2c_u \quad ; \quad \text{spinta attiva}$$

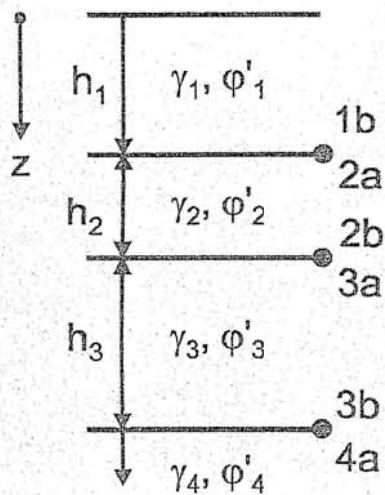
$$\sigma_p = \sigma_{v0} + 2c_u \quad ; \quad \text{spinta passiva}$$

$$\sigma_{v0} = \text{tensione verticale totale}$$



N.B. il contatto muro-terreno è privo della resistenza a trazione, trascurare nelle verifiche di stabilità la spinta attiva negativa.

SPINTA ATTIVA IN UN TERRENO STRATIFICATO



$$1b: \sigma_a = \gamma_1 \cdot h_1 \cdot K_a(\phi'_1)$$

$$2a: \sigma_a = \gamma_1 \cdot h_1 \cdot K_a(\phi'_2)$$

$$2b: \sigma_a = (\gamma_1 \cdot h_1 + \gamma_2 \cdot h_2) K_a(\phi'_2)$$

$$3a: \sigma_a = (\gamma_1 \cdot h_1 + \gamma_2 \cdot h_2) K_a(\phi'_3)$$

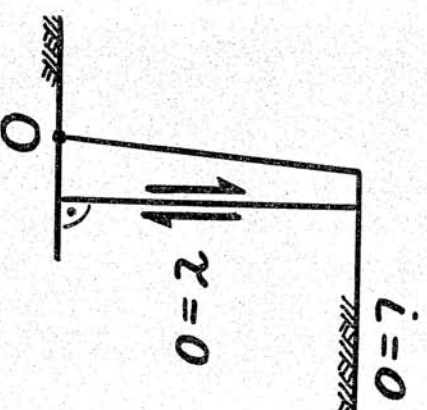
$$3b: \sigma_a = (\gamma_1 \cdot h_1 + \gamma_2 \cdot h_2 + \gamma_3 \cdot h_3) K_a(\phi'_3)$$

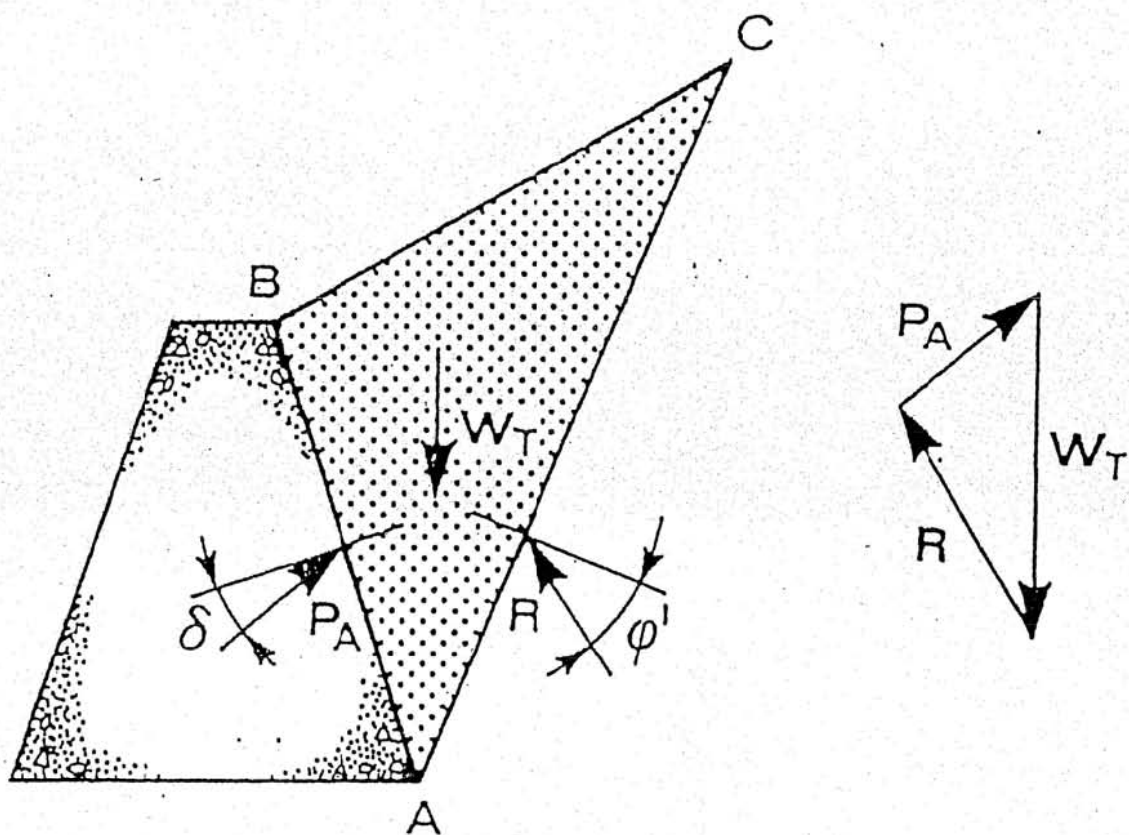
$$4a: \sigma_a = (\gamma_1 \cdot h_1 + \gamma_2 \cdot h_2 + \gamma_3 \cdot h_3) K_a(\phi'_4)$$

Per semplificare, si considera il terreno asciutto ($u=0$)

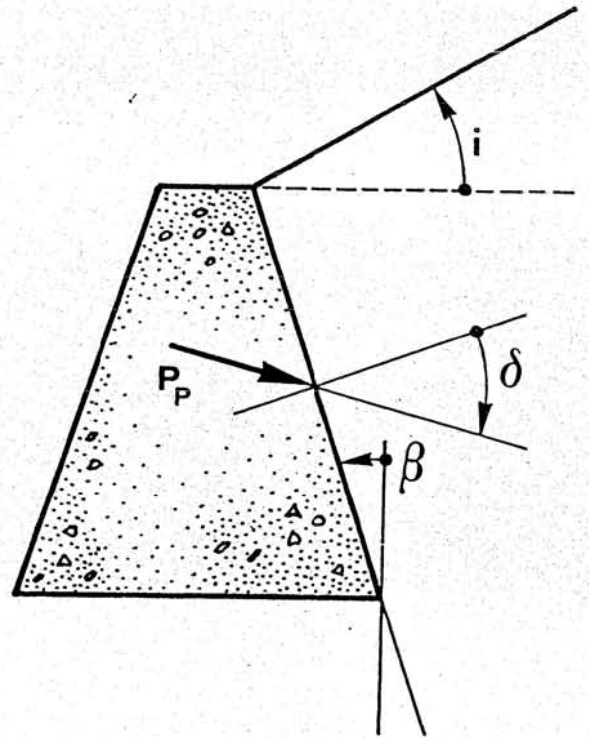
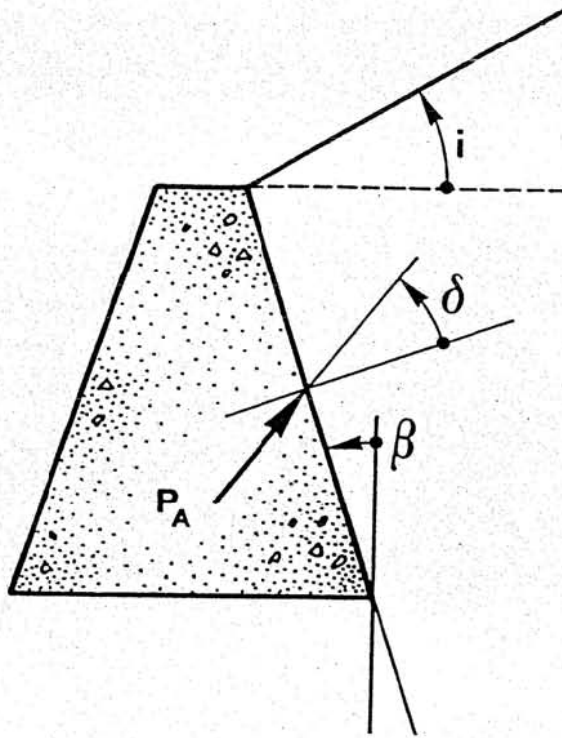
CONDIZIONI PER LA VALIDITA' DELLA TEORIA DI RANKINE

- 1) MURO LISCIO ($\tau = 0$)
- 2) PARAMENTO INTERNO VERTICALE ($\beta = 0$)
- 3) TERRAPIENO A TERGO DEL MURO ORIZZONTALE
($i = 0$)
- 4) MECCANISMO DI ROTTURA = ROTAZIONE ATTORNO
AL PUNTO O



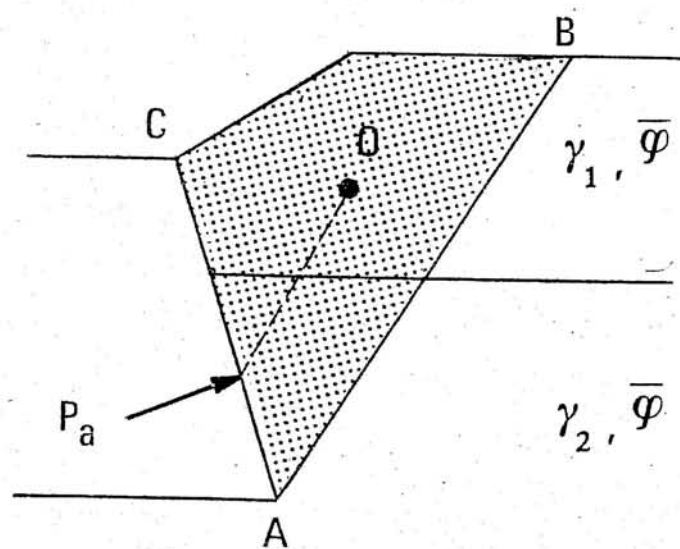
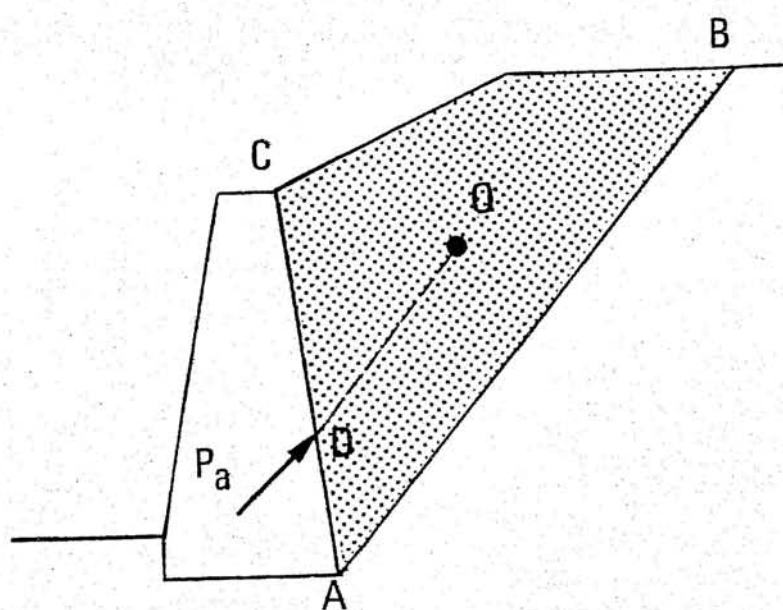


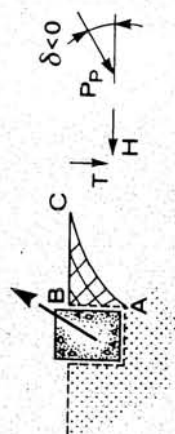
$$(\phi' > 0, c' = 0)$$



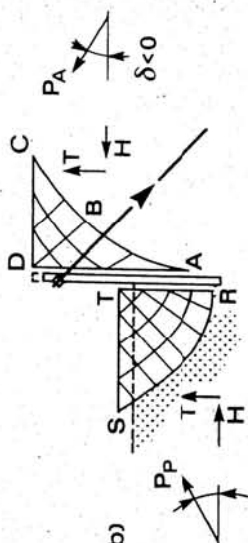
$$K_a = \frac{\cos^2(\bar{\phi} - \beta)}{\cos^2 \beta \cdot \cos(\beta + \delta) \left[1 + \sqrt{\frac{\sin(\delta + \bar{\phi}) \cdot \sin(\bar{\phi} - i)}{\cos(\beta + \delta) \cdot \cos(\beta - i)}} \right]^2};$$

$$K_p = \frac{\cos^2(\bar{\phi} + \beta)}{\cos^2 \beta \cdot \cos(\beta - \delta) \left[1 - \sqrt{\frac{\sin(\delta + \bar{\phi}) \cdot \sin(\bar{\phi} + i)}{\cos(\beta - \delta) \cdot \cos(\beta - i)}} \right]^2},$$

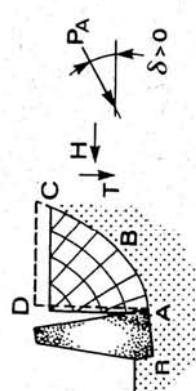




c)

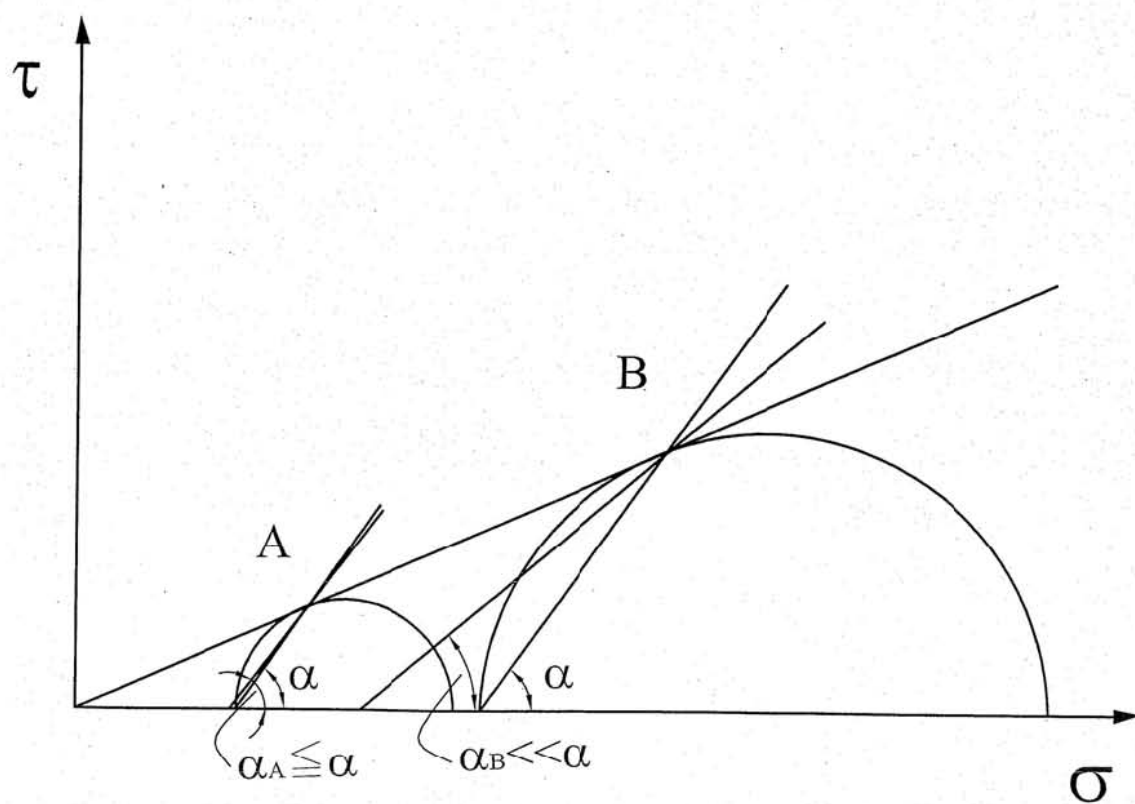
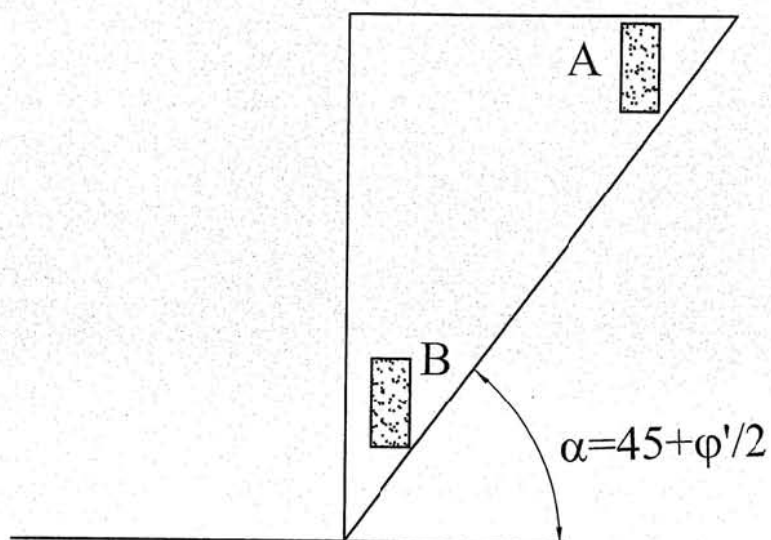


b)

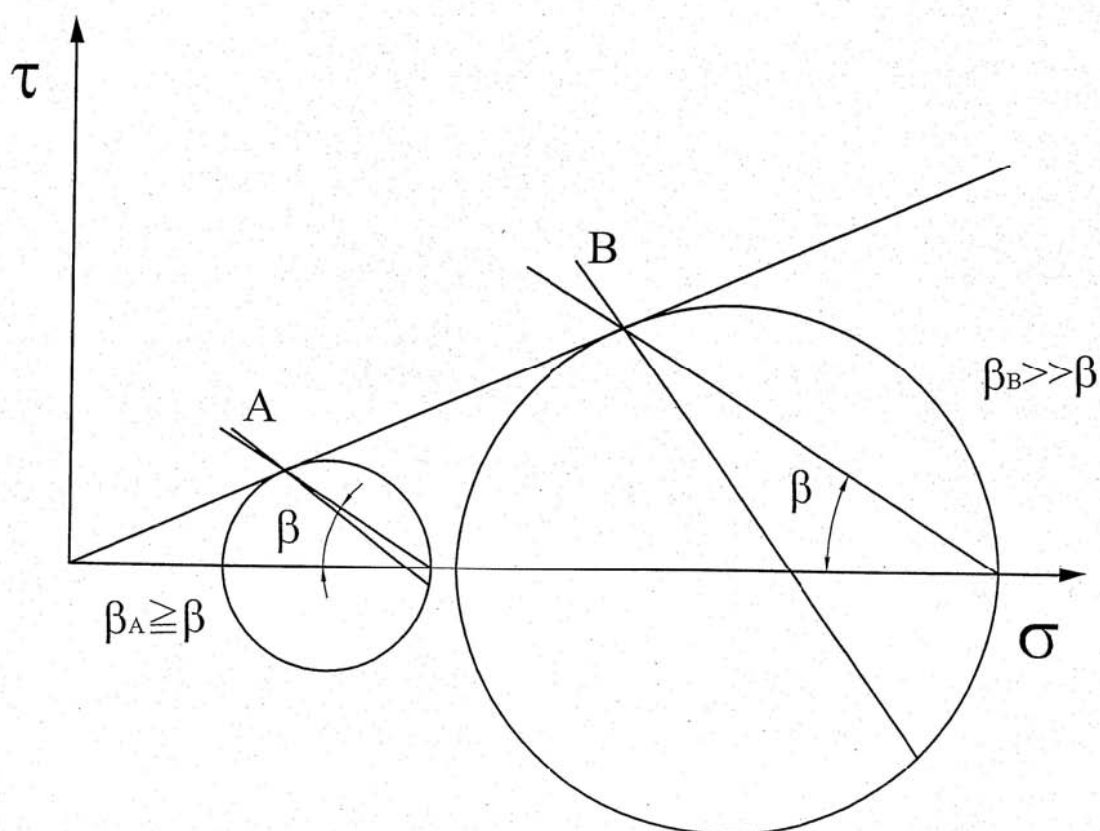
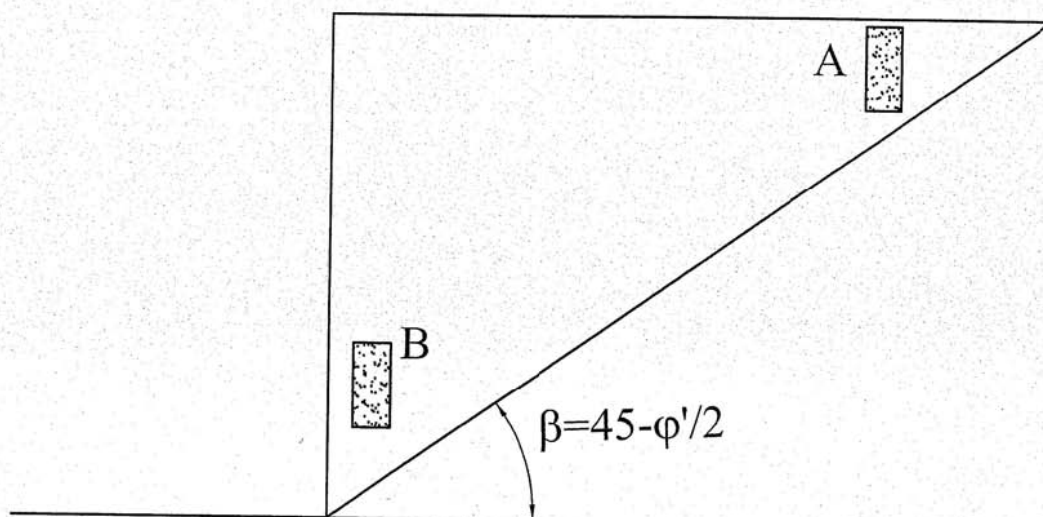


a)

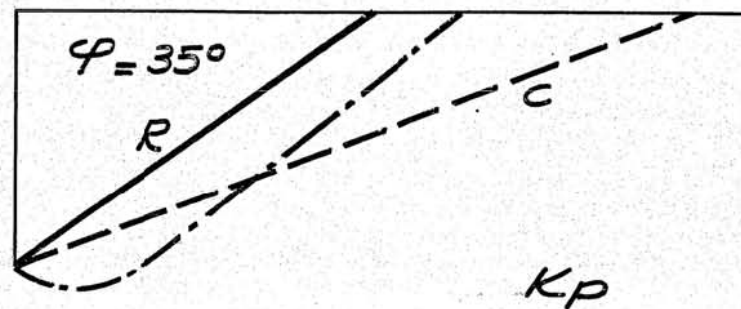
SPINTA ATTIVA



SPINTA PASSIVA

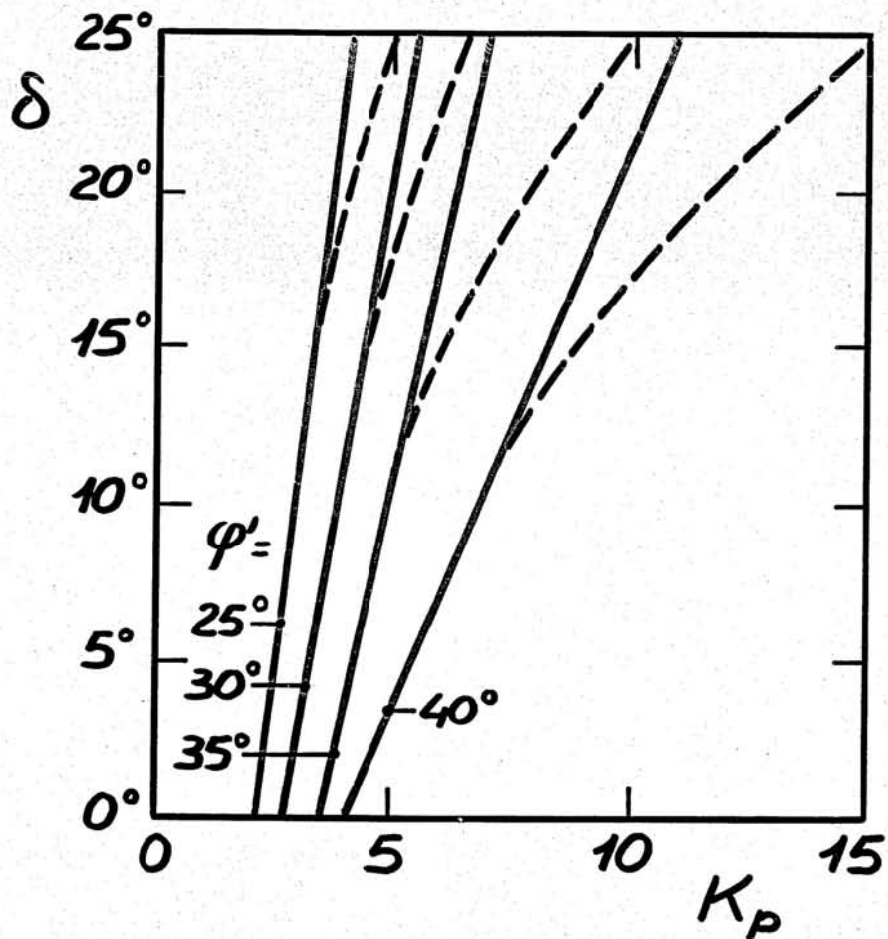


SPINTA PASSIVA CONFRONTO TRA RANKINE, COULOMB E SUPERFICIE DI ROTTURA CURVA



	$\delta = 0$	$\delta = \frac{1}{2} \bar{\varphi}$	$\delta = \bar{\varphi}$
RANKINE	3.69	-	-
COULOMB	3.69	7.02	18.82
(SUP. CURVA)	-	6.50	10.50

— SUPERFICIE DI SCORRIMENTO CURVA
--- SUPERFICIE DI SCORRIMENTO PIANA



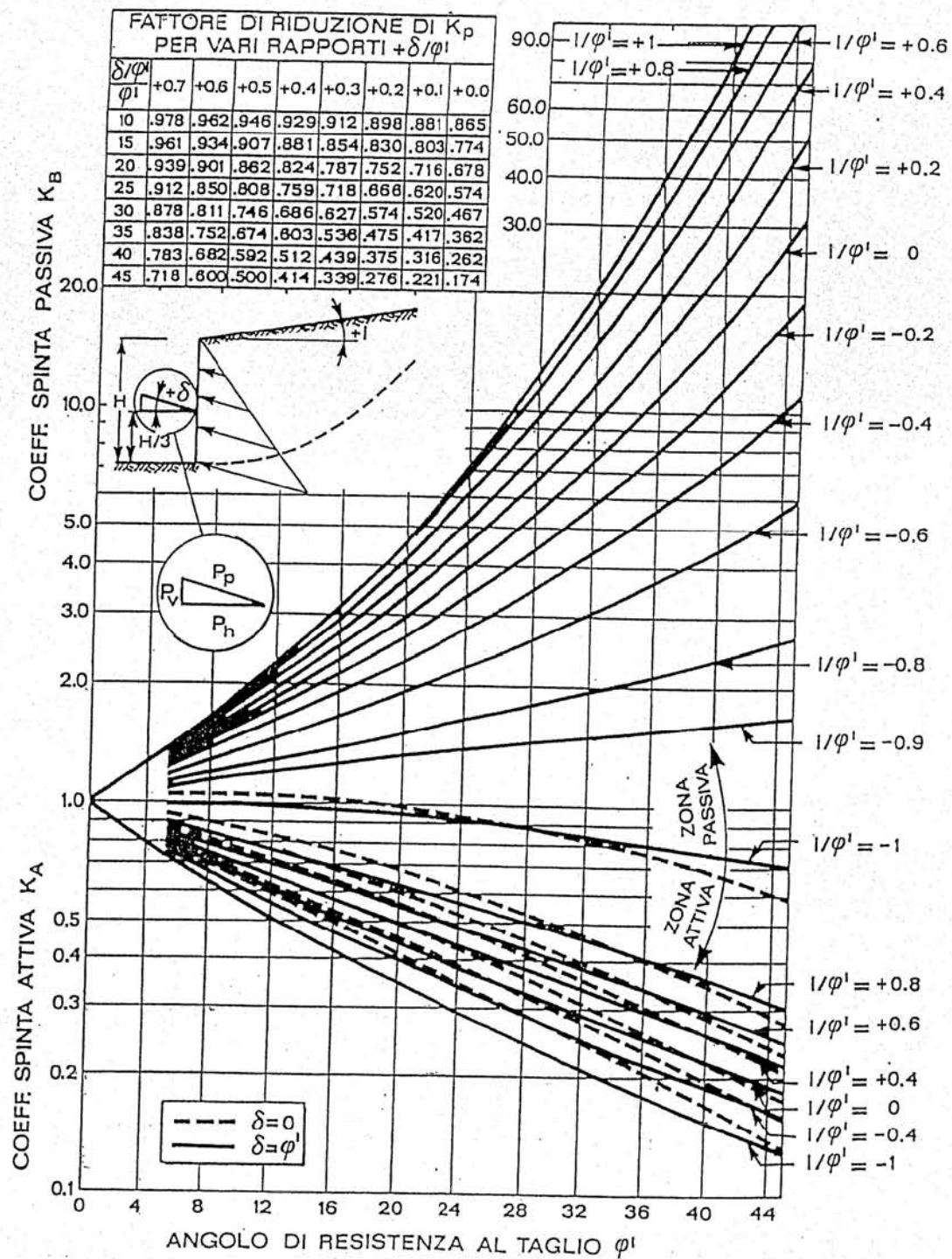


Tabella 13.2.2.

K_{ac} (Packshaw, 1946)				
	$c_a/\bar{c}$	1.0	0.5	1.0
	δ	0	$\bar{\phi}$	$\bar{\phi}$
ϕ	0	2.83	2.45	2.83
	5	2.60	2.10	2.47
	10	2.38	1.82	2.13
	15	2.16	1.55	1.85
	20	1.96	1.32	1.59
	25	1.76	1.15	1.41

K_{pc} (Packshaw, 1946)				
	$c_a/\bar{c}$	1.0	0.5	1.0
	δ	0	$\bar{\phi}$	$\bar{\phi}$
ϕ	0	2.6	2.4	2.6
	5	2.9	2.8	2.9
	10	3.2	3.3	3.4
	15	3.6	3.8	3.9
	20	4.0	4.5	4.7
	25	4.4	5.5	5.7

f) Nel caso in cui la verifica venga effettuata in condizioni NON DRENATE (ossia verifica a "breve scadenza", immediatamente dopo la esecuzione dello scavo e la costruzione dell'opera di sostegno), la determinazione delle spinte scaturisce direttamente dalla applicazione del criterio di rottura espresso in termini di TENSIONI TOTALI (si veda a tale proposito il cap. 6).

Con riferimento così alla figura 13.2.12, nell'ipotesi che l'aderenza c_a muro-terreno sia nulla, si ottiene:

$$\begin{aligned}\sigma_A &= \sigma_{v0} - 2 \cdot c_u \\ \sigma_P &= \sigma_{v0} + 2 \cdot c_u,\end{aligned}\quad (13.2.5)$$

nelle quali:

σ_{v0} = tensione verticale totale;

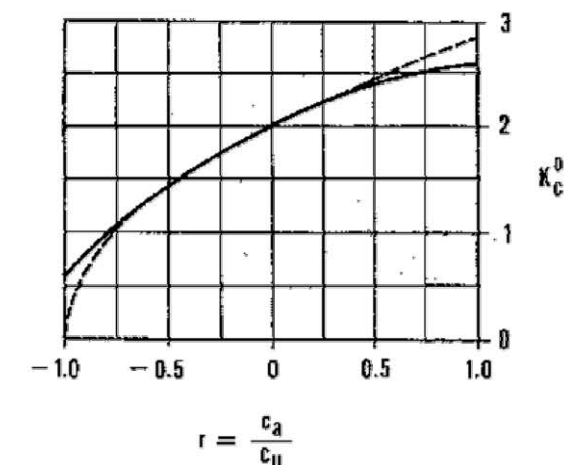
c_u = resistenza al taglio in termini di tensioni totali.

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(JANBU, 1972)

Fig. 13.2.14

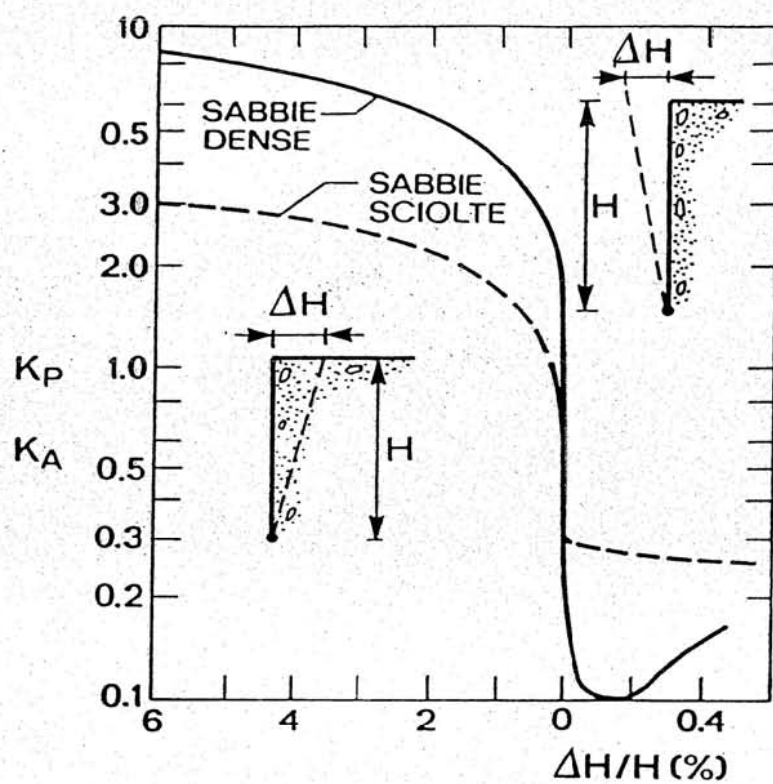
13.2.2 - E' già stato anticipato come l'entità e la distribuzione delle spinte dipendano largamente dallo spostamento relativo che il terreno può subire.

a) Nel caso ad esempio in cui le deformazioni siano impedita da una struttura che non ceda in alcun punto la pressione esercitata è nota come "spinta a riposo" ed è data da:

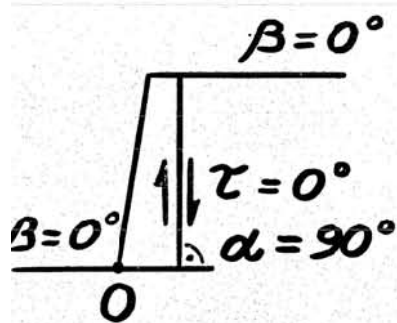
$$\bar{\sigma}_{h0} = \bar{\sigma}_{v0} \cdot K_0, \quad (13.2.17)$$

in cui K_0 è il coefficiente detto appunto di spinta a riposo.

Nel capitolo 2 si è detto diffusamente come tale coefficiente dipenda dalla natura del materiale, dalla "precisa" storia dello stato tensionale e dal tipo di fenomeno di sovraconsolidazione. Ad esempio, nel caso di un terreno normalconsolidato (fig. 13.2.15) esso è una costante (retta A-B) legata solo al tipo di materiale (e può perciò essere calcolato in funzione di

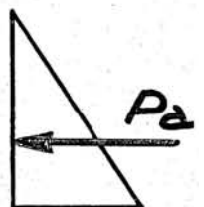
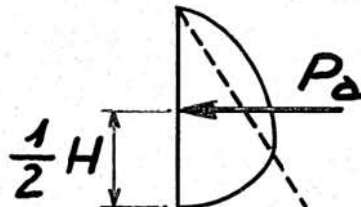
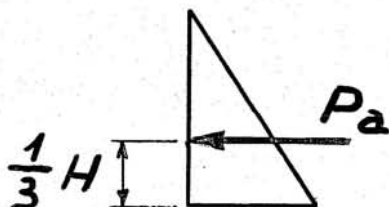
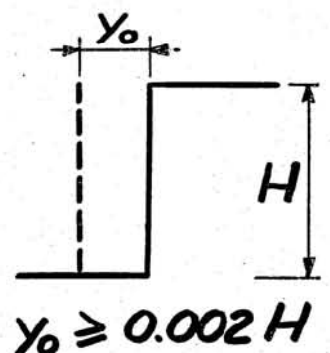
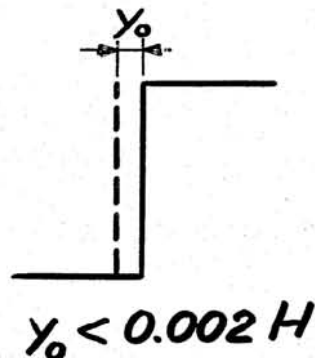
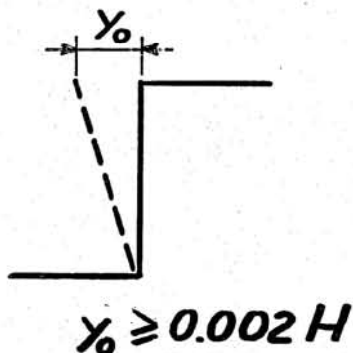


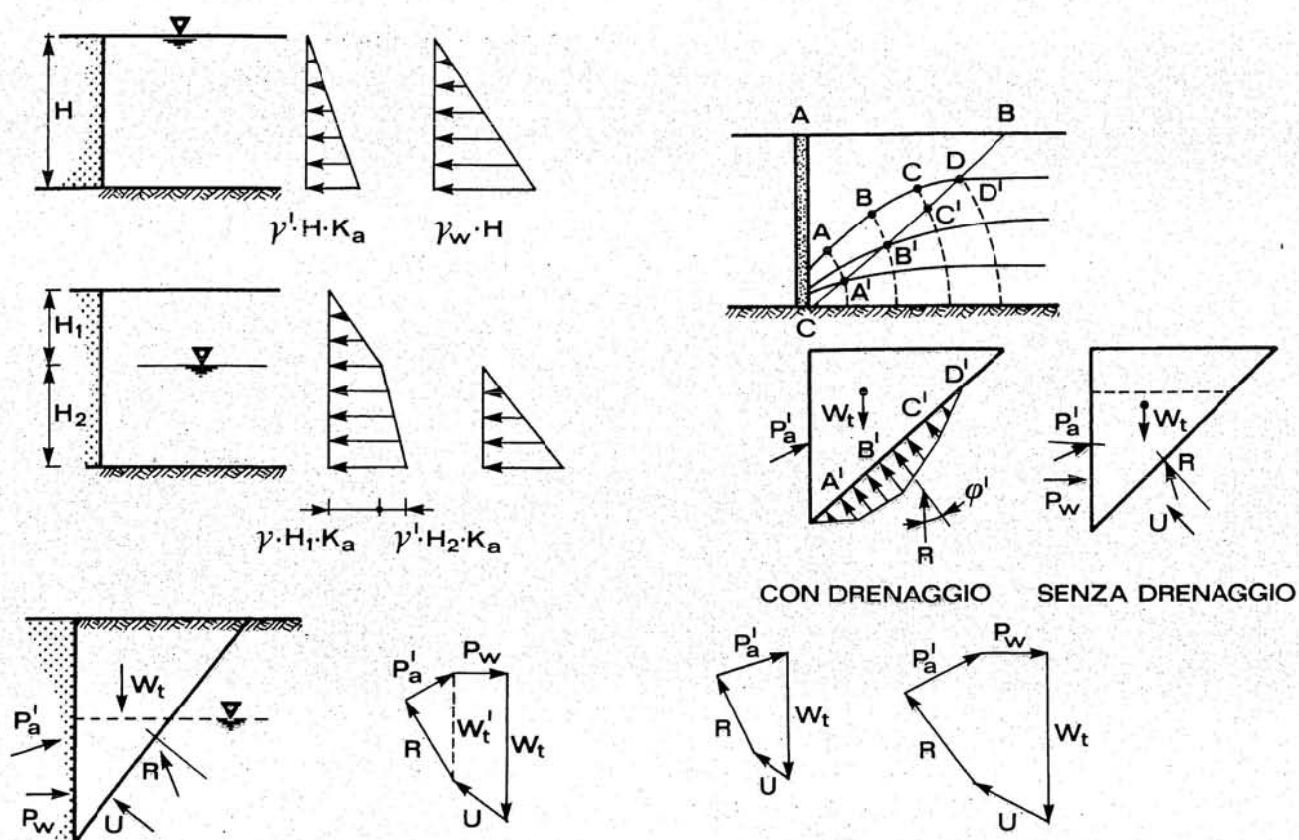
INFLUENZA DELLA CINEMATICA DELLA OPERA DI SOSTEGNO SULLA DISTRIBUZIONE DELLA SPINTA ATTIVA



1. OPERA DI SOSTEGNO LISCIA ($\tau = 0$)
2. MURO VERTICALE ($\alpha = 90^\circ$)
3. SUPERFICIE DI TERRENO ORIZZONTALE
4. IL MURO RUOTA ATTORNO AL PUNTO "O"

QUANDO QUEST'ULTIMA CONDIZIONE NON
E' SODDISFATTA IL DIAGRAMMA DELLE SPINTE
SUL PARAMENTO NON E' PIU' TRIANGOLARE



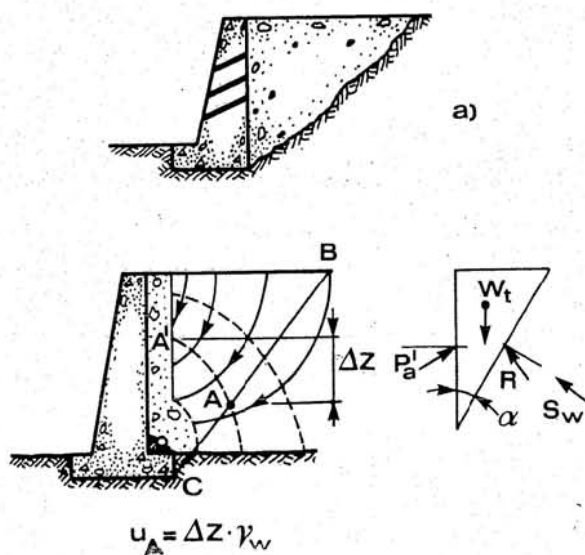


$$h_A = h_{A'}$$

si ha pure:

$$u_{A'} = u_A + \gamma_w(z_A - z_{A'})$$

ed essendo $u_A = 0$, risulta $u_{A'} = \gamma_w(z_A - z_{A'})$.



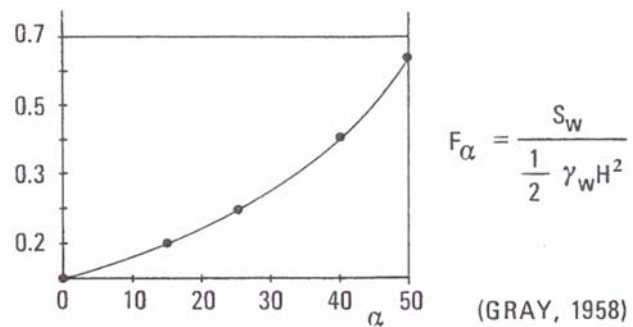
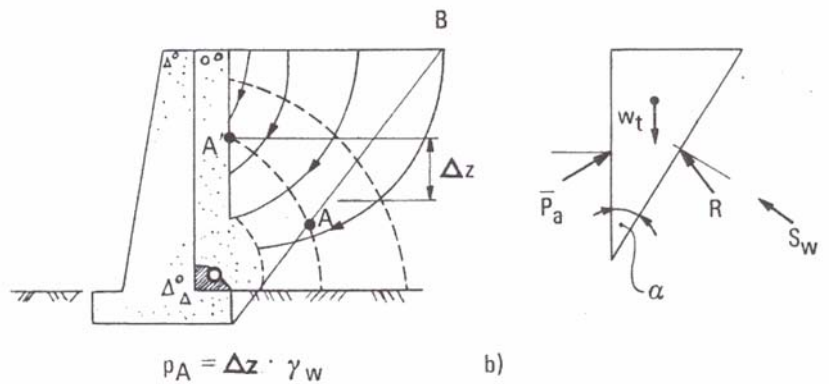
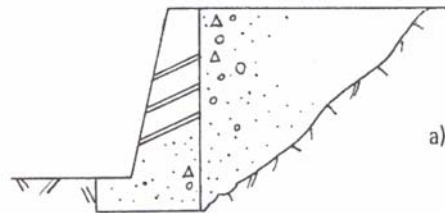
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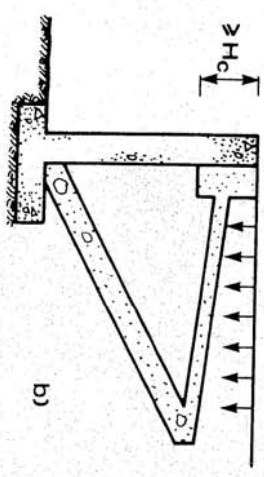
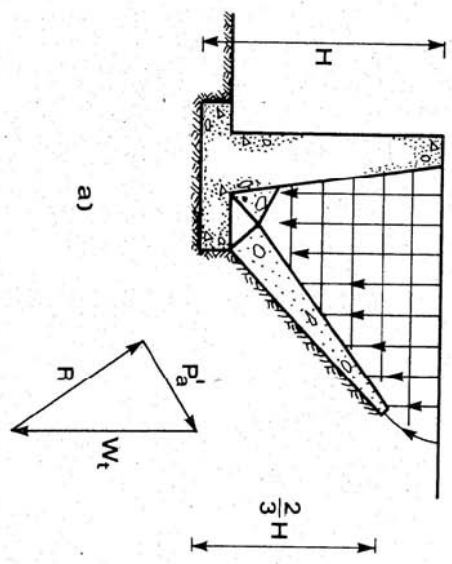
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$$F_\alpha = \frac{S_w}{\frac{1}{2} \gamma_w H^2}$$

(GRAY, 1958)

Fig. 13.4.3



SPINTE DEL TERRENO DOVUTE A SOVRACCARICHI

1. FORMULE DELLA TEORIA DELL'ELASTICITA'

a. VALUTAZIONE DIRETTA DI $\Delta\sigma_h$

- Formule di Boussinesq e Mindlin
- Applicabile solo al calcolo di $\Delta\sigma_a$
- Nel caso delle opere di sostegno rigide,
 $\Delta\sigma_a = 2 \Delta\sigma_h$
- $\nu' = 0.2 \div 0.3$; $\nu_u = 0.5$

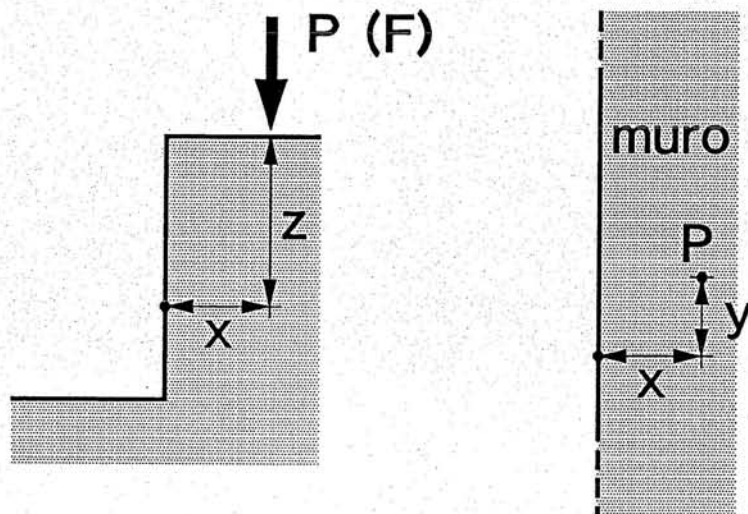
b. VALUTAZIONE INDIRETTA DI $\Delta\sigma_h$

- Le stesse formule del caso 1a)
- $\Delta\sigma_a = K_a \Delta\sigma_v$ e $\Delta\sigma_p = K_p \Delta\sigma_v$

2. FORMULE EMPIRICHE

- Da prove su modelli
- Da misure sulle opere in scala reale

CARICO PUNTIFORME



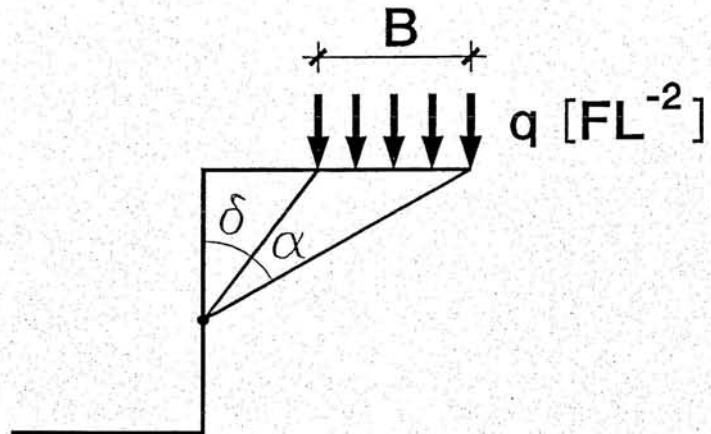
$$\Delta \sigma_h = \frac{P}{2 \pi R^2} \left[\frac{3 r^2}{R^3} - \frac{(1 - \nu) R}{R + z} \right]$$

$$\Delta \sigma_v = \frac{3 P z^3}{2 \pi R^5}$$

$$r = (x^2 + y^2)^{0.5}$$

$$R = (x^2 + y^2 + z^2)^{0.5}$$

CARICO NASTRIFORME PARALLELO AL MURO



$$\Delta \sigma_h = \frac{q}{\pi} [\alpha - \sin \alpha \cdot \cos (2\delta + \alpha)]$$

$$\Delta \sigma_v = \frac{q}{\pi} [\alpha + \sin \alpha \cdot \cos (2\delta + \alpha)]$$

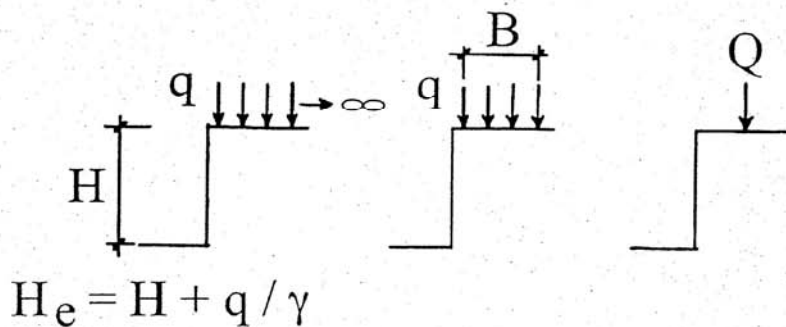
SPINTE DEL TERRENO - FORMULE GENERALI

$$\sigma'_a = K_a \cdot \sigma'_{vo} - K_{ac} \cdot c' + K_{aq} \cdot q \quad [\text{FL}^{-2}]$$

$$\sigma'_p = K_p \cdot \sigma'_{vo} + K_{pc} \cdot c' + K_{pq} \cdot q \quad [\text{FL}^{-2}]$$

$$\left. \begin{matrix} K_a \\ K_p \end{matrix} \right\} = f(\varphi', \delta); \quad \left. \begin{matrix} K_{ac} \\ K_{pc} \end{matrix} \right\} = f(\varphi', \delta, c'_\alpha); \quad \left. \begin{matrix} K_{aq} \\ K_{pq} \end{matrix} \right\} = f(\varphi, \text{tipo di sovracc.})$$

$$\left. \begin{matrix} \varphi' = 0 \\ c' = c_u \end{matrix} \right\} K_a = K_p = 1 \quad K_c = 2\sqrt{1 + c'_\alpha / c_u}$$



TIPI DI SOVRACCARICO

COEFFICIENTI DI SPINTA ATTIVA E PASSIVA TERRENI DOTATI DI COESIONE TENSIONI EFFICACI

		δ'/φ'	c'_a/c	φ'		
				15°	20°	25°
K_a	0	NI*	0.59	0.48	0.40	
	1		0.50	0.40	0.32	
K_{ac}	0	0	1.54	1.40	1.29	
	0	1	2.16	1.96	1.76	
	1	0.5	1.55	1.32	1.15	
	1	1	1.85	1.59	1.41	
K_p	0	NI*	1.70	2.10	2.50	
	1		2.20	2.90	3.90	
K_{pc}	0	0	2.60	2.80	3.10	
	0	0.5	3.20	3.50	3.80	
	0	1	3.60	4.00	4.40	
	0	0.5	3.80	4.50	5.50	
	1	1	3.90	4.70	5.70	

NI = non
influyente

$$\left. \begin{matrix} \sigma'_a \\ \sigma'_p \end{matrix} \right\} = K_p^a \cdot \sigma'_{vo} \mp K_{cp}^a \cdot c'$$

COMMENTI DI CARATTERE PRATICO

1. Le opere di sostegno non sono mai perfettamente lisce.

a. CALCESTRUZZO

$$\frac{1}{2} \leq \frac{\delta}{\varphi'} \leq \frac{2}{3} \quad ; \quad \frac{1}{3} \leq \frac{c_a'}{c'} \leq \frac{1}{2}$$

b. ACCIAIO

$$\frac{1}{3} \leq \frac{\delta}{\varphi'} \leq \frac{1}{2} \quad ; \quad \frac{1}{3} \leq \frac{c_a'}{c'} \leq \frac{1}{2}$$

COMMENTI DI CARATTERE PRATICO

2. La mobilitazione delle spinte attive e passive avviene dopo che l'opera di sostegno ha subito spostamenti e rotazioni di una certa entità.

a. **SPINTA ATTIVA:**

$$\frac{\Delta y(z = 0)}{H} \leq 0.002$$

b. **SPINTA PASSIVA:**

$$0.03 \leq \frac{\Delta y(z = 0)}{H} \leq 0.06$$

COMMENTI DI CARATTERE PRATICO

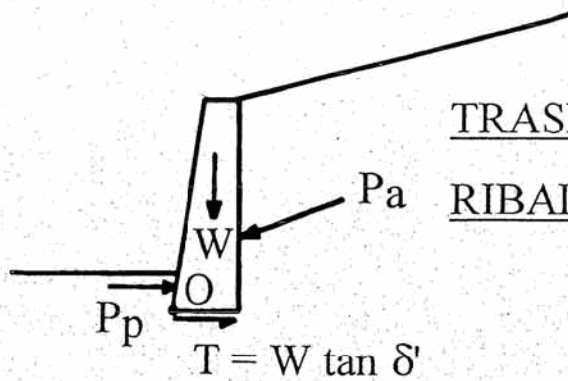
3. La distribuzione delle spinte e il punto di applicazione della loro risultante dipende da:
 - a. Caratteristiche degli spostamenti e delle rotazioni subite dall'opera di sostegno.
 - b. Rigidezza flessionale dell'opera stessa.

CALCOLO DELLE SPINTE

Conclusioni

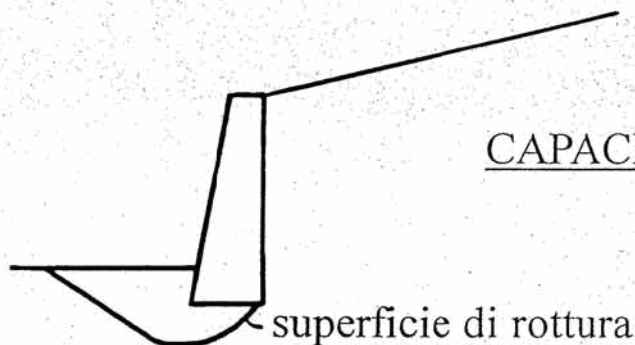
1. Azioni: terreno + acqua + sovraccarichi + sisma
2. Spinta attiva: utilizzare le soluzioni con la superficie di scorrimento piana:
 - Rankine ($i = 0$, $\delta = 0^\circ$, $c'_a = 0$)
 - Coulomb ($i \neq 0$, $\delta \neq 0^\circ$, $c'_a \neq 0$)
3. Spinta passiva: quando $\delta > 1/3 \varphi'$ utilizzare le soluzioni con la superficie di scorrimento curvilinea.
4. Spostamenti necessari per mobilitare la spinta:
 - attiva, molto modesti
 - passiva, rilevanti

VERIFICHE DI STABILITA'

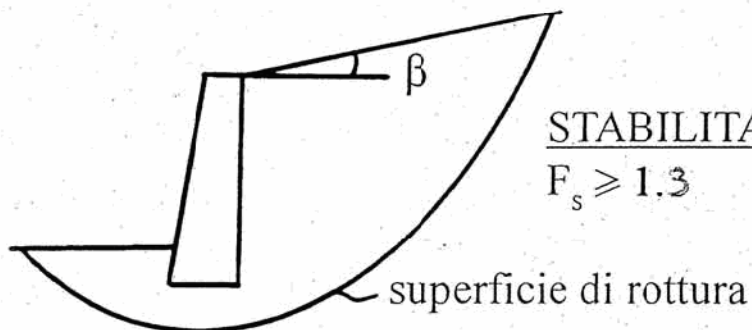


TRASLAZIONE ($\Sigma T = 0$): $F_s \geq 1.3$

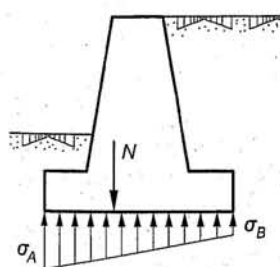
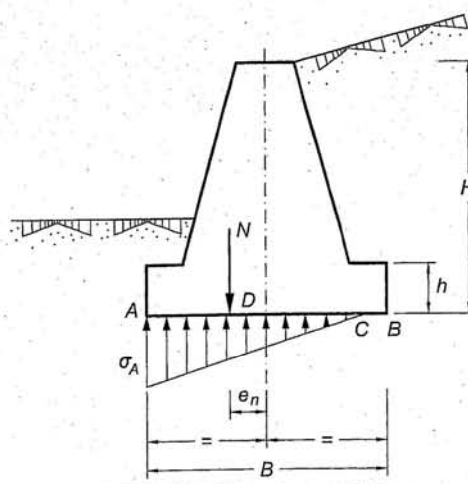
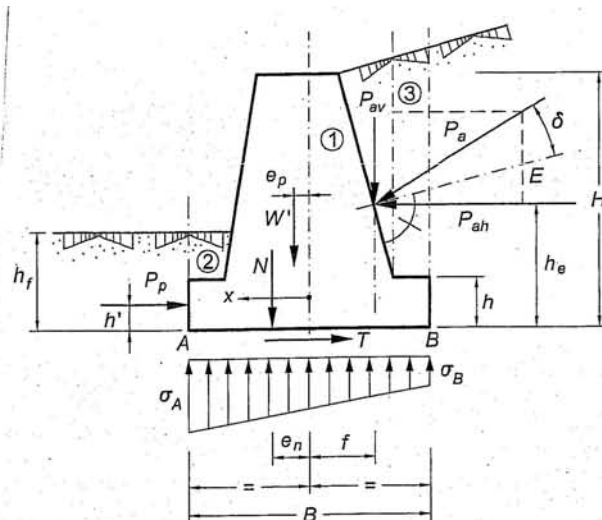
RIBALTAMENTO ($\Sigma M_o = 0$): $F_s \geq 1.5$



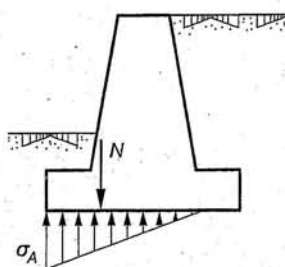
CAPACITA' PORTANTE: $F_s \geq 2$



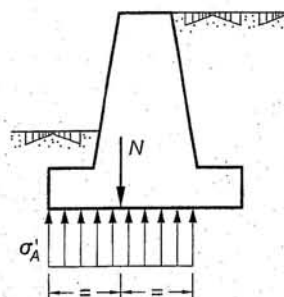
STABILITA' GLOBALE ($\beta \geq 0$)
 $F_s \geq 1.3$



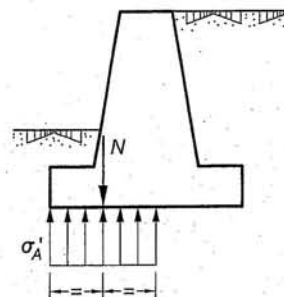
(a)



(b)

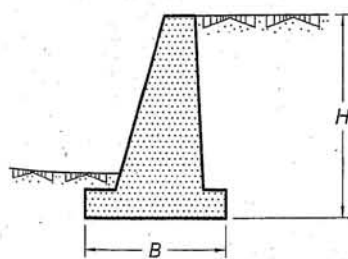


(a')

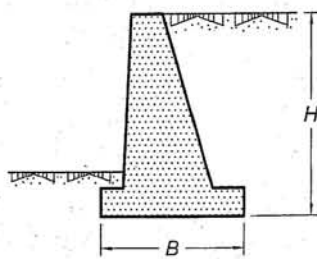


(b')

Figura 4.44 Reazioni in fase di esercizio e allo stato limite ultimo.



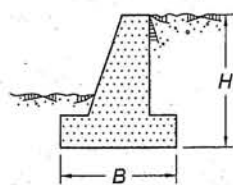
Tipo A



Tipo B

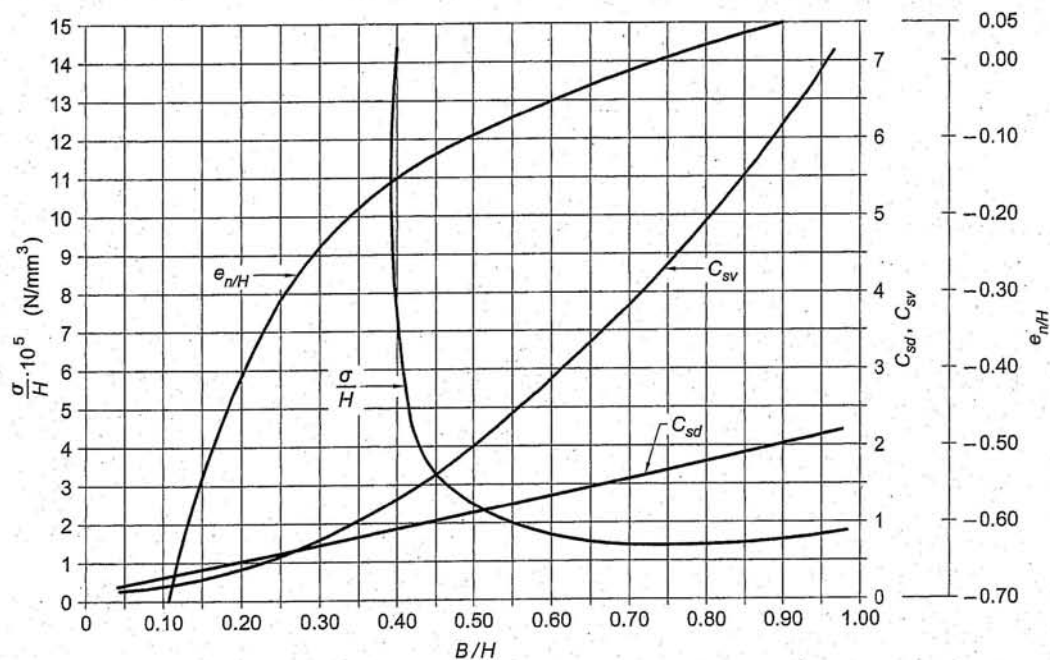
Figura 4.45 Tipologie considerate negli abachi di predimensionamento.

C1 - ABACO DI PREDIMENSIONAMENTO DI MURI A GRAVITÀ

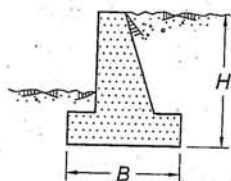


Muro di tipo A

σ	(N/mm ²)
H	(mm)
B	(mm)

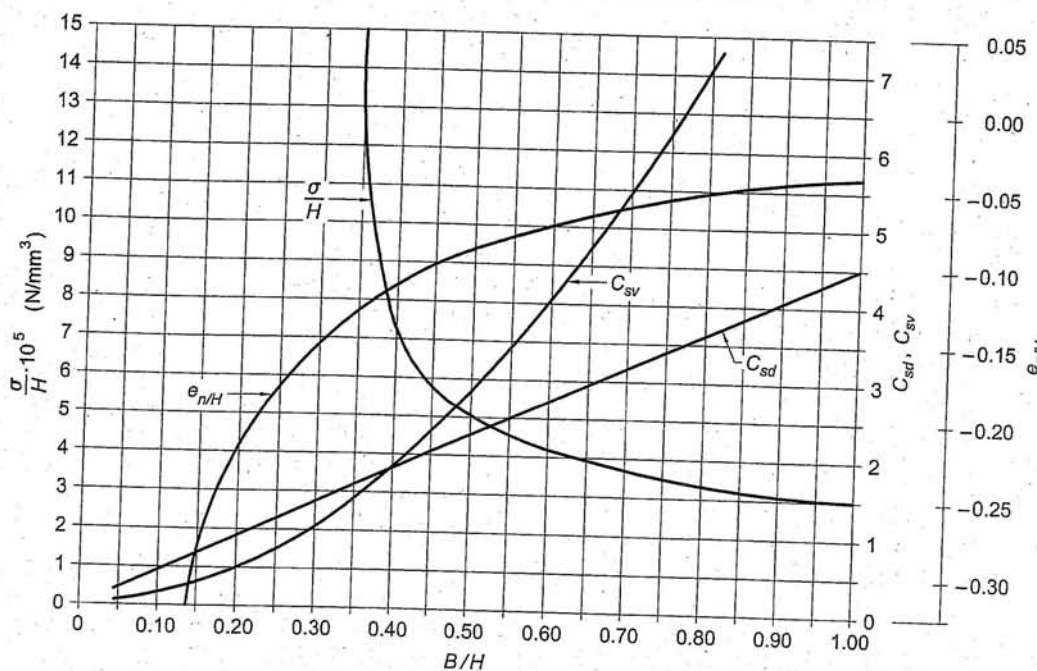


C2 - ABACO DI PREDIMENSIONAMENTO DI MURI A GRAVITÀ



Muro di tipo B

σ	(N/mm ²)
H	(mm)
B	(mm)



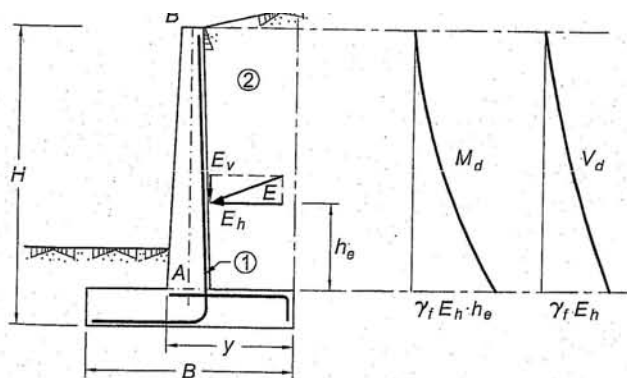


Figura 4.51 Diagrammi di momento e taglio relativi alla mensola verticale.

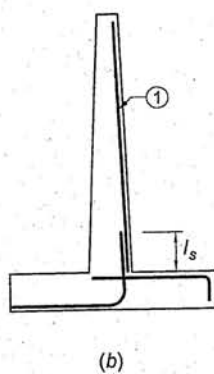
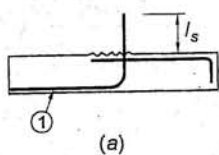
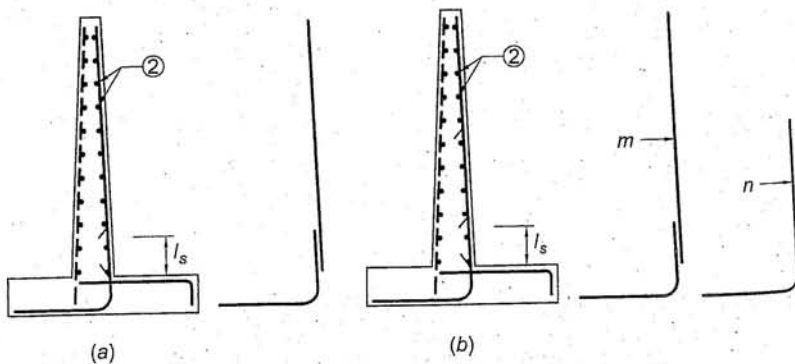
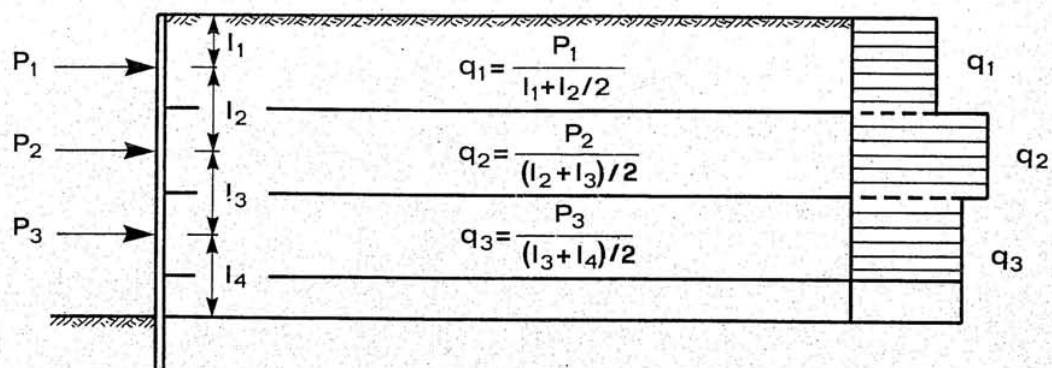
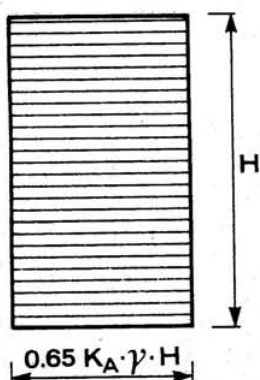


Figura 4.52 Armatura principale di un muro a mensola.



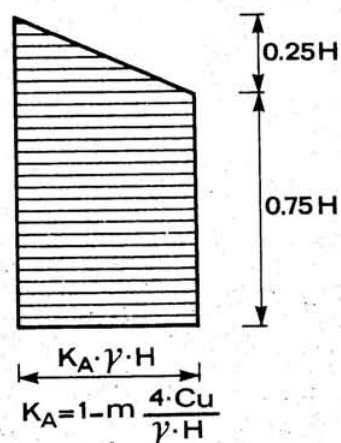


SABBIE



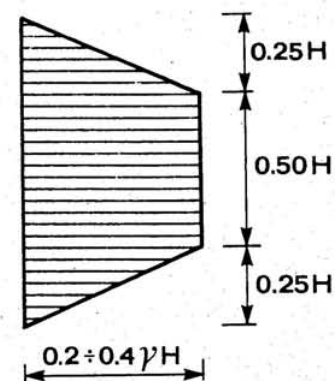
a)

ARGILLA NC



b)

ARGILLA OC



c)

$$N_o = \frac{\gamma H}{C_u}$$

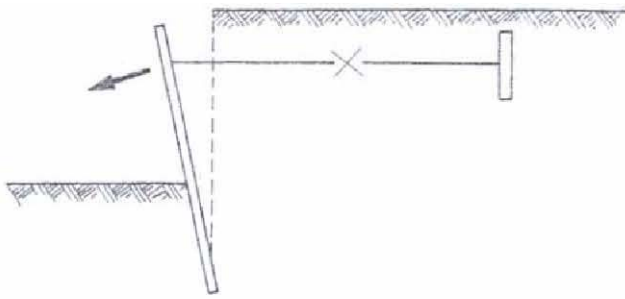
$$N_o = 6 \div 8$$

$$m = 0.4$$

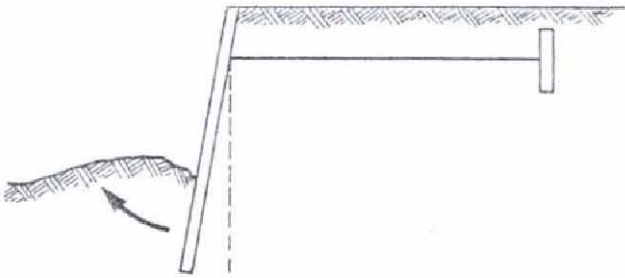
$$N_o = 4 \div 6$$

$$m = 1$$

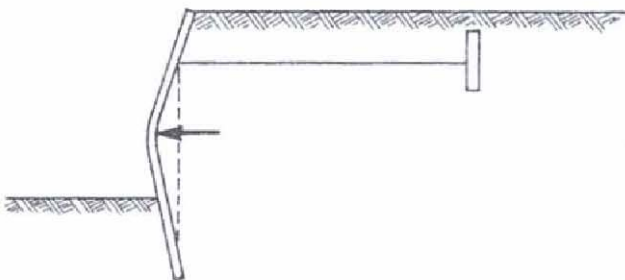
$$N_o = 3 \div 4$$



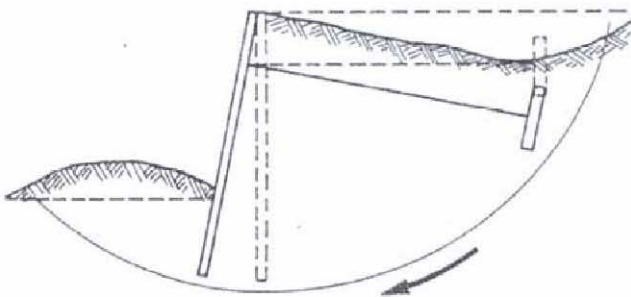
(1) Failure of anchorage system



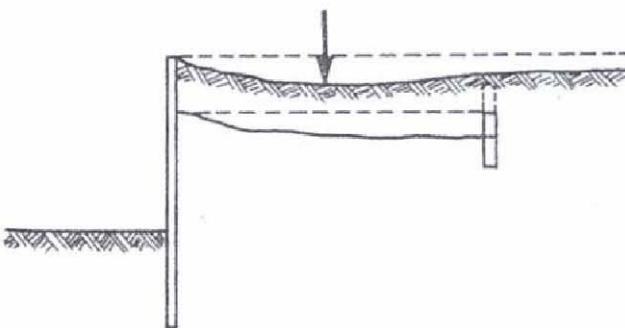
(2) Bottom of piles move outward



(3) Failure of piling in bending



(4) Circular slip in soft clay



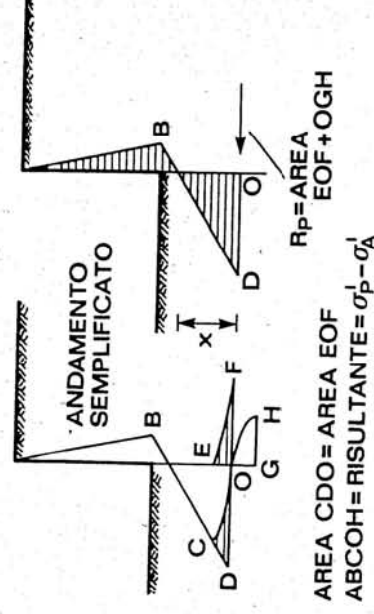
(5) Settlement behind wall

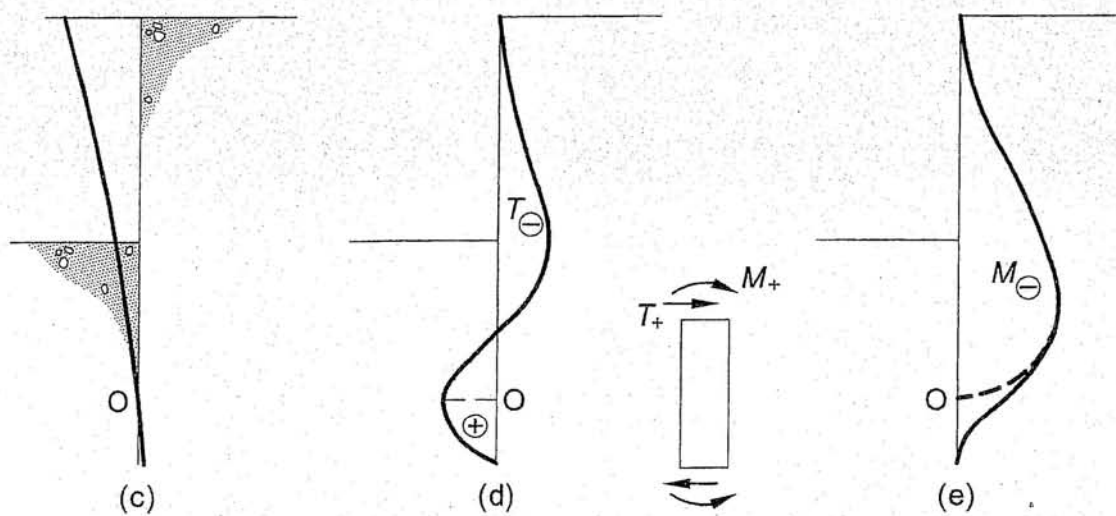
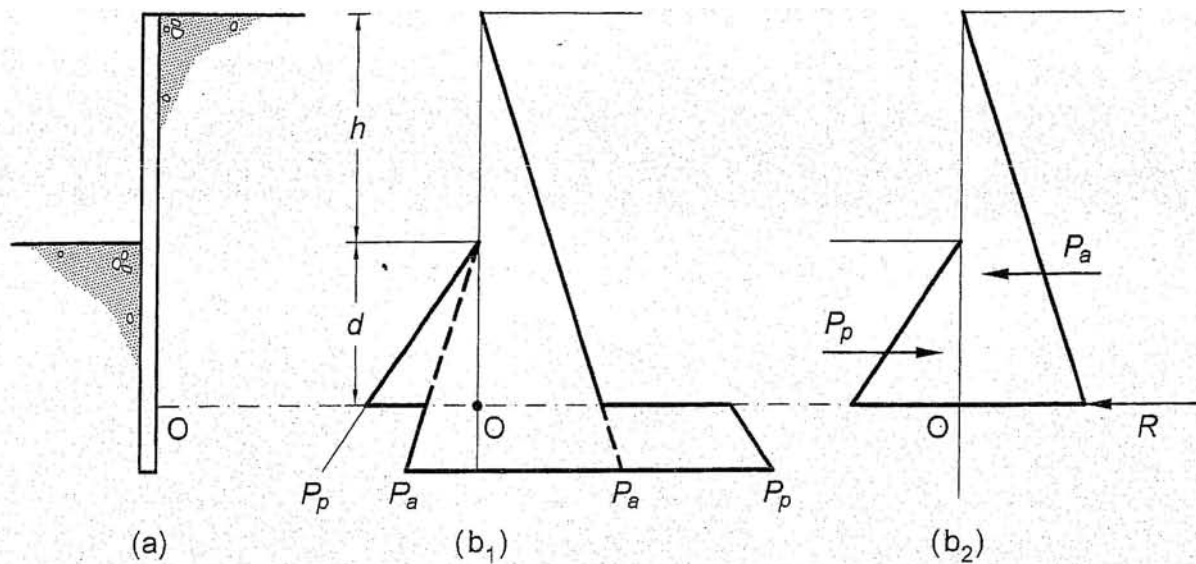
Fig. 13.11 Methods of failure of anchored sheet pile walls.

OPERE DI SOSTEGNO FLESSIBILI

DIAFRAMMI A SBALZO

SI INCREMENTA LA LUNGHEZZA DI INFISSIONE DEL 20 %.
COEFFICIENTE DI SICUREZZA SPINTA PASSIVA PARI A 2.

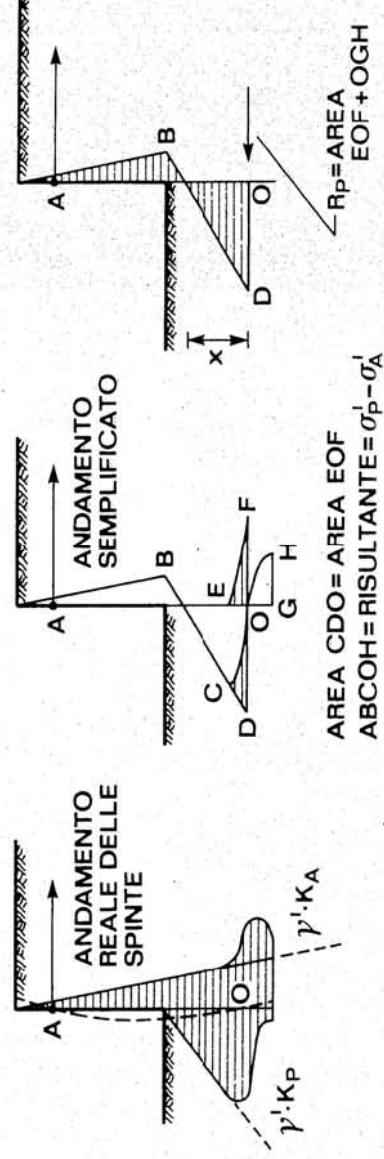




OPERE DI SOSTEGNO FLESSIBILI

DIAFRAMMI MOLTO PROFONDI (FIXED EARTH SUPPORT)
(MOMENTO Nullo IN O, SPOSTAMENTO Nullo IN A;
OPPURE TRAVE APPOGGIATA IN A ED O, ROTAZIONE
Nulla IN O)

EQUAZIONE DI QUINTO GRADO IN x . SI INCREMENTA LA
LUNGHEZZA DEL TRATTO INFLESSO DEL 20 %. FATTORE DI
SICUREZZA SPINTA PASSIVA PARI A 1.

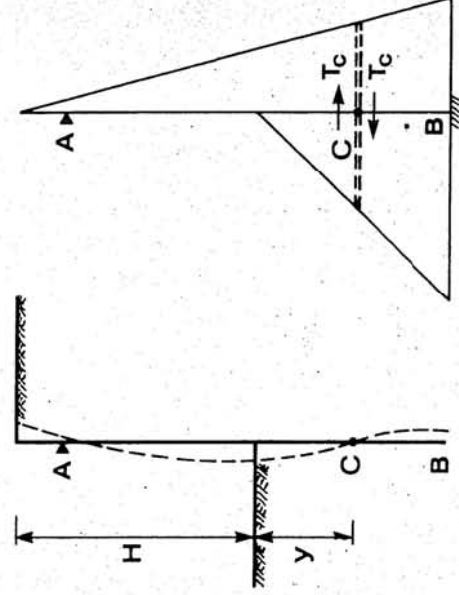


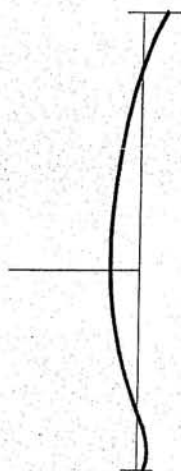
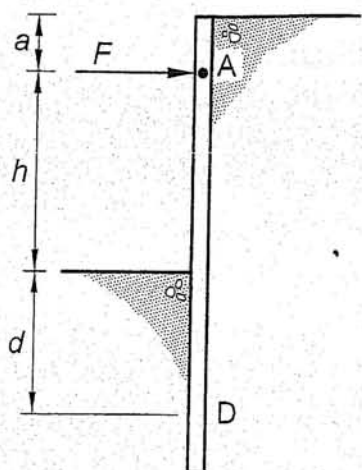
OPERE DI SOSTEGNO FLESSIBILI

DIAFRAMMI MOLTO PROFONDI (FIXED EARTH SUPPORT)

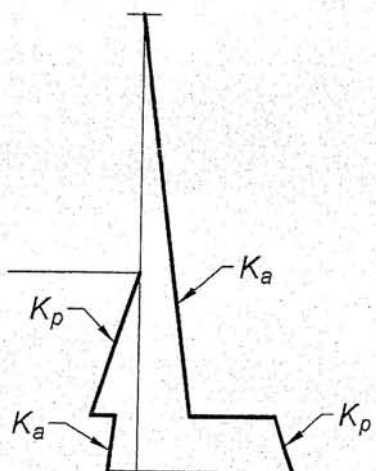
CONOSCENDO LA POSIZIONE DELLA CERNIERA SI PUO' SCOMPORRE LA STRUTTURA IN DUE TRAVI AC E CB. EQUILIBRIO ALLA TRASLAZIONE DI AC SI RICAVALA T E T_c . EQUILIBRIO ALLA ROTAZIONE INTORNO A B SI RICAVALA IL TRATTO DI INFISSIONE

ϕ'	20°	30°	40°
y/H	0.25	0.08	-0.007

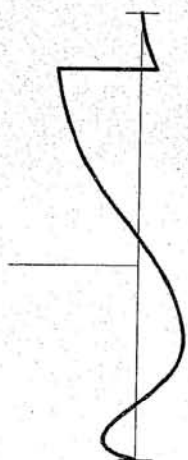




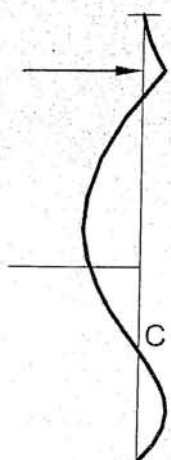
(a) Deformata



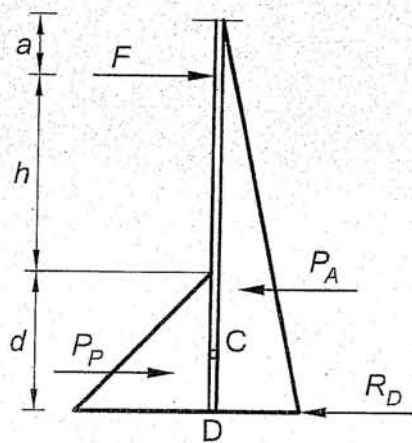
(b) Spinte



(c) Taglio



(d) Momento



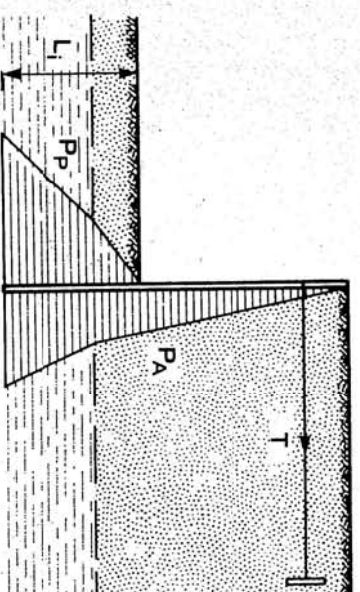
(e) Trave equivalente

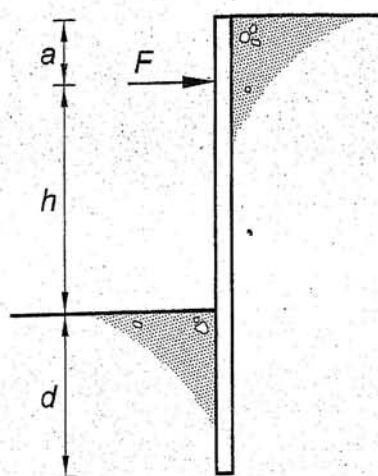
OPERE DI SOSTEGNO FLESSIBILI

DETERMINAZIONE DEL TIRO (T) E DELLA PROFONDITA' DI INFISSIONE (L)

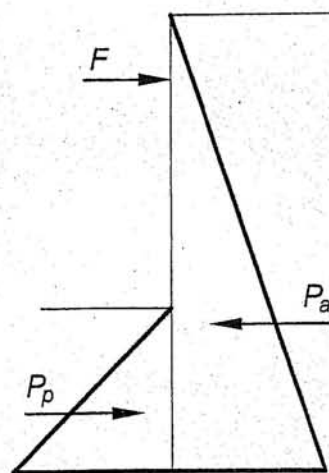
DIAFRAMMI POCO PROFONDI (FREE EARTH SUPPORT)

FATTORE DI SICUREZZA SPINTA PASSIVA PARI A 2

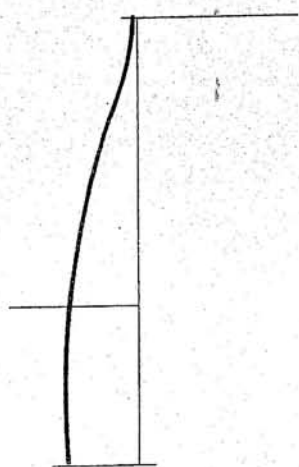




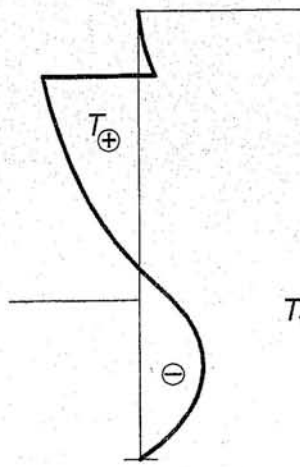
(a)



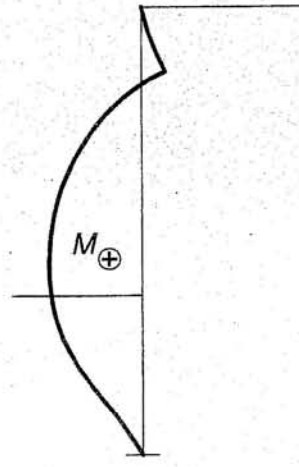
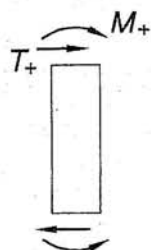
(b) Spinte



(c) Deformata



(d) Taglio



(e) Momento

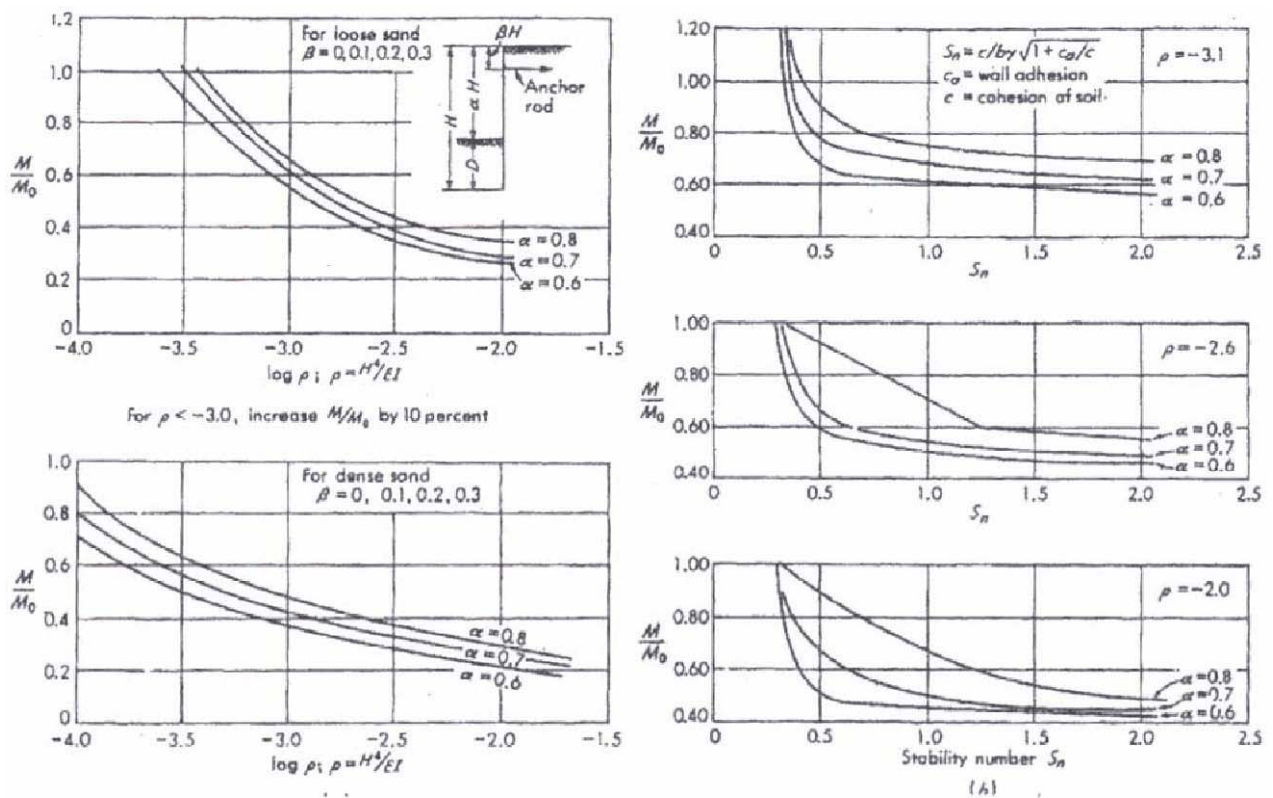
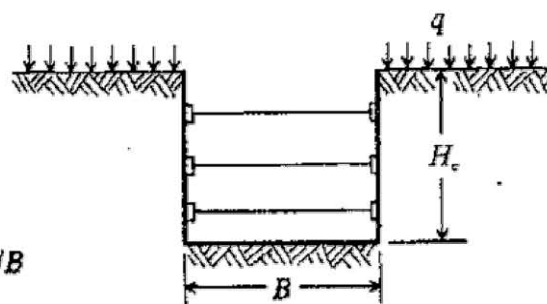
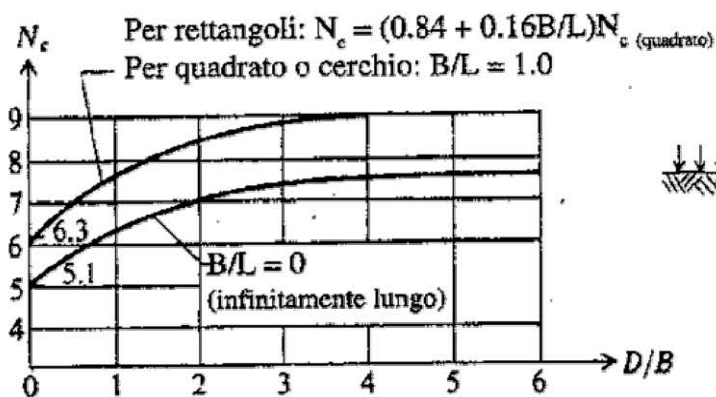
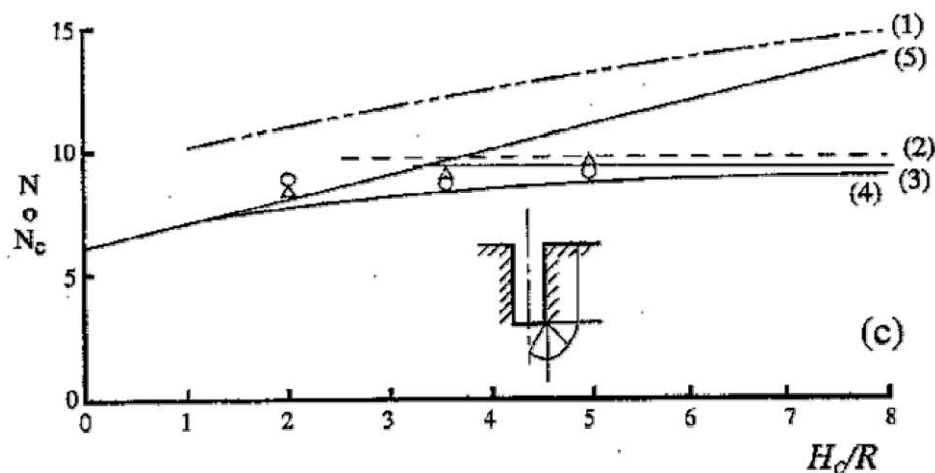


Fig. 3.3: Coefficienti di riduzione del momento flettente massimo proposti da Rowe (1952, 1957). a) terreni non coesivi; b) terreni coesivi (da Bowles 1982).



(b)

(a)



(1) Soluzione estremo superiore

(4) Skempton (1951)

(2) Gibson (1950) $E_u = 200 C_u$

(5) Prater (1977)

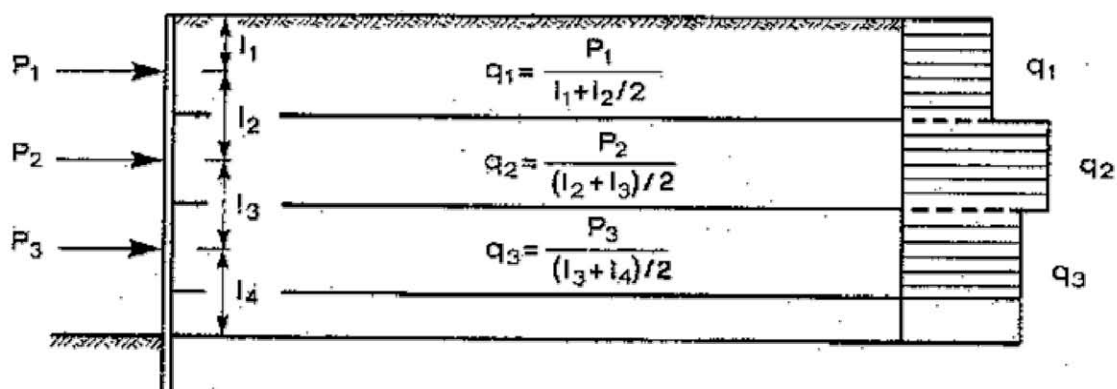
(3) Meyerhof (1951)

△ F. E. M. Crisp (Elasto-plastico perfetto)

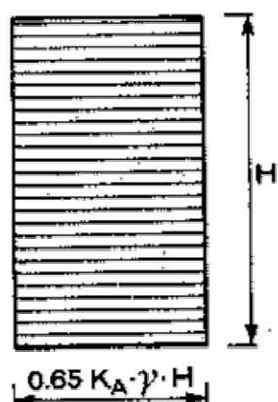
○ Prove in centrifuga

Fig. 2.1: Stabilità del fondo per scavi in terreni coesivi:

a) Geometria del problema; b) Numero di stabilità N_c , soluzione di Bjerrum & Eide (1956); c) Numero di stabilità N_c per scavi assialsimmetrici, in funzione del rapporto tra altezza dello scavo e raggio R (da Britto & Kusakabe 1984).

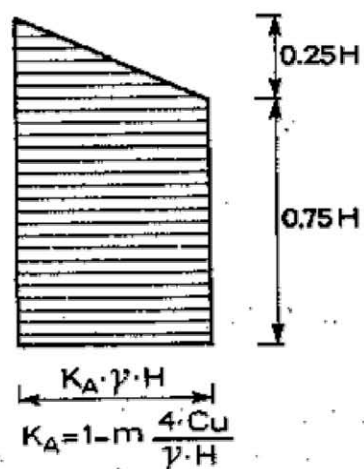


SABBIE



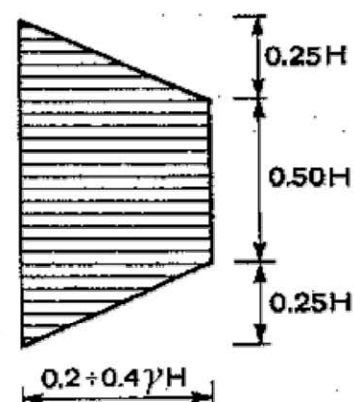
a)

ARGILLA NC



b)

ARGILLA OC



c)

$$N_o > H$$

$$N_o < H$$

$$m = 1$$

$$m = 0.4$$

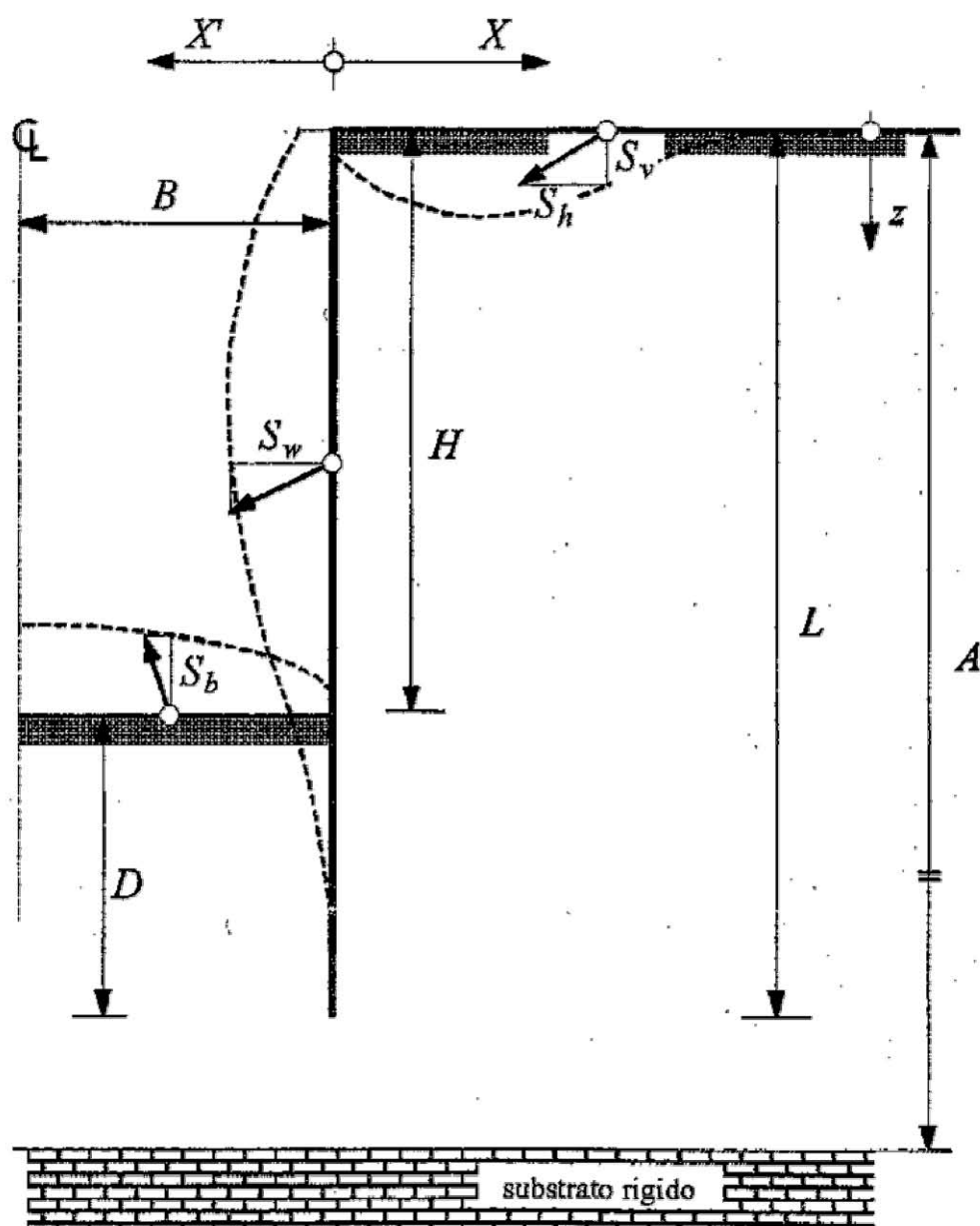


Fig. 4.1: Notazione impiegata per la descrizione dei movimenti del terreno adiacente allo scavo.

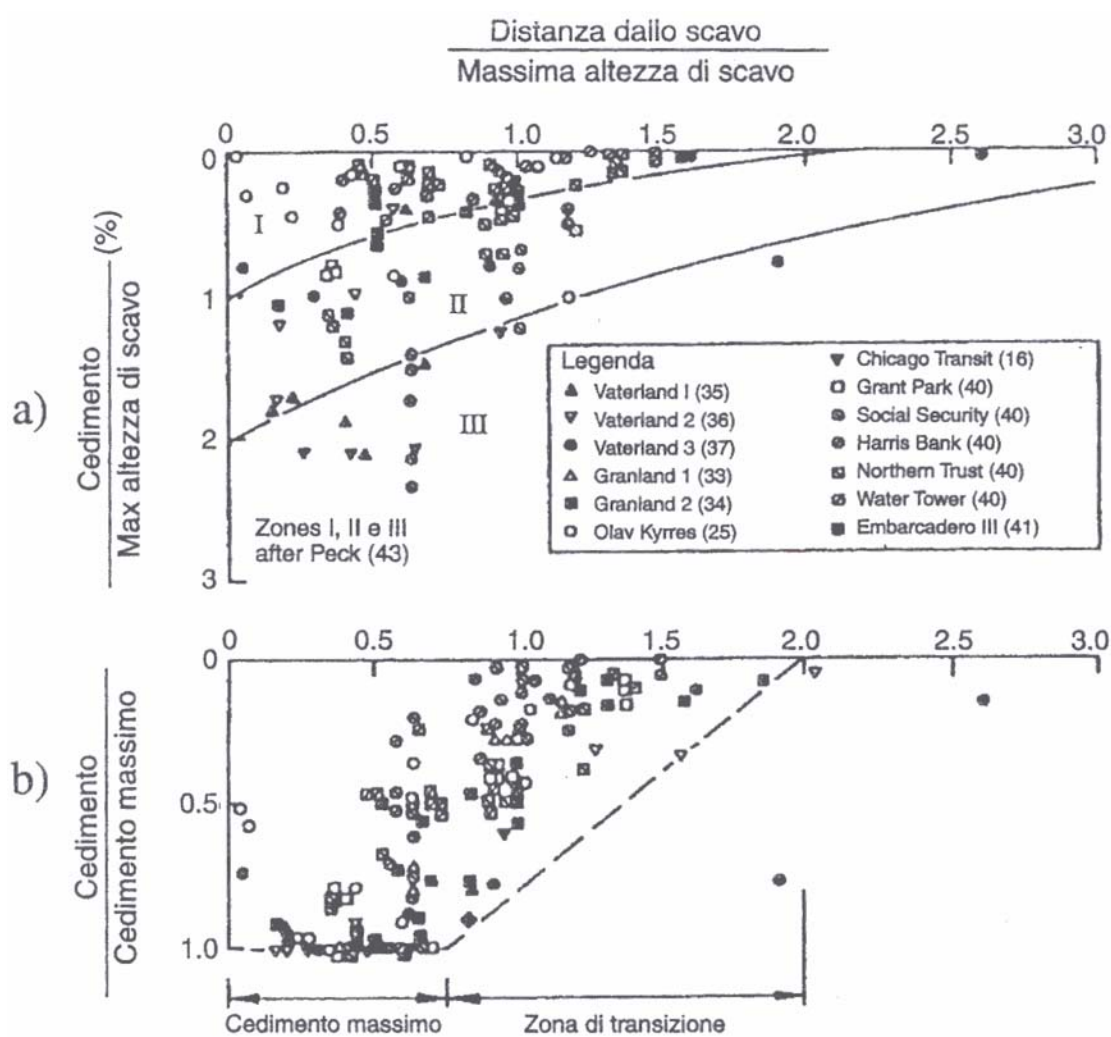


Fig. 4.4: Scavi in terreni coesivi da teneri a mediamente consistenti (da Clough & O'Rourke 1990). a) Spostamenti verticali misurati in funzione della distanza dalla parete; b) Rapporto $S_v/S_{v,max}$ in funzione della distanza dalla parete.

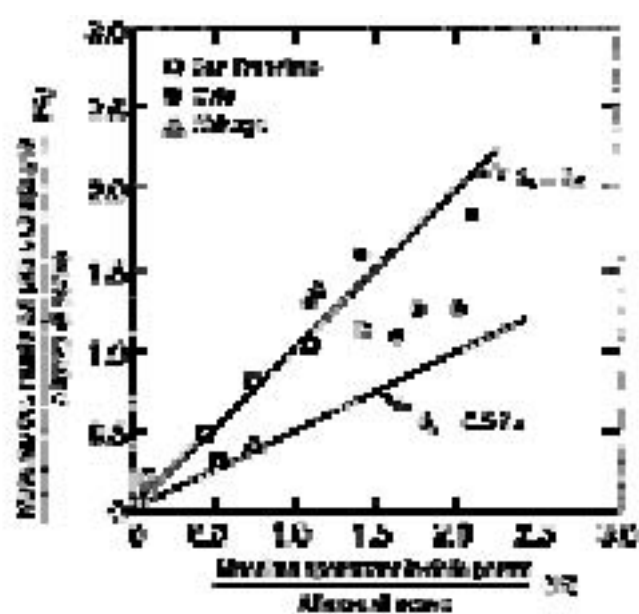


Fig. 4. Relationship between the maximum value of the function, S_{p-1} , and the average value, $\bar{S}_p$, for the data of Fig. 1.

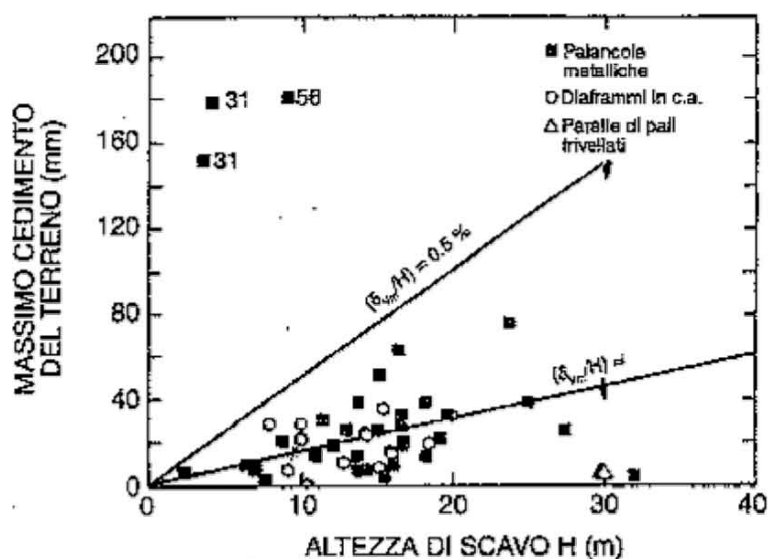


Fig. 4.9: Correlazione tra cedimento massimo ed altezza di scavo (da Clough & O'Rourke 1990).

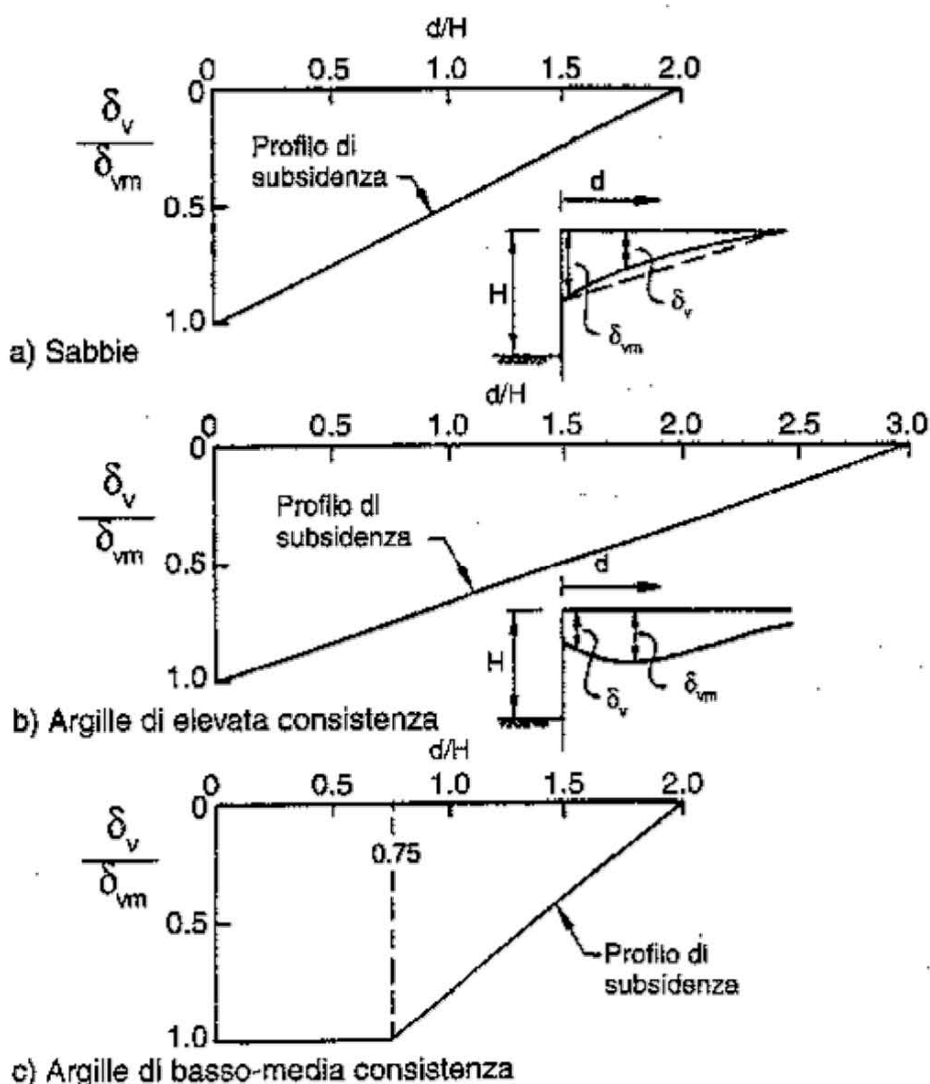
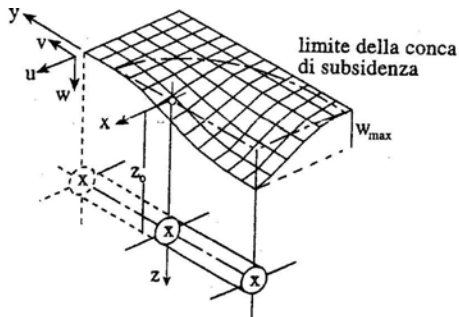
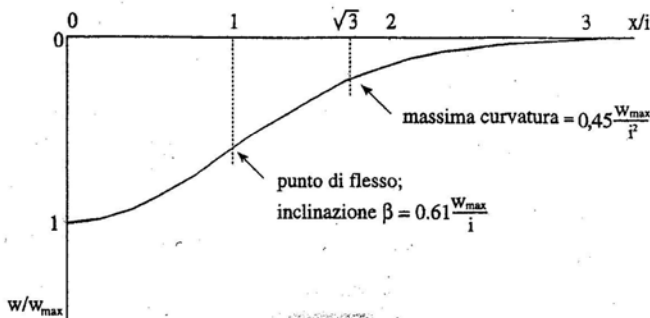


Fig. 4.10: Profili di subsidenza normalizzati proposti da Clough & O'Rourke (1990) (nota: nella figura δ_v indica lo spostamento verticale del piano di campagna).



$$z_0/D \leq 4 \div 5$$

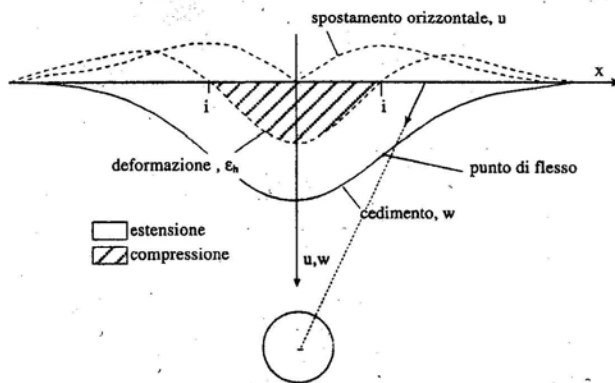


$$\frac{w}{w_{max}} = \frac{1}{e^{\frac{x^2}{2i^2}}}$$

$$i = k \cdot z_0$$

$$k = 0,5 \text{ coesivi}$$

$$k = 0,25 \text{ granulari}$$



$$V_s = \sqrt{2\pi} i w_{max}$$

$$V_s = V' \frac{u D^2}{4}$$

$$w_{max} = \frac{0,31 V' D^2}{k z_0}$$

TBH } SLURRY SHIELD
Earth pressure balance

$V' =$ { 3% argille tenere
1-2% argille consistenti
0,3% granulari sotto falda

$$u = \frac{x}{z_0} w$$

$$\epsilon_h = \frac{\partial u}{\partial x}$$