- Determining the required protective current
- Determining the most suitable number, location, size and type of anode
- Develop specifications for suitable mounting of the equipment to the structure
- Develop specifications for proper maintenance inspections.

Instead of using a cathodic protection system, it is possible to paint the steel elements with anti corrosion compositions or protective coatings to form a barrier to the environmental exposure and thereby delay the corrosion. The usefulness of this is often questionable because these barriers invariably break down after a number of years. Important factors in ensuring optimum performance of the protective coatings are the choice of coatings, the method of application and the thickness of coats.

The ideal and optimum protective system for steel in marine environment, could be a combination of different protective systems, because one system which is economical and effective in one zone might not be suitable for another zone. For example some coatings are effective and economical in the splash zone but less attractive in the submerged part of the structures due to difficult and high maintenance cost. In the submerged part the cathodic protection systems would be the most suitable. Therefore, if combinations of selected coatings and impressed current system are compatible, they can be an economical solution to the corrosion problem.

Where protective coatings or cathodic protection are not practical or their maintenance is doubtful, increased section or extra thickness of steel equal to the amount of corrosion expected for the lifetime of the structure may be economically justified and a technically better solution.

As a rough guide the steel thickness for steel used in marine structures should be minimum 10mm where cathodic protection is not used and 6mm where cathodic protection is used.

3.6 Open Berth Structures

3.6.1 *General*

The open berth structures constitute with their quay platforms a prolongation over the slope from the top of the filled area out to the berth front. In this chapter only berth structures in reinforced concrete, or platforms in reinforced concrete founded on concrete-filled tubular steel piles will be described. Open berth structures built of wooden materials will not be described, but construction principles, load bearing capacity etc. are largely the same as for structures in reinforced concrete.

In the same way as for solid berth structures the open berth structure can also be divided into two main types, depending on the principles according to which the front wall and the platform are designed to resist the loading, so that the berth structures get the necessary stability.

- Lamella Quays:
  The quay platform and the front wall are founded on vertical lamellas which provide the loaded berth structure with a satisfactory stability. The berth structures are stable enough in themselves to resist loads from ships, useful loads, possible pressure from fill at the rear of the structure, etc. without anchoring of the structure. In the same way as for gravity wall structures, one can build the lamella quay itself first, and then fill behind close up to the structure.

- Column or pile Quays:
  The quay platform and the front wall are founded on columns or piles which do not have a satisfactory stability against external forces. Therefore the quay structure must be anchored, for instance by a friction plate in the filling. The structure must then be built simultaneously with the filling or preferably after the fill has been established.

Very often the two types are combined. For instance, in a pier or jetty type of berth structure the shore base is anchored by a retaining wall while at the head of the jetty the horizontal loading is resisted by lamellas. The quay platform between the shore base and the head of the structure is founded on columns and/or piles.

Figure 3.6.1.A shows the cross-section of a pile quay anchored at the rear embankment and indicates the main characteristics of open berth structures. This method of accommodating the horizontal forces by use of an anchor and friction slab, is a typical Norwegian design.

Fig. 21 ESEMPIO DI MURI DI SPONDA SU PALI (Porto Marghera)

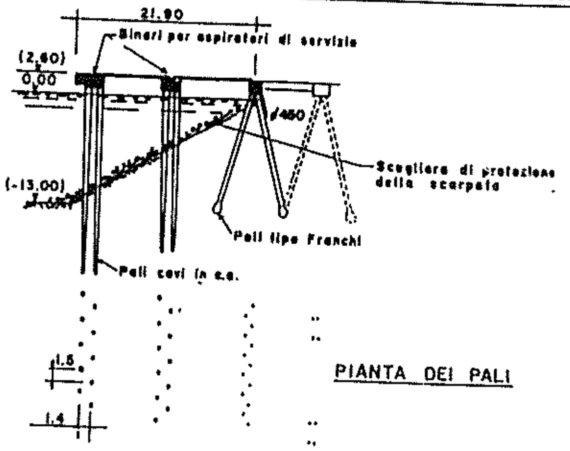
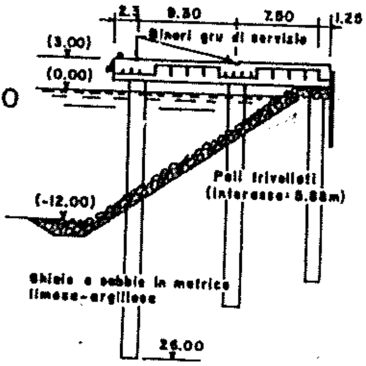
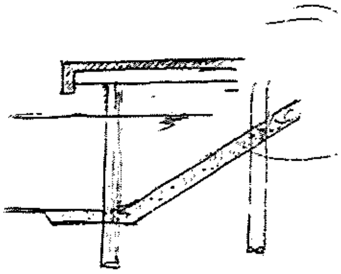


Fig. 22 MURO DI SPONDA SU PALI DI GRANDE DIAMETRO (Porto di Saline Joniche)





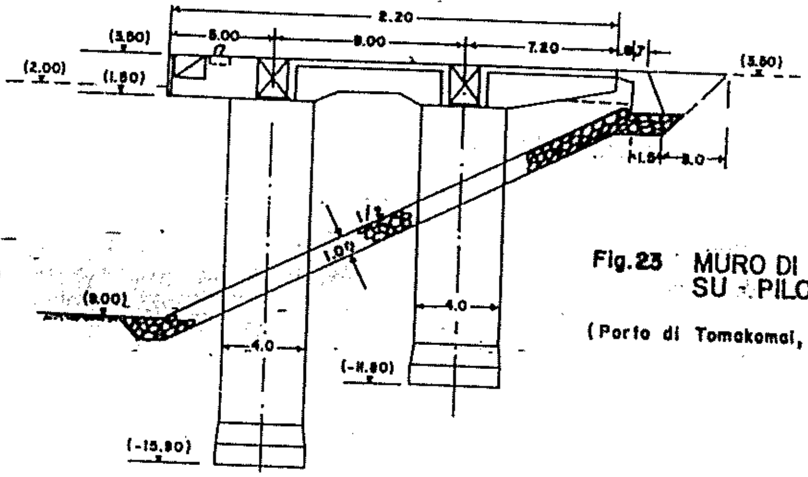
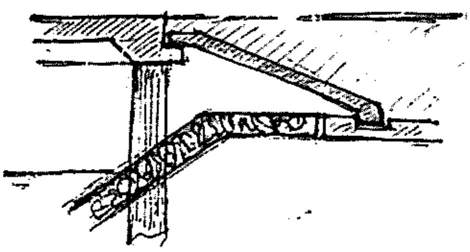
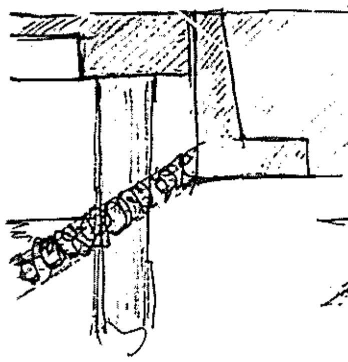
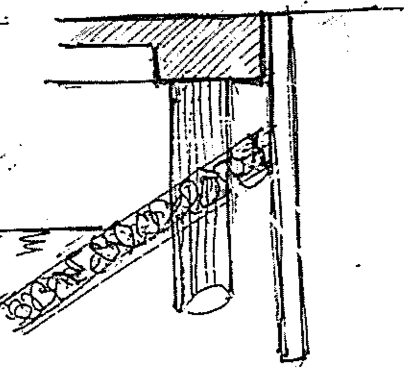


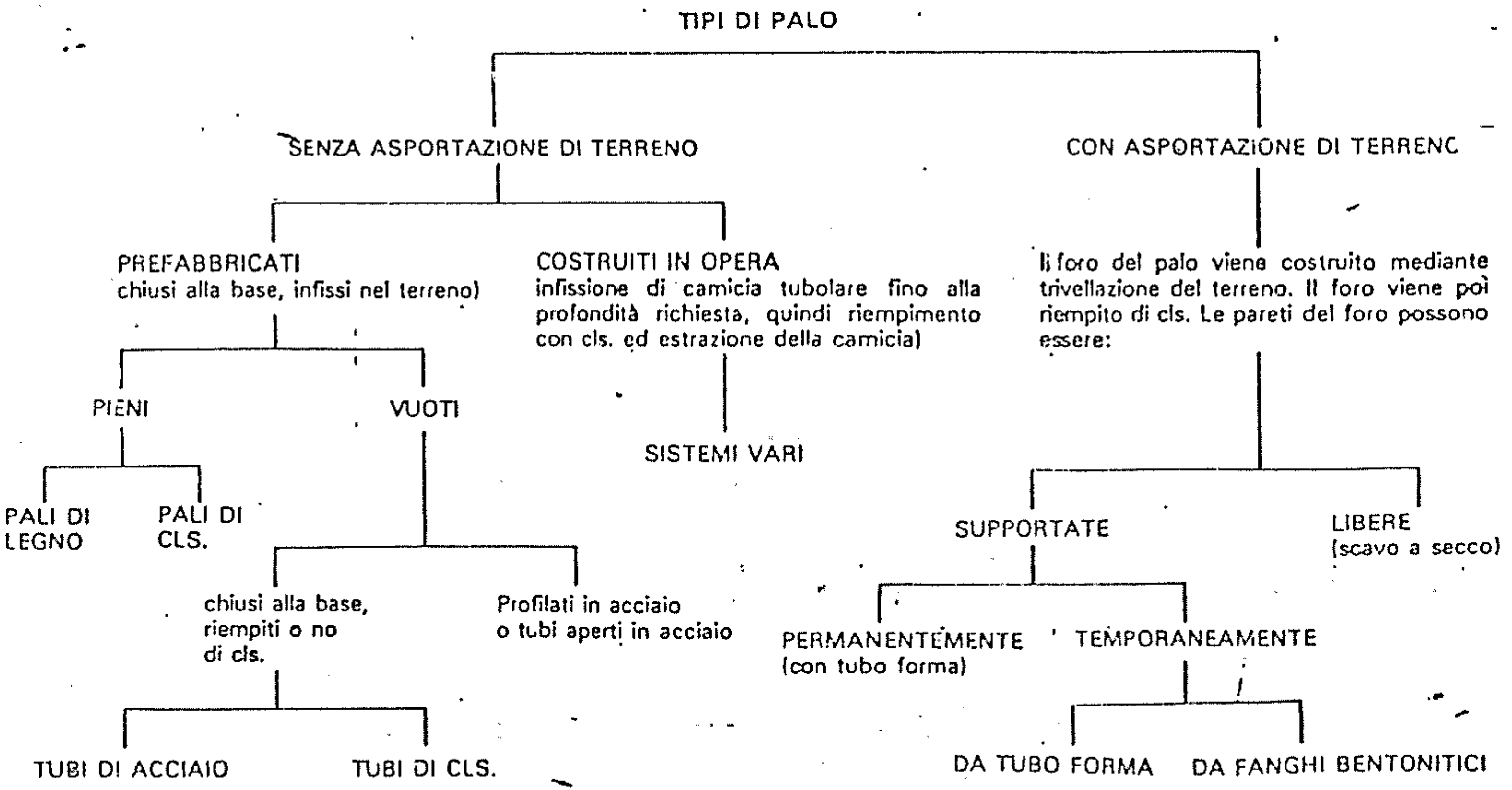
Fig. 23 MURO DI SPONDA SU PILONI

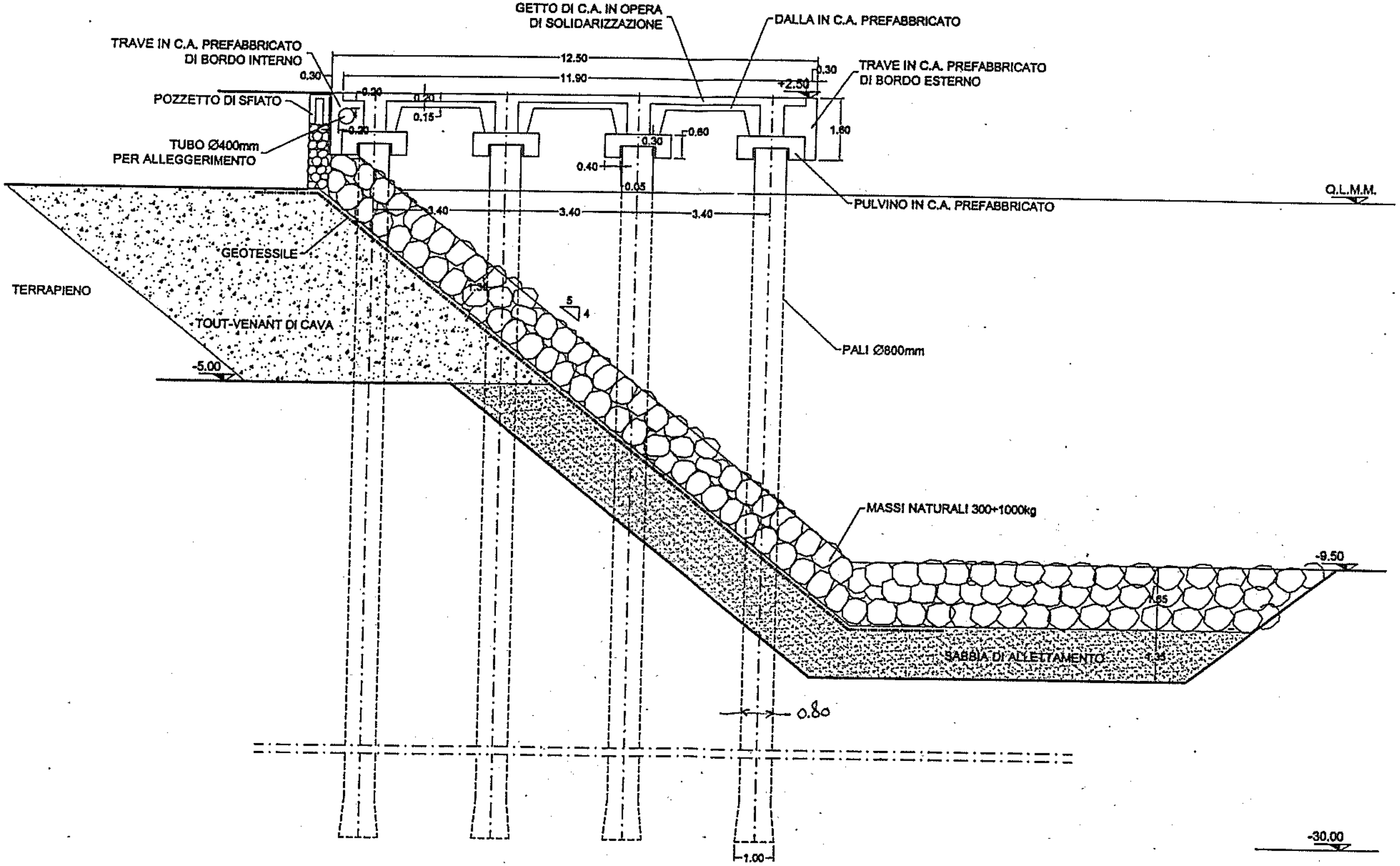
(Porto di Tomakomai, Giappone)








TIPI DI PALO
SENZA ASPORTAZIONE DI TERRENO
CON ASPORTAZIONE DI TERRENO
PREFABBRICATI
chiusi alla base, infissi nel terreno)
COSTRUITI IN OPERA
infissione di camicia tubolare fino alla profondità richiesta, quindi riempimento con cls. ed estrazione della camicia)
Il foro del palo viene costruito mediante trivellazione del terreno. Il foro viene poi riempito di cls. Le pareti del foro possono essere:
PIENI
VUOTI
SISTEMI VARI
PALI DI LEGNO
PALI DI CLS.
chiusi alla base, riempiti o no di cls.
Profilati in acciaio o tubi aperti in acciaio
SUPPORTATE
LIBERE
(scavo a secco)
PERMANENTEMENTE
(con tubo forma)
TEMPORANEAMENTE
TUBI DI ACCIAIO
TUBI DI CLS.
DA TUBO FORMA
DA FANGHI BENTONITICI

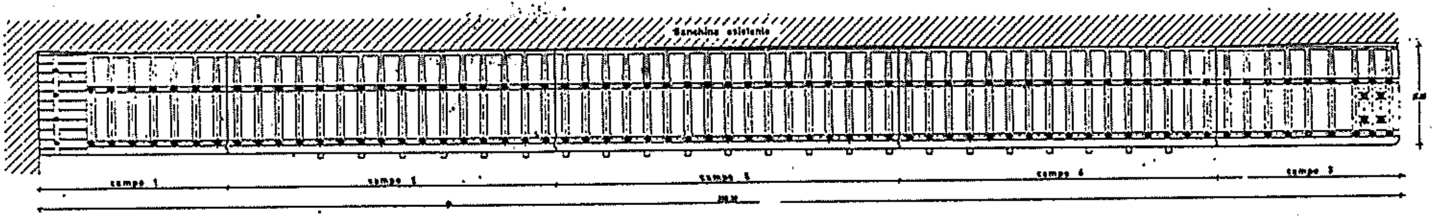


SEZIONE TIPO IN CORRISPONDENZA DI

SILOS DI GENOVA S.p.A.

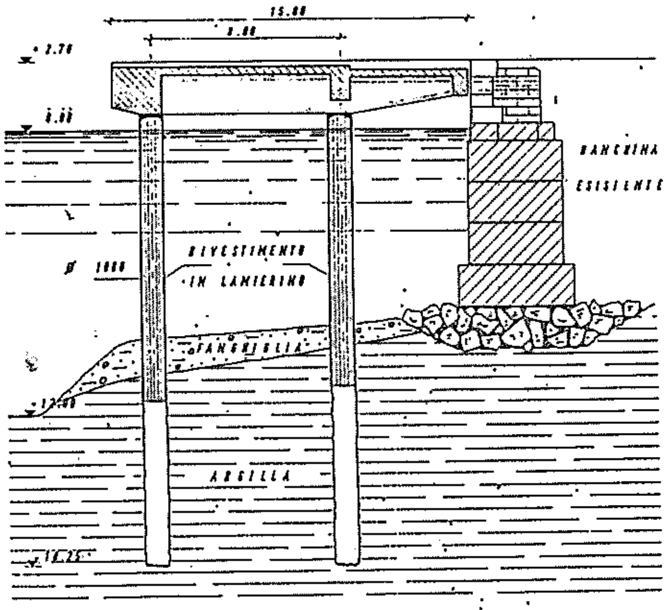
AMPLIAMENTO DELLA BANCHINA DI PONENTE DEL MOLO PARODI NEL PORTO DI GENOVA

PLANIMETRIA



IMPALCATO IN CALCESTRUZZO DI CEMENTO ARMATO FONDATO SU PALI DEL DIAMETRO DI MM. 1000 PARZIALMENTE RIVESTITI CON CASSAFORMA TUBOLARE METALLICA DEFINITIVA A MOMENTO DI INERZIA VARIABILE. I PALI IN CALCESTRUZZO DI CEMENTO ARMATO, SONO STATI REALIZZATI, DA NATANTE, COL METODO DI SCAVO BREVETTATO RODIO-MARCONI

SEZIONE TIPO



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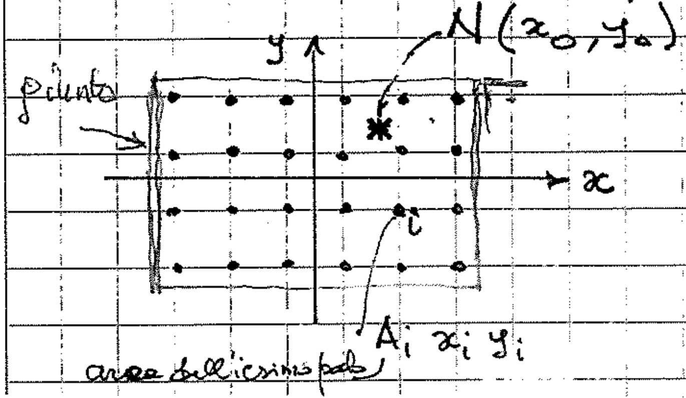
SONDAGGI • CEMENTAZIONI • CONSOLIDAMENTI
PALI • IMPERMEABILIZZAZIONI • FONDAZIONI DIFFICILI
RICERCHE GEOGNOSTICHE • DIAFRAMMI CONTINUI

... tanto per avere un'idea del carico di un palo.

Schema **elementare e preliminare** per valutare il carico sui pali.

Impalcato ~~oo~~ rigido ←

- I carichi e sovraccarichi uniformemente distribuiti si ripartiscono sugli n pali in modo uguale (impalcato rigido)
- Per individuare il carico sull'iesimo palo ~~si può procedere~~ per un carico concentrato N in $x_0 y_0$ si può dare il seguente "colpo di regolo" -



$$M_x = N y_0$$

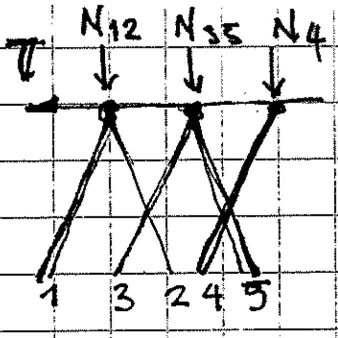
$$M_y = N x_0$$

$$\sigma_i = \frac{N}{\Sigma A_i} \pm \frac{M_x y_i}{\Sigma (A_i y_i^2)} \pm \frac{M_y x_i}{\Sigma (A_i x_i^2)}$$

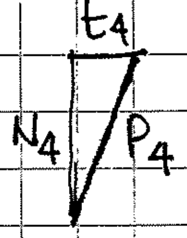
se A è uguale per tutti gli n pali

$$\sigma_i A = N_i = \frac{N}{n} \pm \frac{M_x y_i}{\Sigma y_i^2} \pm \frac{M_y x_i}{\Sigma x_i^2}$$

- Schema elementare e preliminare per avere un'idea del tiro T assorbibile con pali inclinati



ipotesi: impalcato = asta rigida
cerniere tra impalcato e teste pali



$$P_1 = P_1' + P_1^*$$

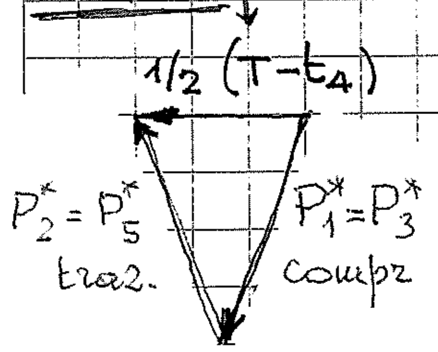
$$P_2 = P_2' - P_2^*$$

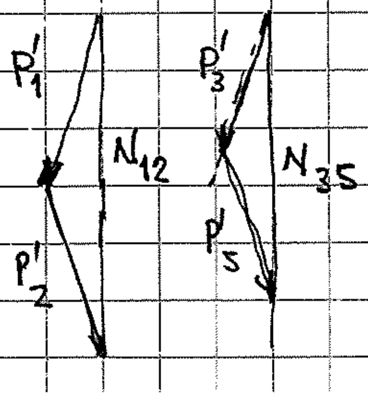
$$P_3 = P_3' + P_3^*$$

$$P_4 = P_4$$

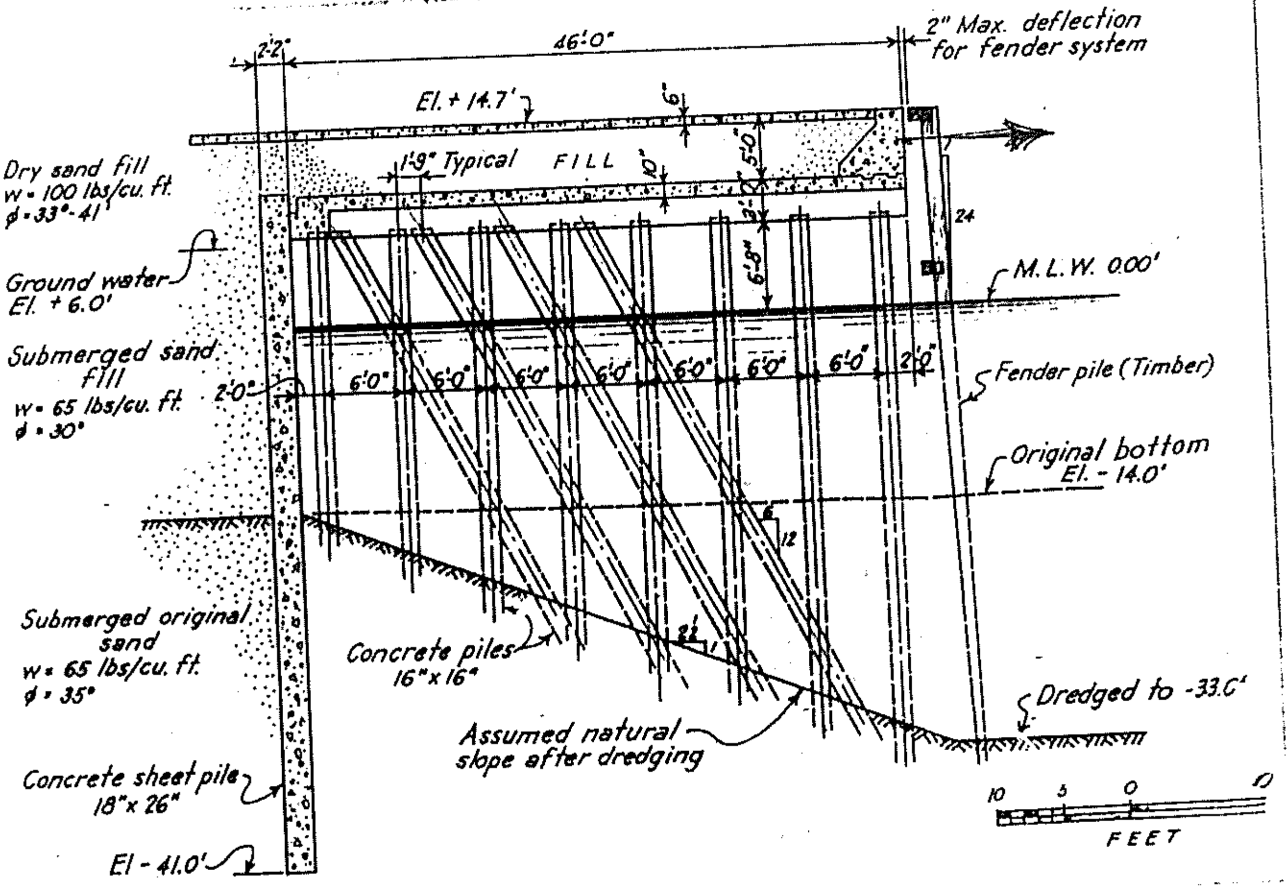
$$P_5 = P_5' - P_5^*$$

INCOGNITA



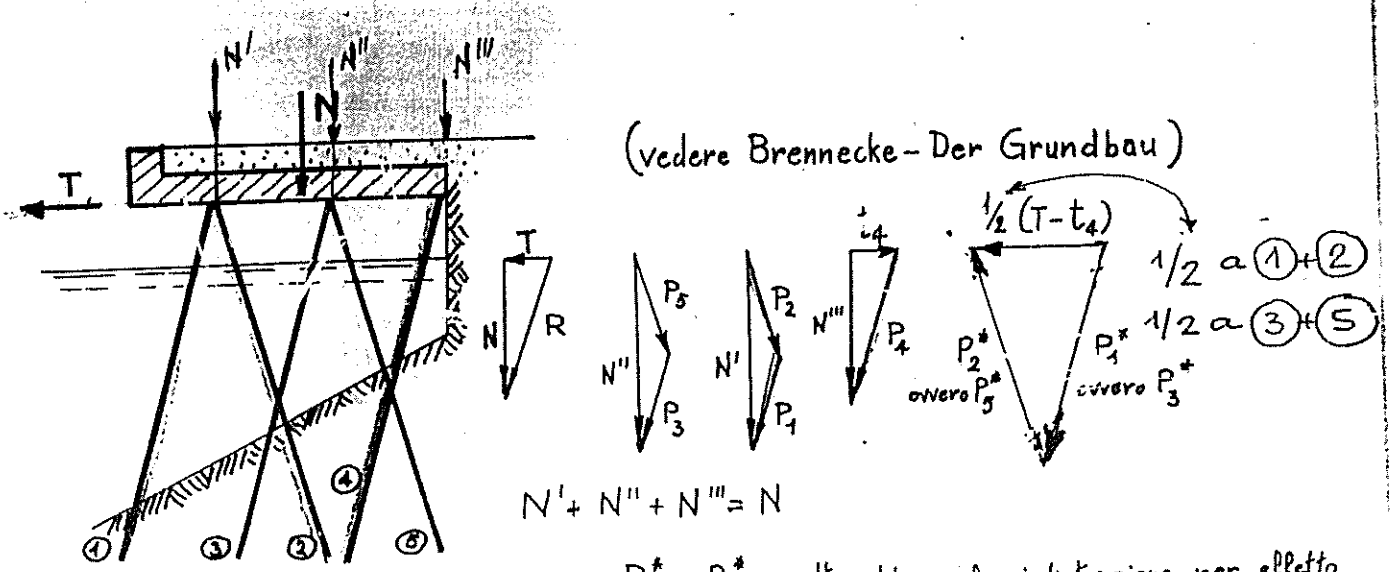


Con questi criteri é stata - p. es. - calcolata la banchina americana della fig. 2 allegato V/D - su pali in c.a. da 16" x 16", interasse stilate 10'. Notare la palancolata in c.a. a tergo.



Quando si può temere un inversione nel segno della componente T si disporranno pali inclinati nei due sensi.

(vedere Brennecke - Der Grundbau)



$N' + N'' + N''' = N$

N', N'' ed N''' determinati con i criteri del paragrafo a) o metodo del trapezio (Terzaghi)

P_2^* e P_5^* risulterebbero sforzi di trazione per effetto della ½ $(T - t_4)$; però combinando con le P_2 e P_5 dovute alla N risultano di norma ancora compressi.

QUINN

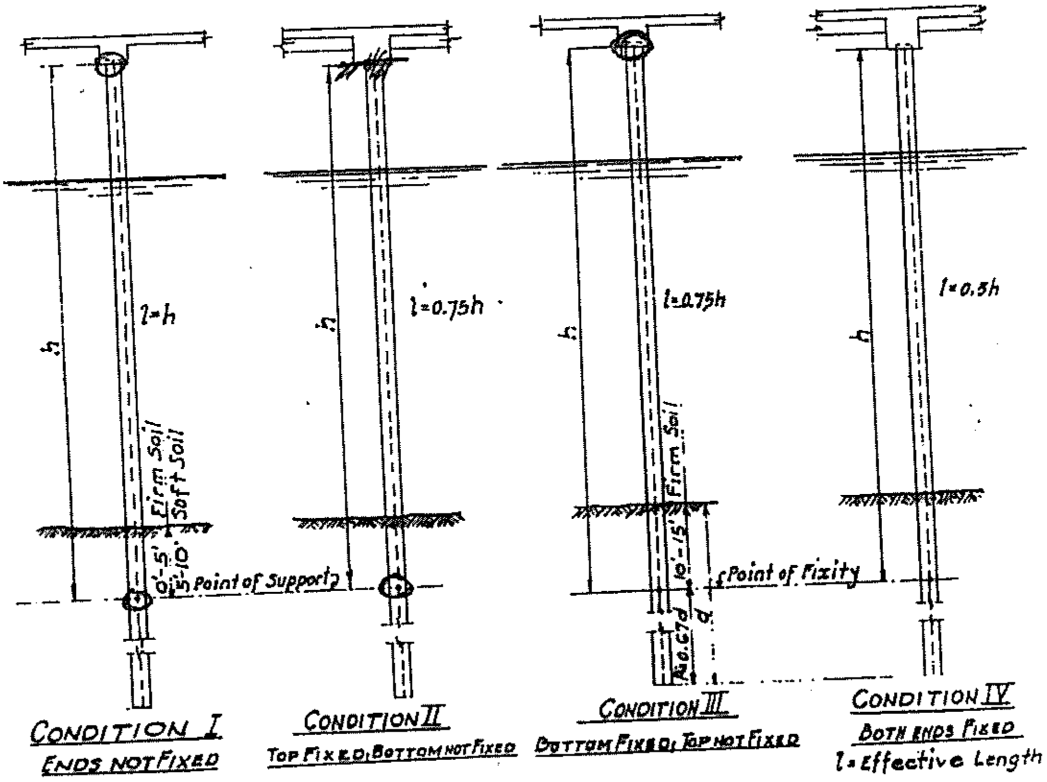
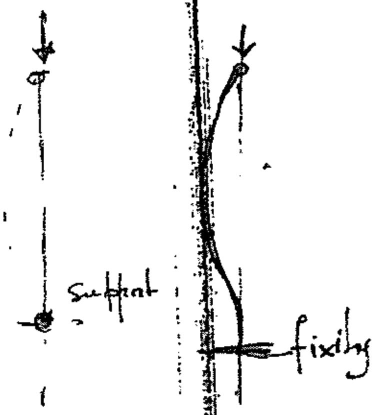


FIG. 5.21 Effective length of pile for various end conditions.

General Design. In designing a pile to carry a certain load it is necessary to determine the condition of support at both the top and the bottom because in dock construction the piles have to be designed as long columns. The pile may be considered fixed if its ends are prevented from rotating. This means that for a vertical pile the axis of the pile must remain vertical at its ends. For fixity at the top of the pile it is essential that the deck be of heavy construction and that the pile be rigidly fastened to the deck either by embedding it, if an H pile, or by extending the reinforcing into the supporting cap or girder, if a concrete pile. To fix the pile at a point not too far below [illegible] bottom, the soil must be a firm material such as compact sand or hard clay into which the pile is driven a substantial distance below the point of fixity. The [illegible] of fixity in this case is assumed to be 10 to 15 ft below the bottom, and, with both ends fixed, the effective length of the pile is taken to be 0.5 of its length between the points of fixity. If it is a soft bottom, such as silt, a point of fixity would occur, if at all, not much above a depth of 20 to 25 ft below the bottom, whereas the pile may be considered as being supported from buckling [illegible] a depth starting 5 to 10 ft below the bottom.



$J = \frac{\pi R^4}{4} = \frac{\pi D^4}{64}$

$J = \frac{\pi}{4}(R^4 - r^4)$

$= \frac{\pi}{4}(R^2+r^2)(R^2-r^2) = \frac{\pi}{4}(R^2+r^2)(R+r)(R-r)$

$= \frac{\pi}{4}\, \underset{\uparrow}{2R^2}\, 2R\, s = \pi s R^3$

$(R \simeq r)$ ipotesi

point of support

Most soils, even though of relatively low strength, will provide sufficient support to prevent the pile from buckling. If the material is firm, and the pile is not driven sufficiently deep to provide a point of fixity, the pile may be assumed to be supported from 0 to 5 ft below the bottom. In the case of soft rock, hardpan, or hard clay, the support may be taken at the surface, but, for other materials that are less dense, and which may be eroded or disturbed near the surface, the point of support should be assumed 5 ft below the bottom. Therefore, if the pile is fixed at the top but not at the bottom, the effective length for design is 0.75 of its length figured from the point of fixity at the top to the point of support at the bottom. If the deck is of light construction such as wood or light steel. the top of the pile cannot be considered as fixed, and the effective length of the pile then becomes the distance between the point of support at the top (usually the underside of the deck) and the point of support at the bottom. These various conditions are shown in Fig. 5.21. Once the effec-

$P_c = \frac{\pi^2 E J}{l^2}$ (Eulero)

$l = k h$

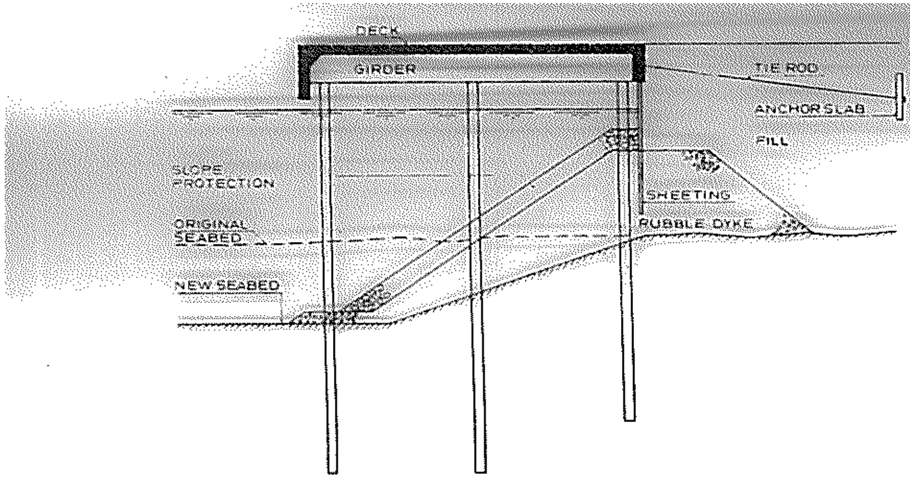


Fig. 8.4 (n) Anchored Open Structure

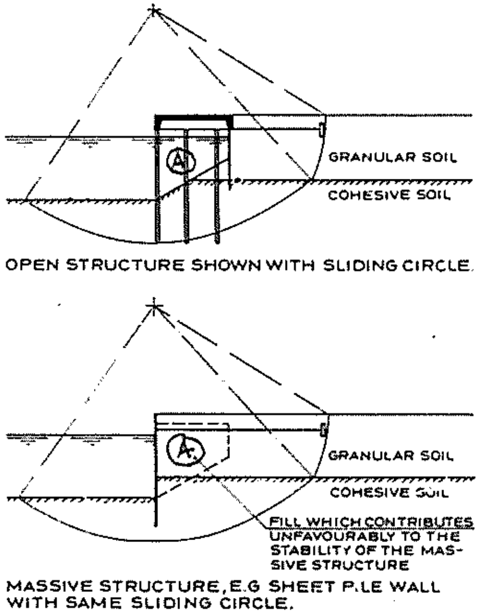


Fig. 8.4 (o) Comparison of Sliding Stability for Open Structure and Massive Structure

forces from ships' impact are transmitted through the deck and absorbed as earth pressure behind the low bulkhead. Mooring forces and active earth pressure at the rear of the low bulkhead are supported either by batter piles or by tie rods and anchor slabs as shown.

It is a fact that a number of open wharves have been constructed with fill placed on top of the deck as shown in Fig. 8.4(p). Both batter piles and tie rods plus anchor slabs have been used for picking up horizontal loads. In the former case the weight of the fill is likely to precompress the vertical pile adjoining the batter pile so that tension will be less or will not develop for horizontal loads such as mooring forces. When tie rods and anchor slabs are used, the fill on top of the deck only adds unnecessary load to the deck and the piles.

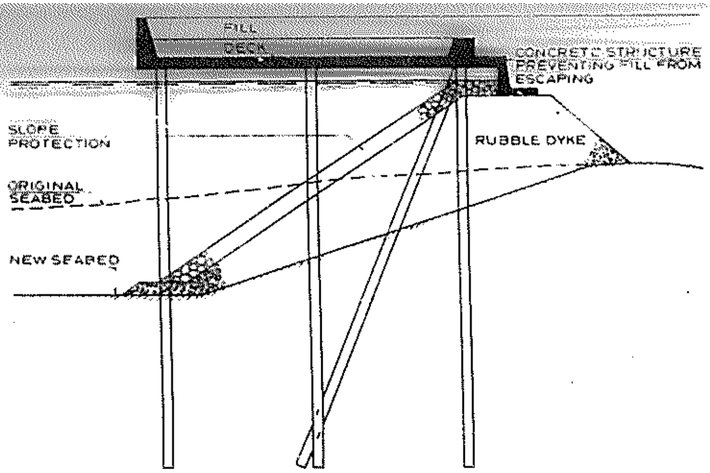


Fig. 8.4 (p) Open Structure with Batter Piles

Often it is necessary to provide the edge of the deck with a curtain or take other measures which will ensure that small vessels with low freeboard are not trapped underneath the deck at low water.

8.5 PIERS

8.5.1 General considerations

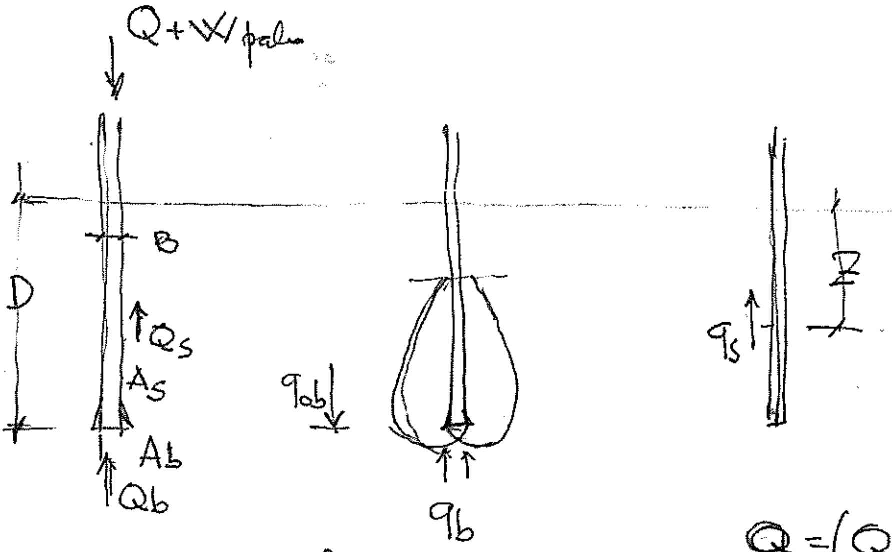
Piers may be divided into three types: (a) piers consisting mainly of reclaimed land and bordered by quays, (b) piled piers, and (c) floating piers. The first type is merely a particular arrangement of a quay structure which is dealt with in 8.4. The other two types are dealt with below.

The layout of piers in the plan also varies. Finger piers abut the shore at one end while T-head and L-head piers are connected to land by a trestle and/or a causeway with an access road.

8.5.2 Piled piers

Piled piers may be chosen rather than massive piers because of cost, e.g. if the pier is located at considerable water depth, because the massive pier may develop unacceptable settlements or because a nonreflecting structure is preferred for wave disturbance reasons.

Piled piers simply consist of a deck supported on piles, but the pattern of piling, the type of piles and the particular design of the deck may be varied and combined almost indefinitely. Piles may be of steel, reinforced or prestressed concrete or timber.



valori limite per deform.

↓

$$Q_{bu} = q_b A_b \quad (0.1 \div 0.3B)$$

$$Q_{su} = \underset{\uparrow \text{medio}}{\bar{q}_s} A_s \quad (5 \div 10mm)$$

$$Q = (Q_{bu} + Q_{su}) / 2 \div 4$$

Res. unitaria alla punta:

$$q_b = (\overset{\downarrow N_c}{9} c + \overset{\downarrow N_q}{1} q_{ob} + \overset{\downarrow N_\gamma}{0})$$ punta in terr. argilloso e limoso ($\overset{c = c_u}{q_{ob}} = \gamma D$)

$$q_b = q'_{ob} N_q \quad \phi = \begin{matrix} 25° \\ 30° \\ 35° \end{matrix} \quad N_q = \begin{matrix} 17 \\ 30 \\ 70 \end{matrix}$$ punta in sabbie/ghiaie ($q'_{ob} = \gamma' D$)

Res. unitaria di «attrito laterale»

$q_s = \alpha c_u$ tab 9.2 terreni coesivi

$q_s = K \sigma'_{v0} \tan\delta$ tab 9.3 terreni sabb./ghiaiosi ($\sigma'_{v0} = \gamma' z$)

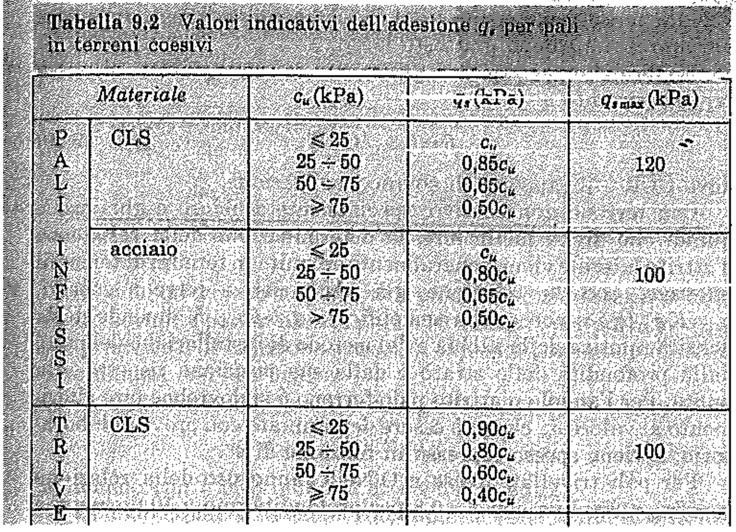


Tabella 9.2 Valori indicativi dell'adesione q_s per pali in terreni coesivi

	Materiale	c_u (kPa)	$\bar{q}_s$ (kPa)	$q_{s\,max}$ (kPa)
PALI INFISSI	CLS	≤25 25 ÷ 50 50 ÷ 75 ≥75	c_u $0,85c_u$ $0,65c_u$ $0,50c_u$	120
	acciaio	≤25 25 ÷ 50 50 ÷ 75 >75	c_u $0,80c_u$ $0,65c_u$ $0,50c_u$	100
TRIVE	CLS	≤25 25 ÷ 50 50 ÷ 75 ≥75	$0,90c_u$ $0,80c_u$ $0,60c_u$ $0,40c_u$	100

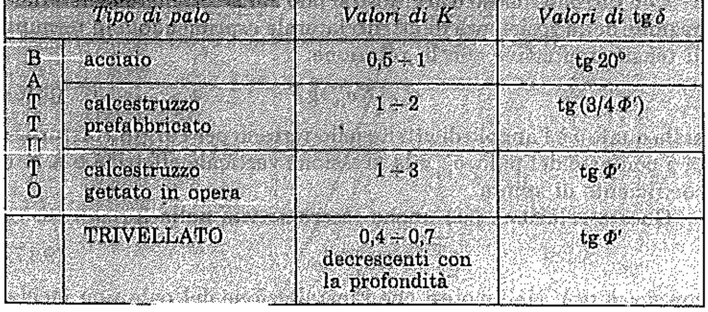


Tabella 9.3

	Tipo di palo	Valori di K	Valori di tg δ
BATTUTO	acciaio	0,5 ÷ 1	tg 20°
	calcestruzzo prefabbricato	1 ÷ 2	tg (3/4 Φ')
	calcestruzzo gettato in opera	1 ÷ 3	tg Φ'
	TRIVELLATO	0,4 ÷ 0,7 decrescenti con la profondità	tg Φ'

Da Colombo Colleselli
Es. di Geotecnica.

Indagini e prove in situ

7.1. Premessa

La necessità e l'opportunità della caratterizzazione geotecnica delle terre in situ o dei materiali che formano alcune opere in terra, insieme alle difficoltà connesse al corretto campionamento specialmente per alcuni tipi di terreno, hanno spinto ad ideare ed utilizzare vari tipi di prove sul posto.

La caratterizzazione geotecnica comprende la definizione della stratigrafia del terreno, delle condizioni dell'acqua nel terreno e la determinazione delle caratteristiche idrauliche e meccaniche del terreno; essa viene ottenuta attraverso prove sul posto e prove di laboratorio.

Le indagini in situ hanno alcuni vantaggi rispetto alle prove di laboratorio; infatti esse consentono di determinare un andamento pressoché continuo delle caratteristiche geotecniche con la profondità, interessano un volume di terreno maggiore e in genere danno indicazioni più attendibili sulla deformabilità e sulla permeabilità di alcuni terreni. Per i terreni incoerenti, per i quali è quasi impossibile prelevare campioni indisturbati, le prove sul posto sono necessarie per individuare le caratteristiche meccaniche e idrauliche; però, mentre nelle prove in laboratorio le condizioni di prova sono in genere ben definite, in quelle sul posto le condizioni di drenaggio e le condizioni al contorno sono più incerte e si hanno gradienti elevati di tensioni e deformazioni.

L'interpretazione dei risultati delle prove in situ presenta talora notevoli difficoltà ed è necessario ricorrere a correlazioni empiriche, facendo riferimento ai dati delle prove di laboratorio, alle tarature degli strumenti di prova eseguite in laboratorio con particolari apparecchiature e ai dati provenienti da indagini particolarmente approfondite eseguite per lavori di notevole impegno.

Le prove in situ e le prove di laboratorio devono essere considerate come due procedimenti complementari che opportunamente utilizzati in parallelo consentono di ridurre parecchie incertezze nella valutazione del comportamento dei terreni.

In talune situazioni, tenendo presenti i vantaggi e gli svantaggi dei vari tipi di prova, può essere conveniente privilegiare certi tipi di prova rispetto ad altri.

Per la determinazione della stratigrafia si usano principalmente i sondaggi con i quali è possibile riconoscere la successione e la natura dei terreni e prelevare campioni rimaneggiati e indisturbati. I sondaggi sono delle perforazioni generalmente ad asse verticale, che consentono di riconoscere la successione e la natura dei terreni attraverso il prelievo di campioni rimaneggiati e indisturbati; nei fori di sondaggio possono essere eseguite prove in situ di vario tipo e possono essere installate apparecchiature di misura quali piezometri, assestimetri, inclinometri, ecc.

Le attrezzature per l'esecuzione del sondaggio possono essere a rotazione o a percussione; attualmente sono usate prevalentemente quelle a rotazione, anche perché causano un minor disturbo al materiale.

Durante la perforazione deve essere mantenuta la stabilità della parete e del fondo del foro ricorrendo spesso a tubazioni di rivestimento e/o all'uso di fango bentonitico e di polimeri.

Il prelievo di campioni rimaneggiati, cioè di campioni che permettono l'esecuzione delle prove di classificazione e a volte la determinazione del contenuto d'acqua, non presenta particolari difficoltà e non richiede il ricorso ad attrezzature e tecniche particolari. Il prelievo di campioni indisturbati, cioè di campioni che mantengono la struttura e il contenuto d'acqua del terreno in situ, richiede l'utilizzazione di campionatori adeguati ai diversi tipi e situazioni dei terreni. È quasi impossibile prelevare campioni indisturbati di materiali granulari incoerenti.

Varie prove sul posto permettono di misurare direttamente alcune caratteristiche e di risalire in via indiretta alla determinazione della stratigrafia.

Molto utilizzate sono le prove penetrometriche che vengono eseguite con penetrometri dinamici e con penetrometri statici.

Nelle prove penetrometriche dinamiche l'utensile viene infisso a percussione e si registra il numero di colpi necessario per ottenere un avanzamento prefissato. L'utensile può essere cavo e costituito da un campionatore standard (*Standard Penetration Test* o SPT) o a punta conica chiusa (SCPT).

Nelle prove penetrometriche statiche (*Cone Penetration Test* o CPT) si infigge nel terreno una punta con avanzamento controllato; viene misurata in superficie la pressione esercitata sulle aste di manovra nei penetrometri meccanici e direttamente sulla punta nei penetrometri elettrici.

In questi ultimi anni si è anche messa a punto e utilizzata la prova penetrometrica con piezocono (CPTU) con la quale, attraverso una punta penetrometrica opportunamente modificata, è possibile misurare anche la pressione dell'acqua nel terreno durante l'avanzamento.

Vi sono poi le prove scissometriche (indicate anche come *Field Vane Test*) con le quali viene infisso nel terreno un utensile avente all'estremità quattro alette poste a croce, utensile che viene fatto

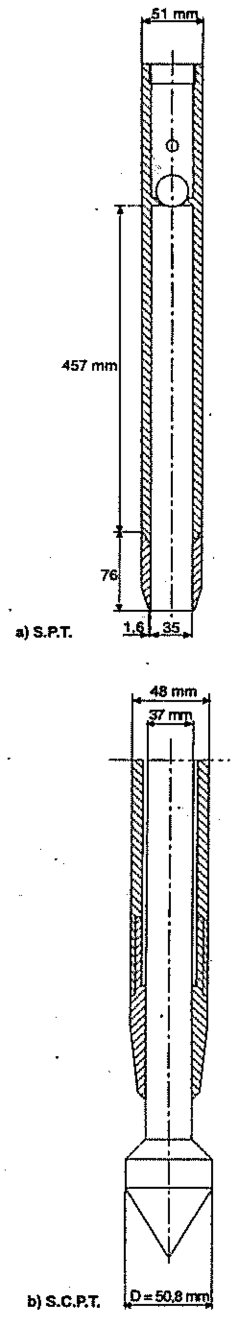


Figura 7.1

forma circolare che viene fatta dilatare misurando la pressione corrispondente all'inizio del moto della membrana e quella corrispondente ad uno spostamento prefissato.

Le prove pressiometriche sono delle prove di carico realizzate installando nel terreno alla profondità voluta una sonda cilindrica dilatabile, facendola poi espandere gradualmente e misurando nel contempo le pressioni applicate e le deformazioni corrispondenti.

Con il pressiometro tipo Menard (MPM), ideato per primo, la sonda viene posizionata in un foro di sondaggio appositamente eseguito. Con i pressiometri autoperforanti (SBP) la prova viene eseguita rimuovendo con lo stesso attrezzo il terreno corrispondente al volume della sonda e limitando quindi l'effetto del disturbo nel terreno.

Abbiamo poi le prove di carico con piastra usate spesso in superficie, specialmente nel campo delle costruzioni stradali, ma talvolta anche in profondità; con queste prove si determinano prevalentemente le caratteristiche di deformabilità. A questo tipo di prove appartengono quelle con piastra ad elica (*screw plate*) con le quali vengono misurati i cedimenti di una piastra ad elica avvitata nel terreno e poi caricata. Vi sono infine prove geofisiche, nelle quali si misura la velocità di propagazione delle onde sismiche, e fra queste si ricordano quelle *cross-hole*, quelle *down-hole* e le prove geoelettriche nelle quali si misura la resistività del terreno.

7.3. Prove penetrometriche dinamiche

Come è già stato detto l'utensile di percussione può essere cavo (SPT) o con punta conica chiusa (SCPT) (figura 7.1).

Nello *Standard Penetration Test* (SPT), molto usato negli Stati Uniti d'America ed in Inghilterra, si misura il numero di colpi necessario per infiggere il campionatore standard (figura 7.1a) per 30 cm (1 piede) di profondità battendo con un maglio di peso di 63,5 kg (140 libbre) e con un'altezza di caduta di 76,2 cm (30 pollici). La prova viene eseguita sul fondo del foro di sondaggio infiggendo il campionatore per 45 cm e tenendo conto dei colpi relativi agli ultimi 30 cm di infissione. Sebbene la prova sia relativamente rozza, da parte di studiosi ed amministrazioni pubbliche americane, sono state approntate tabelle e diagrammi che correlano prevalentemente i risultati della prova standard con le caratteristiche dei terreni granulari, per i quali d'altra parte i risultati delle prove penetrometriche sono più attendibili.

All'aumentare della profondità oltre i 20 m i risultati sono meno attendibili in relazione anche alla diminuzione dell'energia trasmessa alla punta.

Tabella 7.1

N_{SPT} (colpi/30 cm)	*Densità relativa*	
	Terzaghi-Peck	*Gibbs-Holtz*
0 ÷ 4	molto sciolta	0 ÷ 15%
4 ÷ 10	sciolta	15 ÷ 35%
10 ÷ 30	media	35 ÷ 65%
30 ÷ 50	densa	65 ÷ 85%
> 50	molto densa	85 ÷ 100%

Una prima correlazione si ha tra la densità relativa D_r e i valori di N_{SPT}, numero di colpi misurati nella prova SPT, correlazione già indicata in via qualitativa da Terzaghi e Peck nel 1948 e poi in via quantitativa con prove in cella di calibrazione da Gibbs e Holtz (1957) (tabella 7.1).

Questa correlazione è valida per le sabbie normalconsolidate mentre sembra sopravvalutare D_r per le sabbie sovraconsolidate.

Vi sono anche delle correlazioni (figura 7.2, Schmertmann, 1977) tra l'angolo d'attrito Φ' e la densità relativa D_r (%) per terreni con differenti granulometrie.

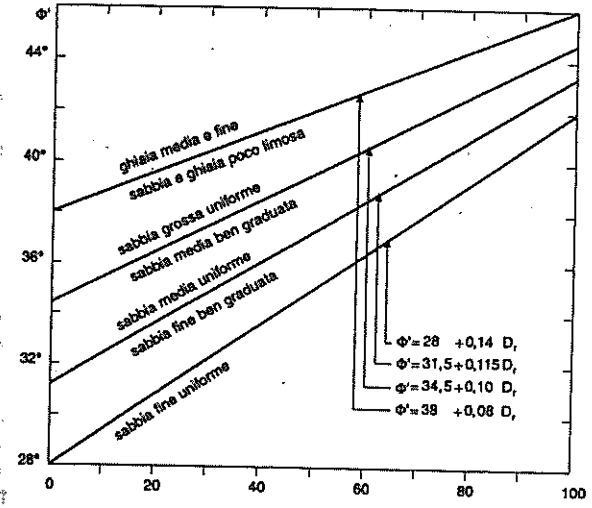


Figura 7.2

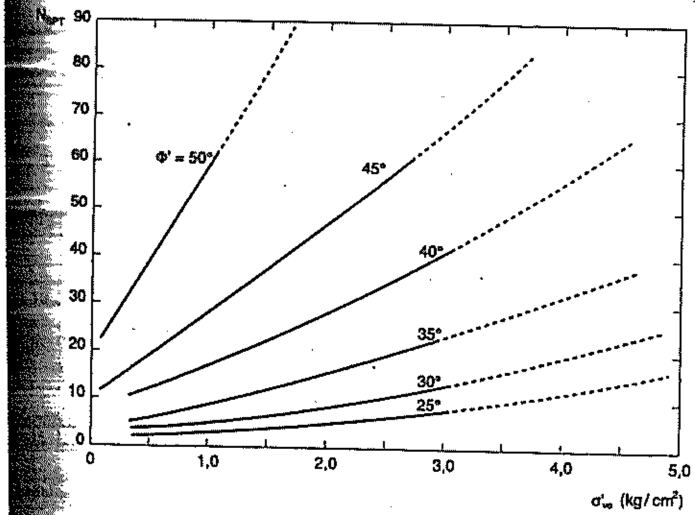


Figura 7.3

ed to the top
ıy contain a
usually rests
ı reasonably
(particularly
hammer are
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) be inserted

: soil and are
the hammer

fit inside the
ɔme the pipe
ich basically
shell is not

: hard strata
or simply a
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ecial driving
e rock slope.
the pile that
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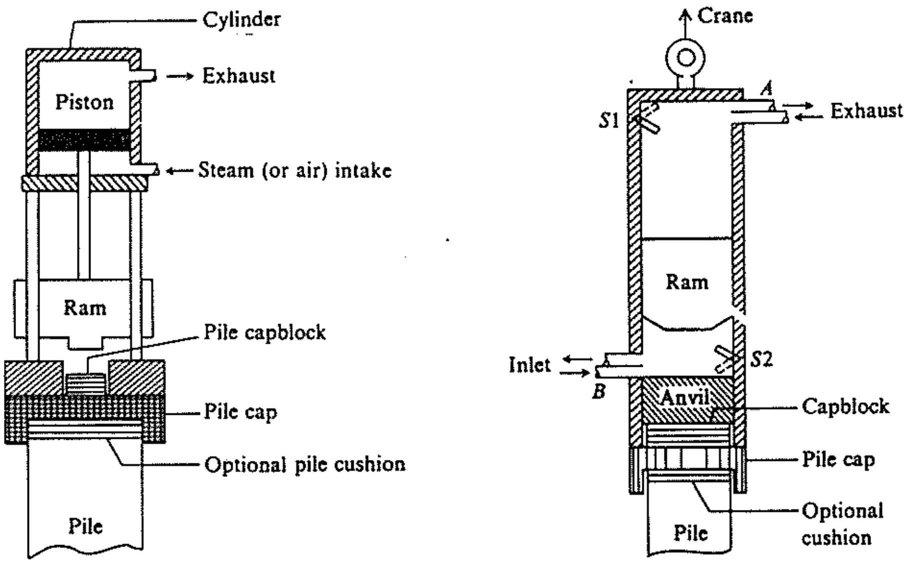


(*a*) Single-acting hammer. At bottom of stroke, intake opens with steam pressure raising ram. At top of lift steam is shut off and intake becomes exhaust, allowing ram to fall.

(*b*) Double-acting hammer. Ram in down position trips *S2*, which opens inlet and closes exhaust valves at *B* and shuts inlet and opens exhaust at *A*; hammer then raises from steam pressure at *B*. Ram in up position trips *S1*, which shuts inlet *B* and opens exhaust; valve *A* exhaust closes; steam enters and accelerates ram downward.

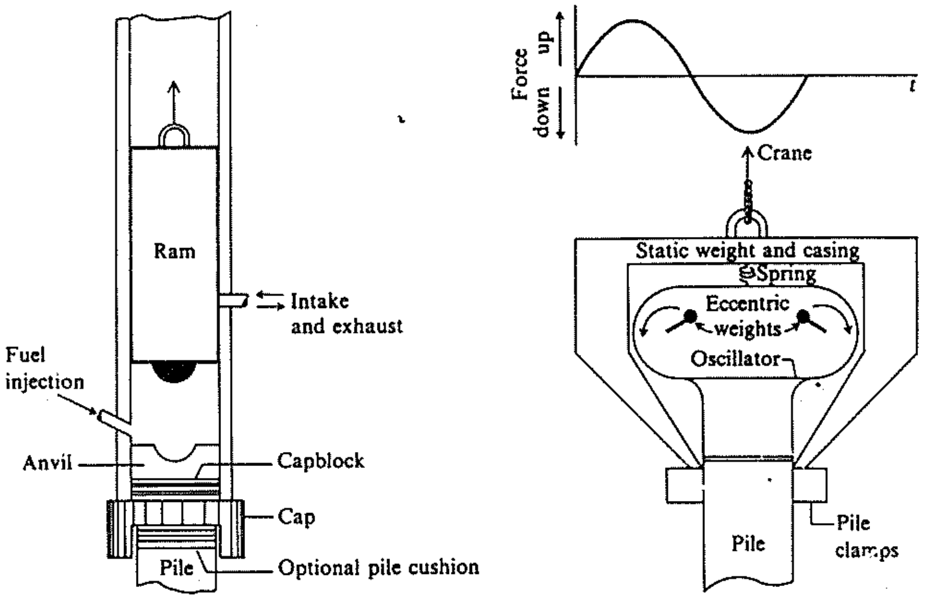
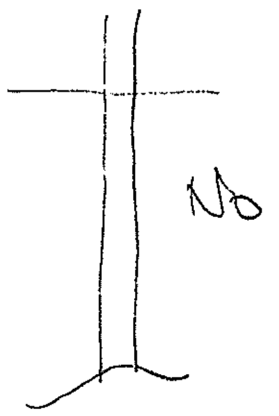
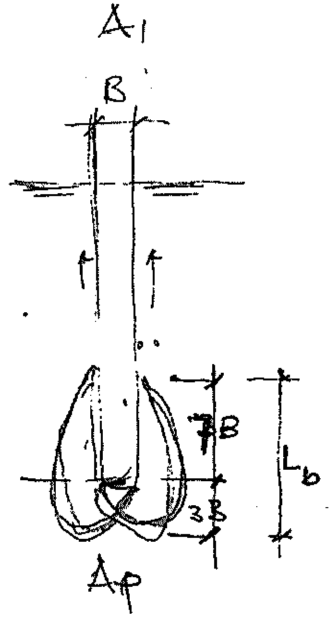


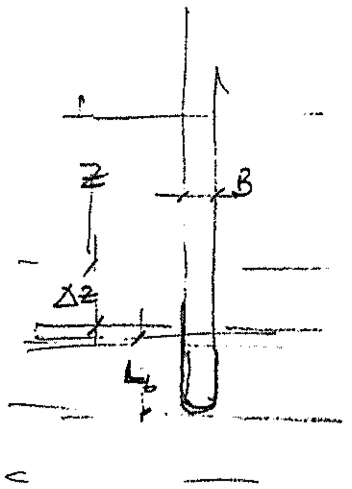
FIGURE 17.1
Schematics of several pile hammers.

(*c*) Diesel hammer. Crane initially lifts ram. Ram is released and falls; at select point fuel is injected. Ram collides with anvil, igniting fuel. Resulting explosion drives pile and lifts ram for next cycle.

(*d*) Vibratory hammer. External power source (electric motor or electric-driven hydraulic pump) rotates eccentric weights in relative directions shown. Horizontal force components cancel—vertical force components add.







NEI VARI STRATI:

NB: - se il suolo è granulare $c = 0$
$\varphi = \varphi'$ (drenato)
$\gamma = \gamma'$ (peso unit. terr. immerso)
pressione efficace

- se il suolo è coesivo $c \neq 0$
$\varphi = \varphi$ (undrenato) piccolo
$\gamma = \gamma_{sat}$ (peso unit. dell'ammasso sat.)
pressione totale

ultimo

area punta

SPT medio (assumere N_{55}) nella zona portante alta L_b

← 10B

$$P_{pu} = A_p (40 \cdot N) \cdot \frac{L_b}{B} (kN) \text{ (terreni granulari)} \; ; \; P_{pu} = (\overset{N_c}{9} c + \overset{N_q}{1} q_{op}) A_p$$ (terreno dotato di coesione e attrito)

deriva da $q_u = c\overset{9}{N_c} + \bar{q}\overset{1}{N_q} + \frac{1}{2}\gamma B \overset{0}{N_\gamma}$

$$P_{si} = \text{perimetro} \times \Delta z \times f_{si}$$

↑ contributo azione laterale

$$f_{si} = \alpha c + \overset{\gamma z}{q_o} K \, tg \overset{2/3 \varphi}{\delta}$$

α generalmente assunto 0.4 se $\varphi \neq 0$ e 1 se $\varphi = 0$

K generalmente assunto $= K_0 = (1 - \sin\varphi')\sqrt{OCR}$

pressione laterale senza spostamento

angolo d'attrito drenato

overconsolidation ratio = 1

$$P_u = P_{pu} + \sum P_{si}$$ (non del tutto corretta)

$$P_a = P_u / 2 \div 4$$

spesso 2.5

La mobilitazione della portanza di punta ultima richiede un abbassamento della punta del 10% di B per pali battuti e del 30% di B per pali per perforazione mentre la resistenza laterale ultima è mobilitata per abbassamenti minori (5÷10 mm)

$$P_u = \frac{e_h W_r h}{S + \frac{1}{2}(K_1+K_2+K_3)} \quad \frac{W_r + n^2 W_p}{W_r + W_p}$$

e_h = efficienza dell'apparato

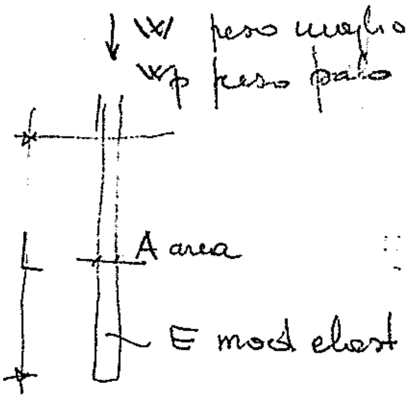
K_1 K_2 K_3 compressione elastica di cappello, palo, suolo

n = coefficiente di restituzione

W pesi (mazza, palo)

h altezza di caduta

S avanzam.to della p.ta per colpo



$\mathcal{E}_m$ = en. maglio per un colpo = W · h (alt. di caduta) o data dal costruttore per magli a vap. o diesel

$\bar{e}$ = efficienza del battipalo (0.7 ÷ 1)

r = rifiuto medio su 10 colpi (da misurare in una ripresa di infissione circa due o tre giorni dopo)

$$P_u = \frac{\bar{e}\,\mathcal{E}_m}{r\,K}$$

$$K = C\left(1 + \sqrt{1 + \frac{\lambda}{C}}\right) \qquad \lambda = \frac{\mathcal{E}_m L}{A E r^2}$$

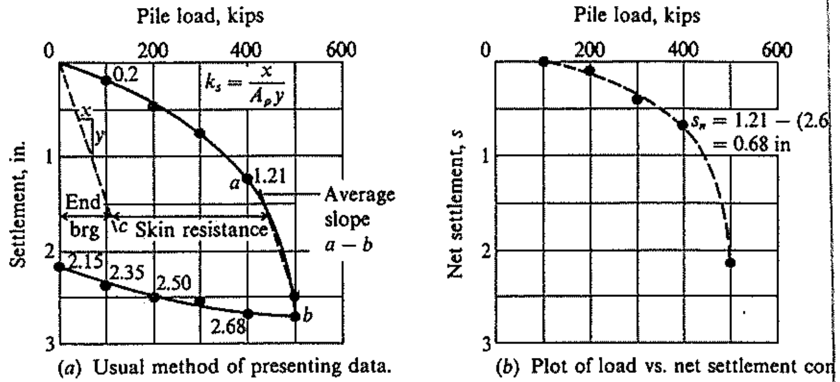
$$C = 0.75 - 0.15 \frac{W_p}{W}$$

formula di Janbu
(Colombo pag 188)

spans across the test pile and is securely attached to the reaction piles. capacity jack is placed between the reaction beam and the top of the tes produce the test load increments. The general set-up (Fig. 17-6*c*) is simila plate load test shown in Fig. 4-8 with the plate being replaced by the pile. has been standardized as ASTM D 1143; however, local building cod stipulate the load increments and time sequence. Somewhat similar me used to test laterally loaded piles. Here the lateral load may be applied by two adjacent piles apart or suitably connecting several piles for the lateral r

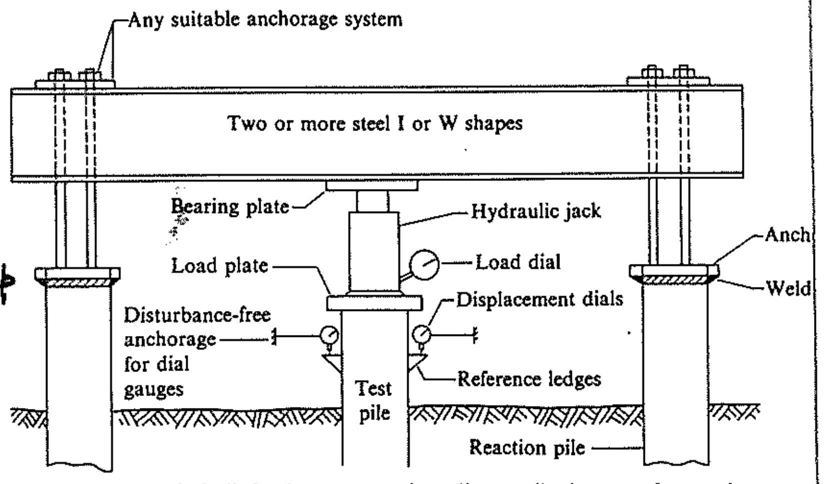
Figure 17-6 illustrates typical data from a pile-load test. Figure 17-6

FIGURE 17-6 Pile-load test data. This is actual pile-load test for pile *C* of Prob. 16-7 with data shown in F Pile is 14-in-diameter pipe with 0.312-in wall and 50 ft long. (*a*) Usual method of presenting plot of load versus net settlement computed as shown on the figure. (*c*) Typical pile load test se adjacent piles in group for reaction.



(*a*) Usual method of presenting data.

(*b*) Plot of load vs. net settlement co as shown on the figure.



(*c*) Typical pile load test setup using adjacent piles in group for reaction.

$P_a = \frac{1}{2} P^*$

P^* = forza che comporta

$\delta \leq 0.005$ in/Kip

1 inch = 2.54 cm

1 Kip = 453,6 Kgf

usual plot for a load test [and using kips—many persons continue to use tons (of 2000 lb) to describe pile capacity].

The ultimate pile load is commonly taken as the load where the load-settlement curve approaches a vertical asymptote as for the 500-kip load shown in Fig. 17-6*a*, or the load corresponding to some amount of butt settlement, say 25 mm and based on the general shape of the load-settlement curve, design load of the pile, and local building code (if any). The load-settlement curve must be drawn to a suitably large settlement scale so that the shape (and slope) is well defined. Referring to Fig. 17-6*a*, it is self-evident that reducing the vertical scale by a one-half factor would make it very difficult to determine that the curve is becoming nearly vertical between the 400- and 500-kip load.

An alternative method of interpreting Fig. 17-6*a* is based on the concept that the load is carried mostly by skin resistance to a shaft slip (butt settlement) sufficient to mobilize the limiting value. When the limiting skin resistance is mobilized the point load increases nearly linearly until the ultimate point capacity is reached. At this point further applied load results in direct settlement (load curve becomes vertical). Referring to Fig. 17-6*a*, these statements translate as follows:

1. From 0 to point *a* the capacity is based on the skin resistance plus any small point contribution. The skin resistance capacity is the principal load-carrying mechanism in this region. Point *a* usually requires some visual interpretation since there is seldom a sharp break in the curve.
2. From point *a* to *b* the load capacity is the sum of the limiting skin resistance (now a constant) plus the point capacity.
3. From point *b* the curve becomes vertical as the ultimate point capacity is reached. Often the vertical asymptote is anticipated (or the load to some value is adequate) and the test terminated before a "vertical" curve branch is established.

This concept was introduced by Van Weele (1957) and is only more recently beginning to be used [e.g., Brierley et al. (1979), Leonards and Lovell (1979), among others]. According to Van Weele if we draw the dashed line 0 to *c* through the origin and parallel to the point capacity region from *a* to *b* the load-carrying components of the pile are as shown on Fig. 17-6*a*. In this figure we have at $\Delta H = 1$ in the load carried as follows:

Point = 60 kips
Skin resistance = 290 kips
Total = 350 kips shown on figure

Local building codes usually stipulate how the load test is to be run and interpreted and pile design loads above which a load test is required (usually $P_d > 50$ kips). For example, the Chicago, Atlanta, and New York building codes stipulate the test as follows:

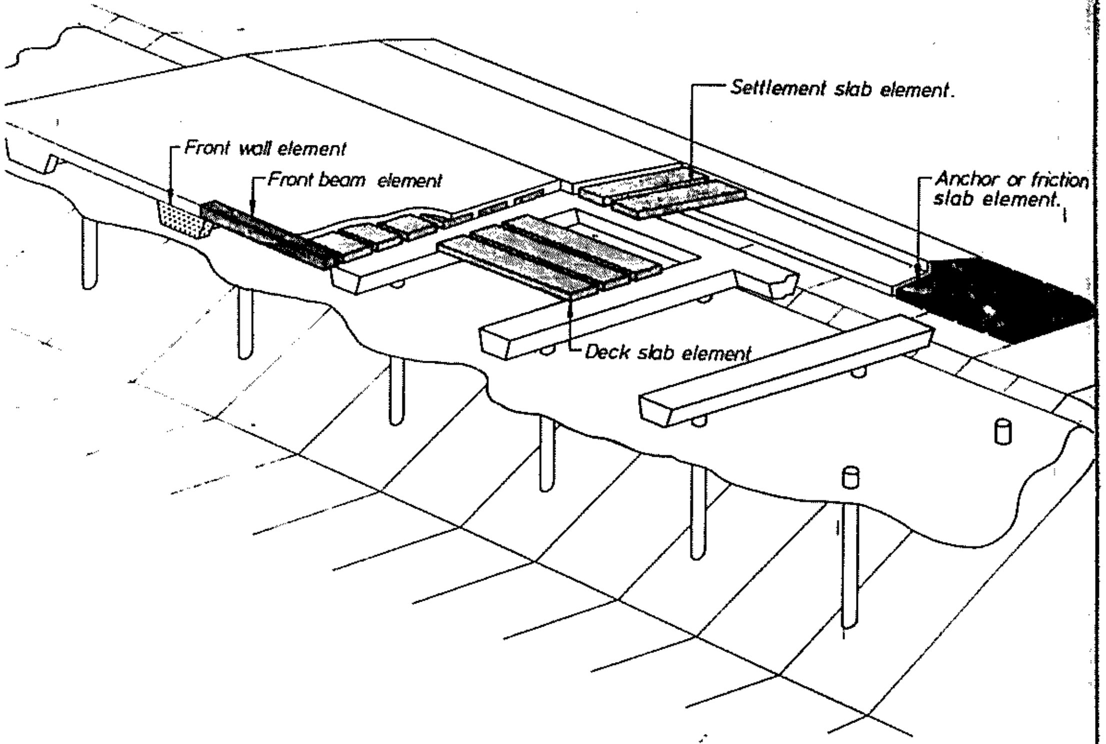


Fig. 3.6.4.E Quay with non-prestressed elements

Which of these systems to be chosen is a question that has to be solved in each separate case. Important factors are then the expected lifetime of the quay, durability requirements and the contractor's experience and equipment.

It generally applies that the formwork should have an over-height at mid-span corresponding to the deflection due to dead weight of formwork and concrete. The top of the slab should, with a view to the practical use of the quay, lie about 50 cm above the highest high water observed and it must also be put at a level high enough to permit the beams under it to be concreted in a dry form.

To resist traffic wearing on the slab it should be provided with a protective pavement on top. If made of concrete, this top layer can either be placed together with the concreting of the slab itself thus constituting a 3 – 5 cm additional part of the cast-in-situ monolithic slab, or it can be made separately, after the curing of the slab, as an 8 – 10 cm reinforced top slab. Generally, the first method is recommended, but if very difficult weather conditions can be expected during the concreting, the top layer should be placed at a later stage under more favourable weather conditions.

To ensure good surface drainage, the top layer should be provided with a slope of between 1:60 and 1:100 as part of a drainage system. When working out the quay slab details due regard must be taken to the installation of quay equipment such as fenders, bollards, sockets for power and telephone, water outlets, etc. If the quay slab exceeds a certain length, expansion joints must be provided at certain intervals perpendicular to the quay front. Usually their spacing is about 60 m, but this depends very much on the system used for the lengthwise anchoring of the quay. If the quay is anchored by a lamella wall at the middle of the quay stretch, it is quite possible to make the quay 100 – 200 m long without providing any expansion joint. Horizontal forces acting perpendicularly to the joints have to be absorbed by some kind of indentation.

3.6.5 *Erosion Protection*

The erosion protection of the stonefill slope under an open pile berth structure, as illustrated in figure 3.6.5.A, will depend upon the angle of the slope, the coarseness of the materials in the front of the filling, the danger of erosion from wave actions at the upper part of the filling and the danger of erosion from propeller current from the main ship propellers and the ship bow and stern thrusters at the lower part of the filling as illustrated in figure 3.6.5.B. The introduction of the ship bow and stern thrusters around 1960 was due to the need for increasing the ship's manoeuverability and thereby minimizing its manoeuvering time in the port.

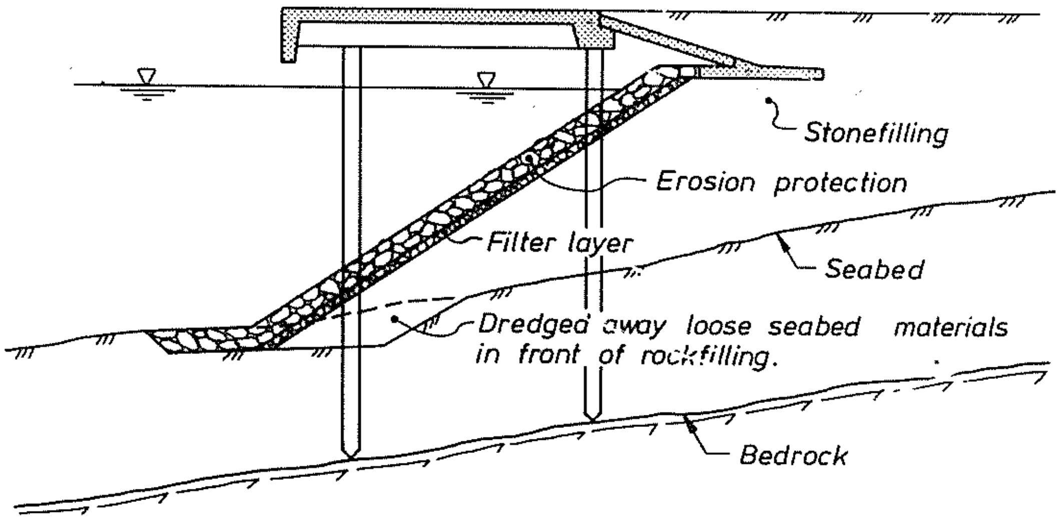


Fig. 3.6.5.A Erosion protection of stonefilling

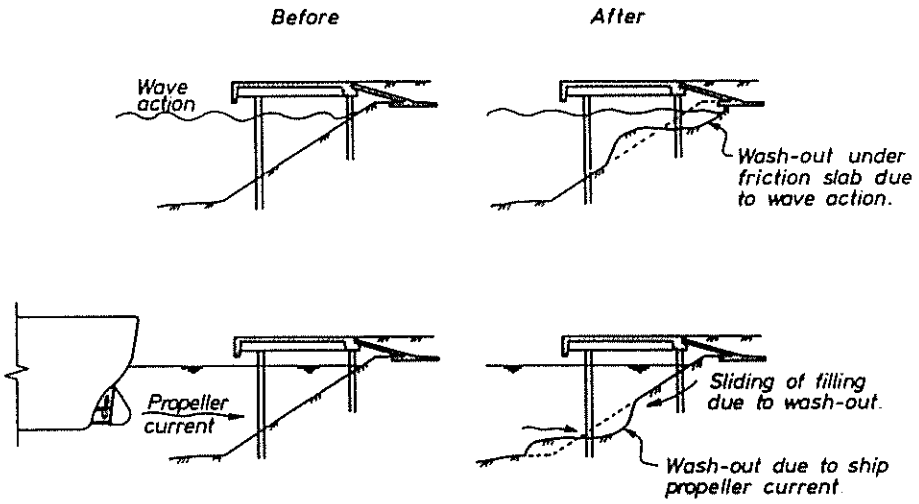


Fig. 3.6.5.B *Erosion due to wave action and ship proppeller current*

The design of the erosion protection due to wave action of the front of the stonefilling, can be based on the Hudson formula:

$$W = \frac{\gamma_s \cdot H_{des}^3}{K_D \left(\frac{\gamma_s}{\gamma_w} - 1\right)^3 \cot a}$$

where:

W = Individual block weight in kN
H_{des} = Design wave height = H_s to 1,4 H_s
γ_s = Specific gravity of block unit of quarrystone = 26 kN/m³
γ_w = Specific gravity of sea water = 10,26 kN/m³
a = Slope angle of the cover layer
K_D = Shape and stability coefficient which varies primarily with the shape of the block unit. For quarrystone and nonbreaking wave:
berth front = 3,2
berth end = 2,3

The thickness of the protection layer should not be less than about 1,0 to 1,5 metres. In addition to quarrystone, the following materials can also be used as protection: reinforced concrete units like Tetrapod and Dolos, wire boxes like Grabions, etc. Between the core stonefilling and the protection layer a filter layer should be constructed. The thickness and the size of the stones in the filter layer will depend on the materials in the core stonefilling. A detail of the erosion protection is shown in figure 3.6.5.C.

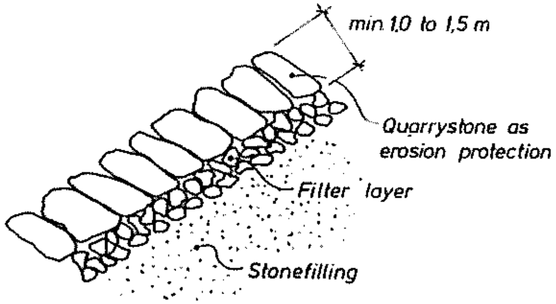
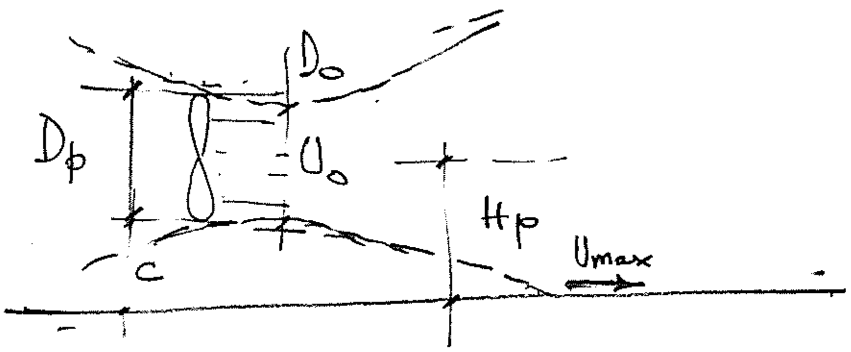


Fig. 3.6.5.C *Detail of erosion protection*

The exact design of the erosion protection against the main ship propellers and the bow and stern thrusters action, is very difficult since the size and type of protection will depend on the velocity of the propeller current which again will depend on the ship engine power, the speed and shape of the propeller and the diameter of the propeller. To get all this information for all the different ships calling the berth will be impossible. The experience however, has shown that if the erosion protection due to the propeller action has the same stone sizes and filter as due to the wave action from a disign wave height of about 1.5 to 2.0 m, this will in most cases give sufficient erosion protection. For a further technical study, see the PIANC Bulletin No. 58.

In case of soft seabed in front of the stonefill slope under an open pile berth structure, or where the propellers may act against a solid guaywall resulting in a strong downward current, the erosion protection layer should be extended at least 3 to 5 metres out in front of the berth line.



P_d, potenza utilizzata in manovra = 10% P_d max

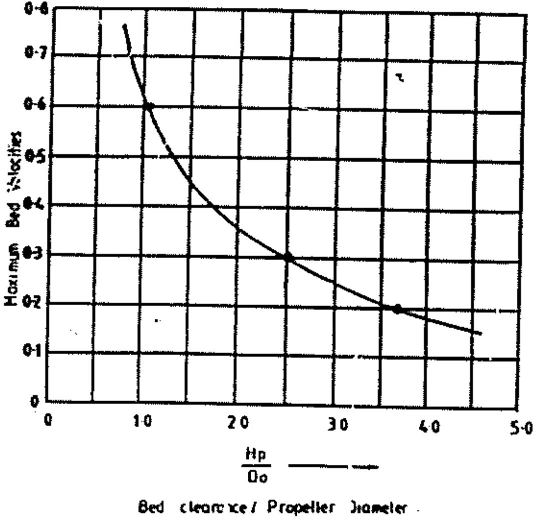
$U_o = \alpha \left[\frac{P_d}{D_p^2}\right]^{1/3}$ $\alpha = 1.48$ elica non intubata

$\alpha = 1.17$ elica intubata

$D_o = \beta D_p$ $\beta = 0.71$

$\beta = 1$ idem

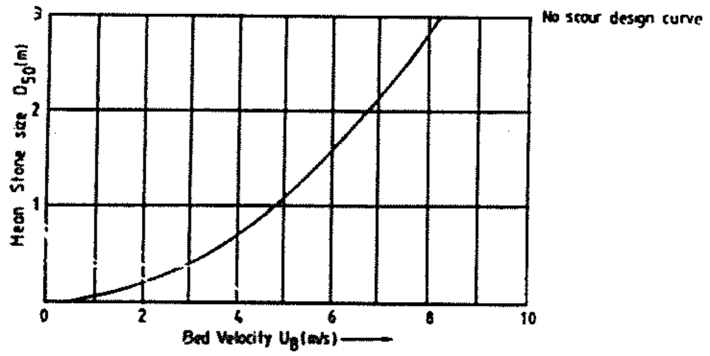
$H_p = c + D_p/2$ $H_p/D_o \xrightarrow{7.1} \frac{U_{max}}{U_o}$ $U_{max} \xrightarrow{7.2} D_{50} \begin{cases} 0.5 \\ \updownarrow W_{50} \\ 2 \end{cases}$



Note :
1. Ship with single propeller and rudder at zero deflection without obstruction to flow.
2. Rudder with vertical axis mounted astern of propeller

Source : Ref. 11

Figure 7.1 - Velocity at Bed Level



Note :

(i) Data given for stones of specific weight Ws = 2650 kg/m^3 and sea-water, W = 1026 kg/m^3, D_{50} = U2/C.

(ii) For stones of different specific weight multiply D by (Ws - W)/1.58 W.

(iii) For sloping banks increase D_{50} by 50%.

Source : Ref. 18, which is based on Hydraulic Design Criteria Sheet 712-1 published by US Army Corps of Engineers (revised 9-70)

Figure 7.2 - Stone Size to Give no Scour for Given Bed Velocity

Esempio

Traghetto lungo 60 ÷ 70 m

per eliche 3000 KW (300 in manovra)

$D_p = 2.5 m$ (non intubato)

$c = 0.50 m$

$$U_0 = 1.48 \left(\frac{300}{2.5^2}\right)^{1/3} = 5,40 \, m/s$$

$D_0 = 0.71 \times 2.5 = 1.78 \, m$

$H_p = 0.5 + {}^{2.5}/_{2} = 1.75$

$H_p / D_0 = {}^{1.75}/_{1.78} \simeq 1$

$U_{max} / U_0 = 0.6$

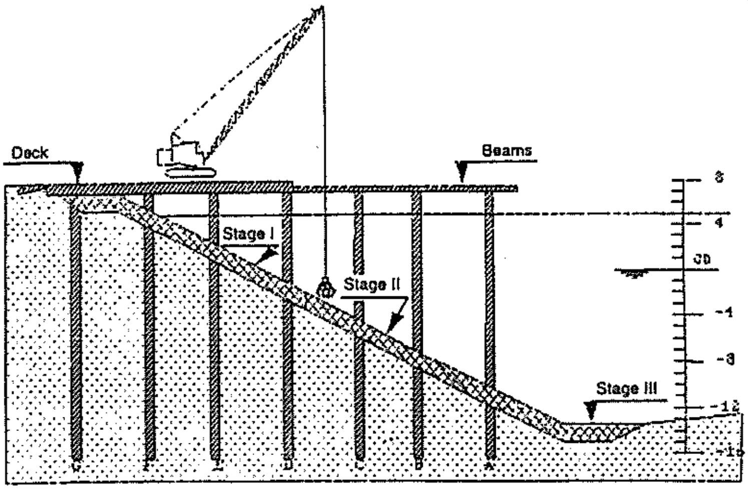
$U_{max} = 5.40 \times 0.6 = 3.24 \, m/s$

$D_{n50} = 0.5 \, m$

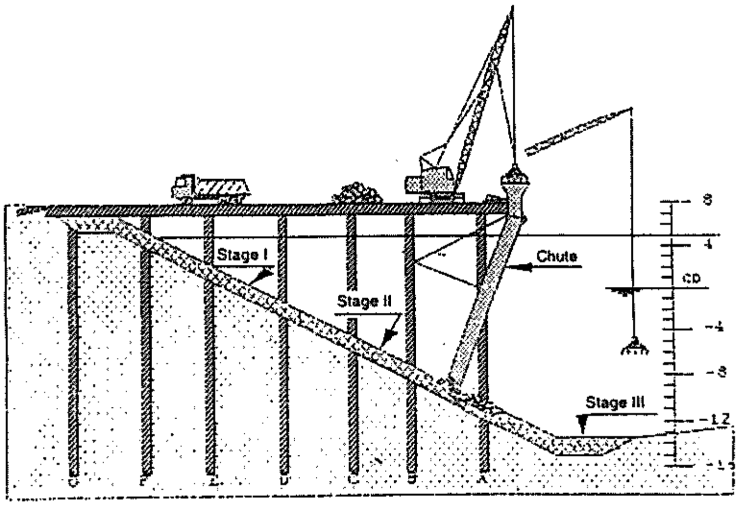
$M_{50} = 331$ kg sul piano
- 165 kg (200)
- 660 kg (700)

$D_{n50} = 0.75 \, m$

1118 kg sulla scarpata ≃ 1000 kg
- 500 kg
- 2000 kg



A - Using Hydraulic Bucket



B - Using a Chute

Figure 9.1 - Beira - Construction Sequence

Fig. 9.1 shows that the work was carried out in three stages, namely :

Stage I

Place upper length of slope protection
Construct deck to approximately half its width.

Stage II

Place lower length of slope protection.
Complete the deck.

Stage III

Place toe of slope protection by means of steel chute, consisting of an 0.8-m-diameter steel tube, rather like a large tremie pipe for concrete, which can be inclined by a hydraulic ram so that rip-rap can be placed under the seaward face of the deck of the quay. The chute was supported on a counterweighted rail bogie, travelling on rails parallel to the face of the quay. The pipe was braced below deck level to the piles. Rip-rap was dumped into a hopper on the top of the chute by crane grab.

Horizontal and Vertical Control

Hand lead soundings were used on a very close grid, with a spacing nearly equal to the bucket diameter. Because of the strong tidal currents, surveying was limited to periods of slack high water and slack low water, but at spring high tide it was not possible for the survey boat to pass under the deck beams, which restricted survey work to periods of low water slack tide.

As a result, production rates were limited by delays due to interruptions in the survey work. It is estimated that idle time was about 60% of the working time. Average production rates per 10 hour shift, including the effects of idle time, were 40 m^3 for placing by crane and grab/bucket and 60 m^3 for placing by chute.

9.3 Port of Rotterdam, Netherlands

Project Title :	Caldac Chemie Quay, Caland Canal
Employer :	Caldac Chemie NV
Contractor :	Van Oord ACZ BV
Date :	1991
Tides and Currents :	Canal water level range -0.65 m to +1.05 m above marine datum Currents - slight
Type of Slope Protection :	Rip-rap at 1:2.9 slope on underlayer on fascine mattress Primary layer : 10-60 kg, 0.5 m thick Underlayer : 10-125 mm, 0.2 m thick Fascine mattress including geotextile of 325 gr/m^2
Embankment Material :	Sand and clay
Canal Bed Material :	Silty sand

Construction Sequence

The completed work is shown in Fig. 3.4.

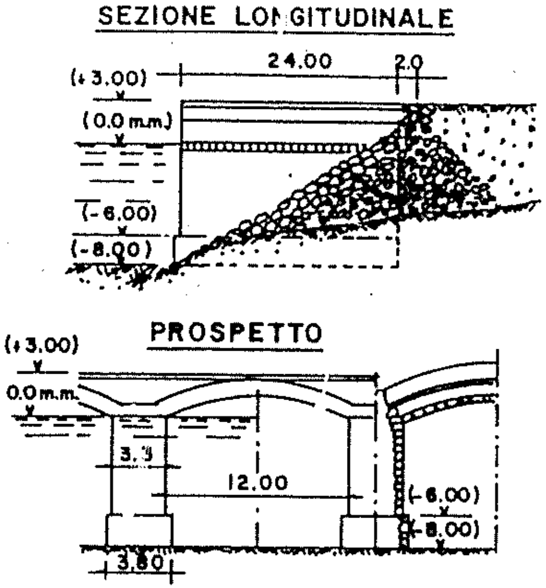


Fig. 24 a)

ESEMPIO DI MURO DI
SPONDA A SPERONI
ED ARCHI

Fig. 24 b)

ESEMPIO DI MURO DI SPONDA
A SPERONI CON IMPALCATO
DI COLLEGAMENTO