In questa seconda parte del corso di

COSTRUZIONI MARITTIME E PORTUALI

si esaminano le opere di accosto e ormeggio delle navi, in generale ubicate in zone riparate all'interno dei porti salvo alcune particolari come i pontili in rada o gli ormeggi isolati per le rinfuse liquide.
Se ne illustrano le applicazioni portuali e per ciascuna tipologia di opere si forniscono schemi di calcolo elementari (il cosiddetto"colpo di regolo" che controlla il risultato del computer). Tali schemi hanno il solo scopo di introdurre alla comprensione del comportamento statico della struttura in esame; essi possono eventualmente consentire un primo approccio al proporzionamento, ma solo per una verifica di applicabilità della struttura di cui si ipotizza l'adozione.

Ovviamente per la progettazione vera e propria, con gli opportuni procedimenti e software specialistici, si rimanda ai relativi Corsi di insegnamento.

OPERE ed IMPIANTI DI ACCOSTO

1 - Le opere od installazioni, sia portuali che off-shore, per l'accosto delle navi consentono (in misura totale o parziale) il carico e lo scarico delle merci (o dei passeggeri), il deposito delle merci, il rifornimento e la riparazione (in fase galleggiante), ovvero la sosta di attesa, delle navi.

2 - Vengono così ad assolvere ~~sia nella totalità che in limitata misura~~ compiti funzionali diversi, quali:

- fornire un dispositivo d'appoggio o di ormeggio alle navi;
- consentire l'installazione ed il funzionamento di mezzi terrestri di carico/scarico delle merci (gru, scaricatori, pompe, ecc.);
- assicurare il collegamento tra nave e terraferma con i più diversi mezzi (viari, ferroviari, tubodotti, nastri, ecc.);
- consentire il deposito, sia di transito che di sosta, delle merci (magazzini, stoccaggio all'aperto, serbatoi, ecc.);
- fornire alla nave i necessari servizi (allacciamento reti acqua potabile e di zavorra, telefono, F.M., aria compressa, antincendio, ecc.).

3 - E' consueto distinguere ~~le~~ tali opere ~~di accosto~~ in:

a) ~~banchine~~ calate, ~~[illegible]~~ formate da piazzali ~~od terrapieni~~ (delimitati verso mare da banchine*) e ~~[illegible]~~li (distribuite negli sporgenti, o ponti, portuali, calate di riva, ecc.); ~~e che~~ che possono permettere di svolgere tutti i compiti di servizio e di deposito inerenti al traffico portuale - ricordati al soprascritto punto 2) - nella più ampia misura (offrendo sempre un accosto continuo alle navi);

b) pontili, che si differenziano dalle ~~banchine~~ calate essenzialmente per la loro assai limitata o nulla disponibilità al deposito delle merci, mentre possono assolvere a tutti gli altri compiti di ormeggio, collegamento a riva, ecc. (pur offrendo in molti casi spesso un accosto non continuo alle navi);

c) dolphins o ducs d'Albe, che sono opere isolate di per se stesse idonee alla sola funzione di appoggio, o di ormeggio, della nave (occorrono evidentemente più ~~[illegible]~~ dolphins per offrire un accosto); spesso impiegati per la formazione, la protezione, ecc. di opere portuali (pontili, o anche banchine) più complete.

4 - Come si può rilevare la distinzione tra i vari tipi di accosto é essenzialmente funzionale; in effetti, si ha spesso una distinzione strutturale, dovuta al fatto che gran parte dei pontili, ed a maggior ragione dei dolphins, non deve sopportare la spinta delle terre per assenza di terrapieno sopportato. Ne seguono differenze strutturali notevoli, ~~però non é una caratteristica del tutto generale~~.

5 - Costituiscono un complesso a parte gli ormeggi isolati, od in mare aperto (off-shore), tipo "campo di boe", torri o piattaforme di monormeggio, ecc., in cui non si ha un fronte di accosto, come nelle opere ricordate nei paragrafi precedenti, però la nave può - per alcuni particolari tipi di merci (essenzialmente idrocarburi) - svolgere operazioni di car~~di~~/scarica, con avviamento a terra, a mezzo "sea-line" poggiate sul fondo.

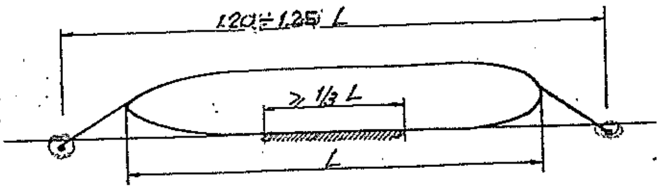
* - Si definiscono muri di sponda o di banchina o semplicemente banchine le opere che delimitano le calate verso mare.

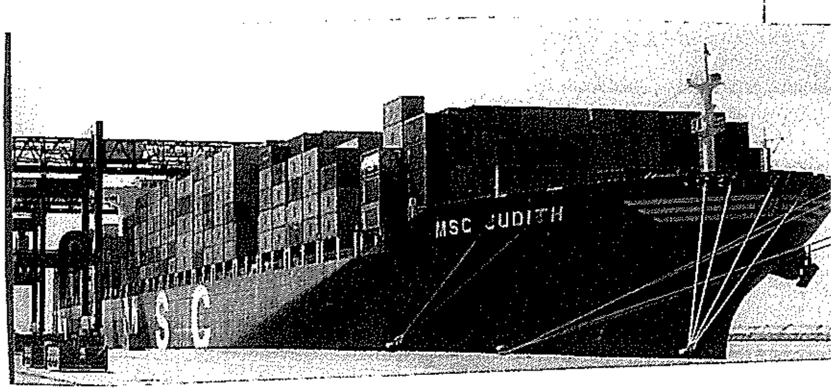
I muri di banchina ~~[illegible]~~ costituiscono il posto di accosto.

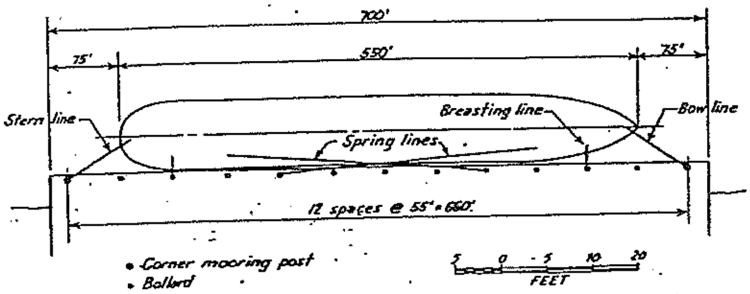
Posto di accosto

1) Per l'accosto di una nave si richiedono:

a) due punti rigidi (bitte e bittoni su banchine, su piattaforme o dolphin rigido – per es. dolphin K all'Allegato VI/M – ecc.), rispettivamente a prora ed a poppa della nave, alla distanza tra loro di almeno 1,20/1,25 L (essendo L la lunghezza della nave) a cui ammarare i cavi d'estremità della nave; tale distanza é necessaria per dare ai cavi stessi sufficiente lunghezza e quindi elasticità.







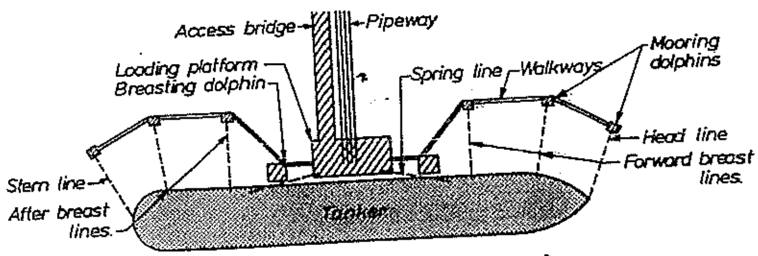
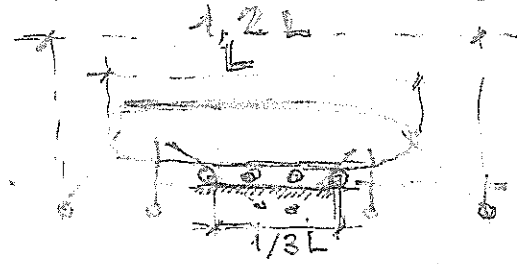


Fig. 2.3.5.F Typical berth for tankers

b) un fronte di accosto centrale lungo almeno 1/3 L, per evitare che la nave tenda a ruotare attorno allo stesso, tale lunghezza può elevarsi sino a 2/3 L se vi sono risacche, vento, ecc. rilevanti.
Questo tratto centrale può essere continuo come nelle banchine ma pure discontinuo cioé formato con elementi (dolphins elastici, paraurti, ecc.) convenientemente distanziati (per es. al pontile di Stoccolma – dell'Allegato VI/N – é fornito dalla piattaforma centrale, al pontile di Wilhamshaven – allegato VI/M – é fornito da una successione di 4 dolphins elastici – ciascuno a gruppo di 5 pali metallici, duc d'Albe H, ecc);

c) altri punti fissi (bitte) ai lati della zona centrale per l'ormeggio dei cavi intermedi quali traversini e springs (di stabilizzazione della nave alle oscillazioni dello specchio liquido); questi punti possono essere sistemati sullo sviluppo corrente continuo di una banchina (o pontile continuo) ovvero su dolphins rigidi intermedi, opportunamente intervallati, nel caso di un pontile di tipo discontinuo (vedere già citato pontile di Stoccolma ed i pontili di Levera (Marsiglia) e di Genova Multedo – Allegato VI/N; il pontile di Wilhelmshaven – Allegato VI/M, pontili dell'Allegato 300, ecc.)

OPERE DI ACCOSTO E ORMEGGIO



MURI DI BANCHINA (detti anche BANCHINE)

PONTILI (e JETTIES)

DOLPHINS (detti anche BRICCOLE E DUC D'ALBE) 3

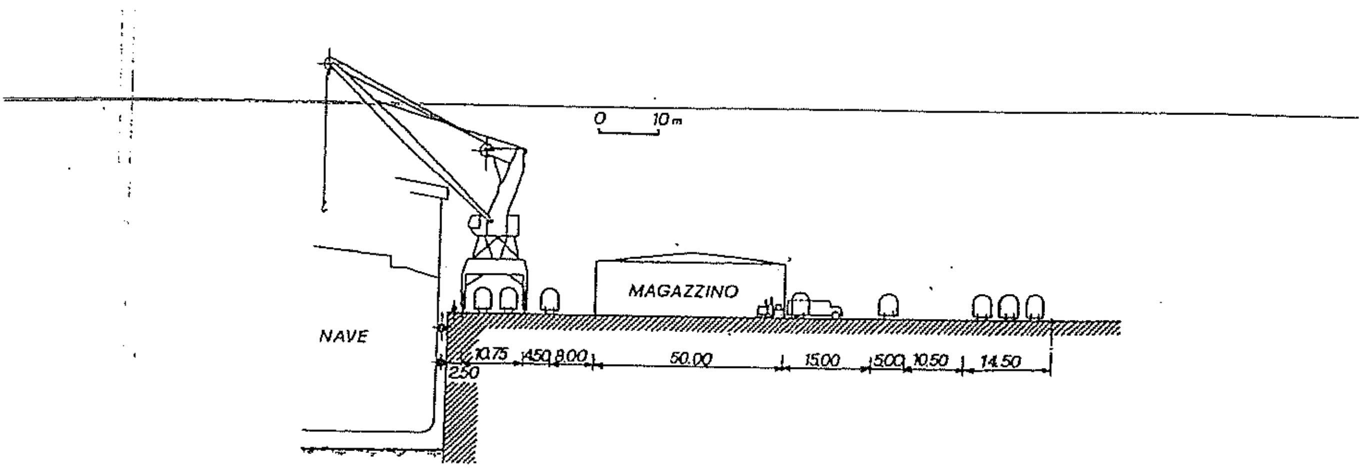


Fig. 117 –Schema di arredamento di una banchina.

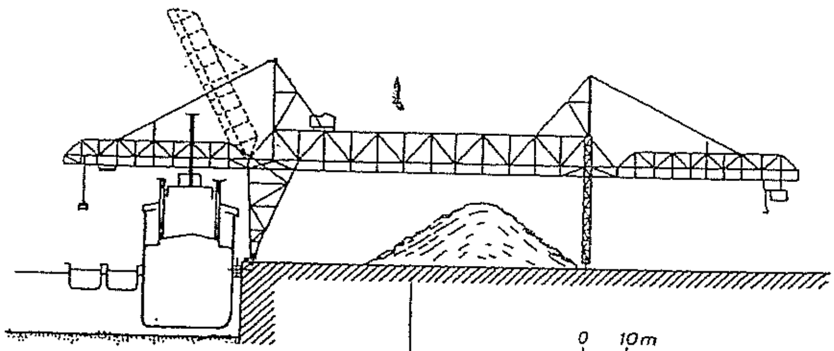
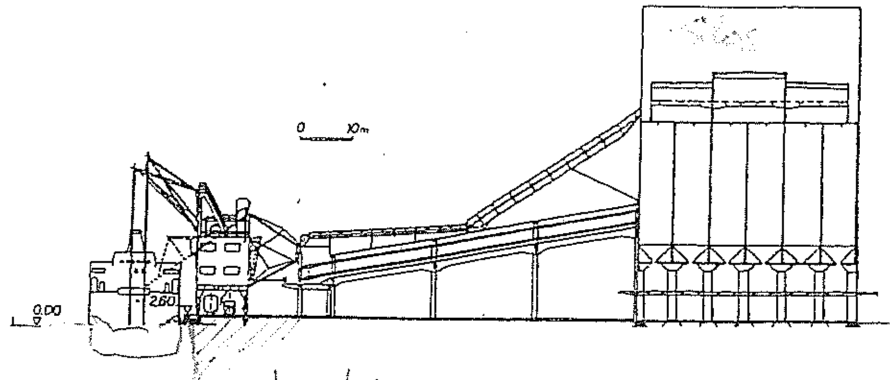
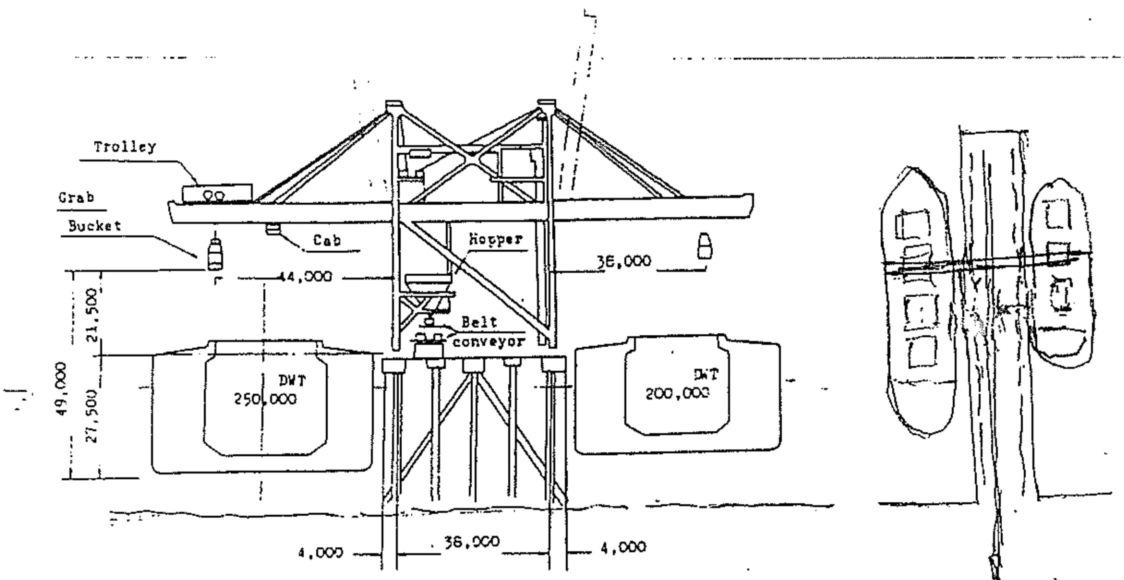


Fig. 116 - Porto di Rouen (*Francia*). Banchina per merci alla rinfusa.



banchina cereali



Pontile a sezione continua (Rinfuse secche)

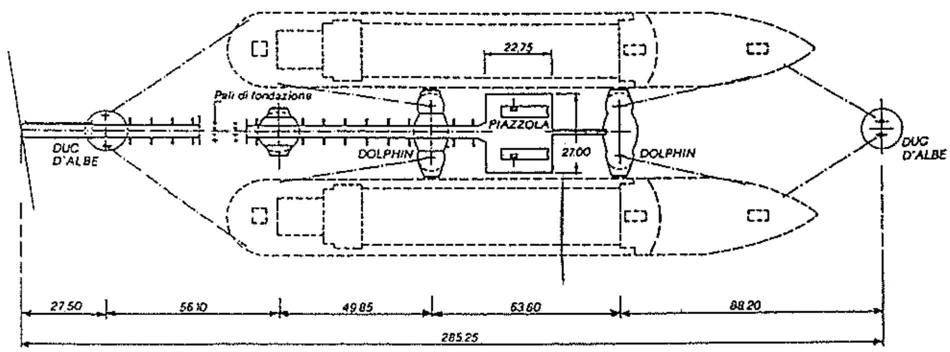
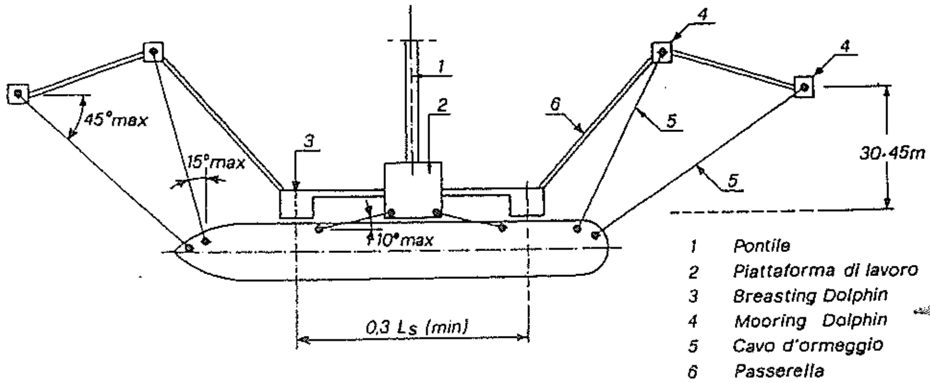


Fig. 97 - Porto di La Spezia. Pontile per petroli.



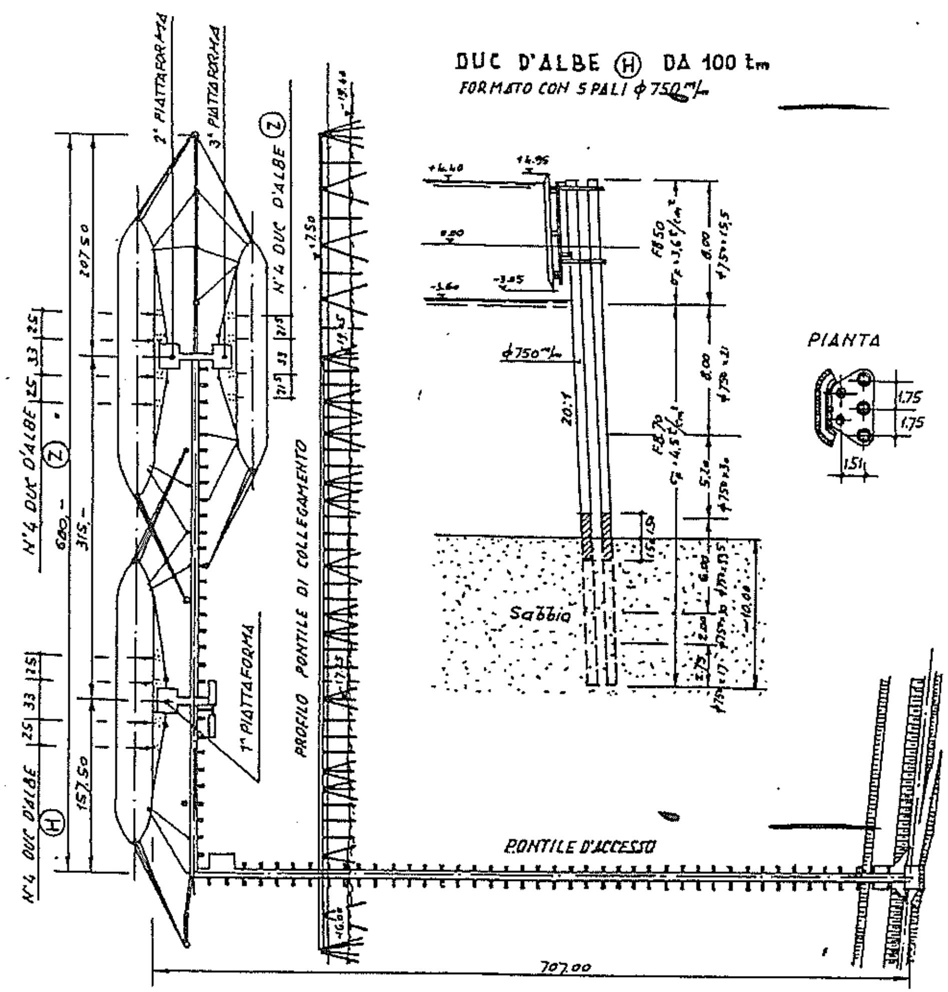
dolphin

Fig. 98 - Attracco per navi cisterna (tankers).

Jetty

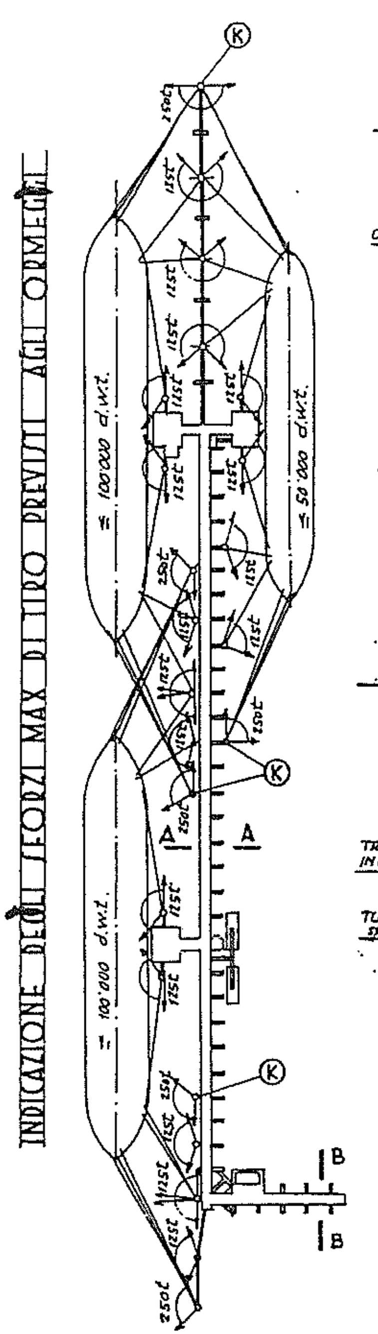
IMPIANTO DI SCARICO DI WILHELMSHAVEN (1958)

TERMINALE, DELL'OLEODOTTO WILHELMSHAVEN (COLONIA)

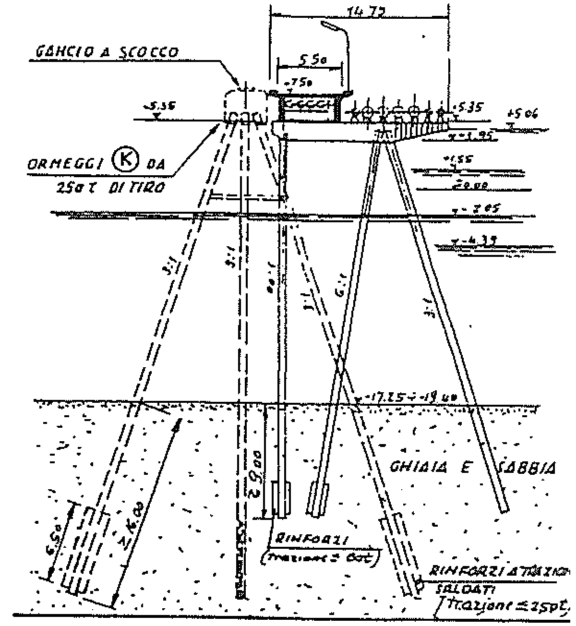


DUC D'ALBE (Z) DA 60 tm. SONO FORMATI CON 4 PALI : RINFORZATI A 100 tm CON AGGIUNTA EVENTUALE DI UN PALO INTERMEDIO

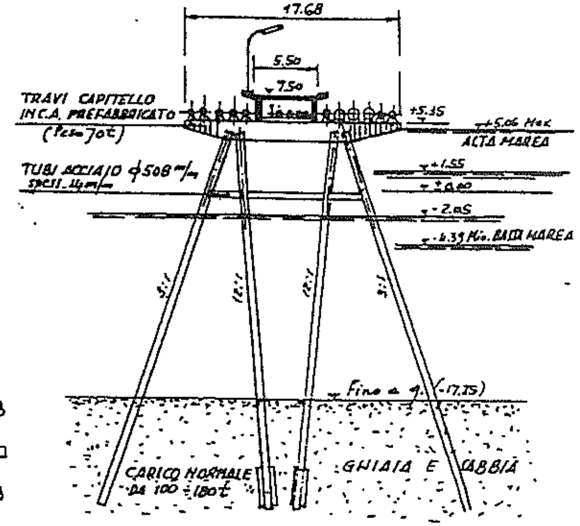
INDICAZIONE DEGLI SFORZI MAX DI TIRO PREVISTI AGLI ORMEGGI



SEZIONE A-A TIPO PONTILE DI COLLEGAMENTO TRA LE PIATTAFORME D'ACCOSTO



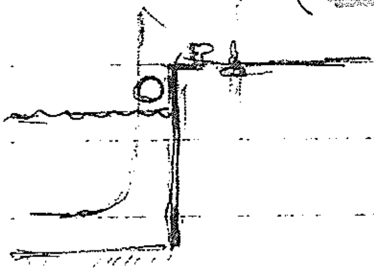
SEZIONE B-B TIPO - PONTILE D'ACCESSO



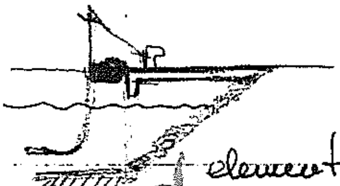
PALI TUBOLARI IN ACCIAIO φ566 m/m st.50
spessori: 13,5 ÷ 15,5 m/m RISERVA PER CORROSIONE

D 1221

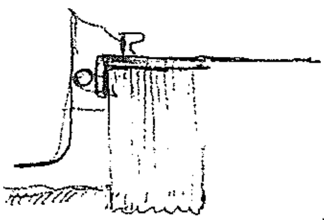
VARI TIPI DI BANCHINA (MURI DI



- a parete (continua)
  - di tipo pesante (a gravità)
    - a fondaz. superficiale (terreni buoni in sup.)
      - a piloni di massi prefabbricati { cavi o speciali (2)
      - a cassoni di c.a. prefabbricati (5) (sol. moderna)
      - muri veri e propri di cls. o di c.a. prefabbricati (10) o gettati in situ
    - a fondazione "prof." (tipologia in disuso)
      - a cassoni autoaffondanti (3)
      - affondati con l'ausilio dell'aria compr
  - di tipo "leggero" vincolate nel terreno
    - a palancole di ferro o di c.a. (pref. e infisse)
    - a paratia di c.a. o di c.a.p. (gettata in situ)



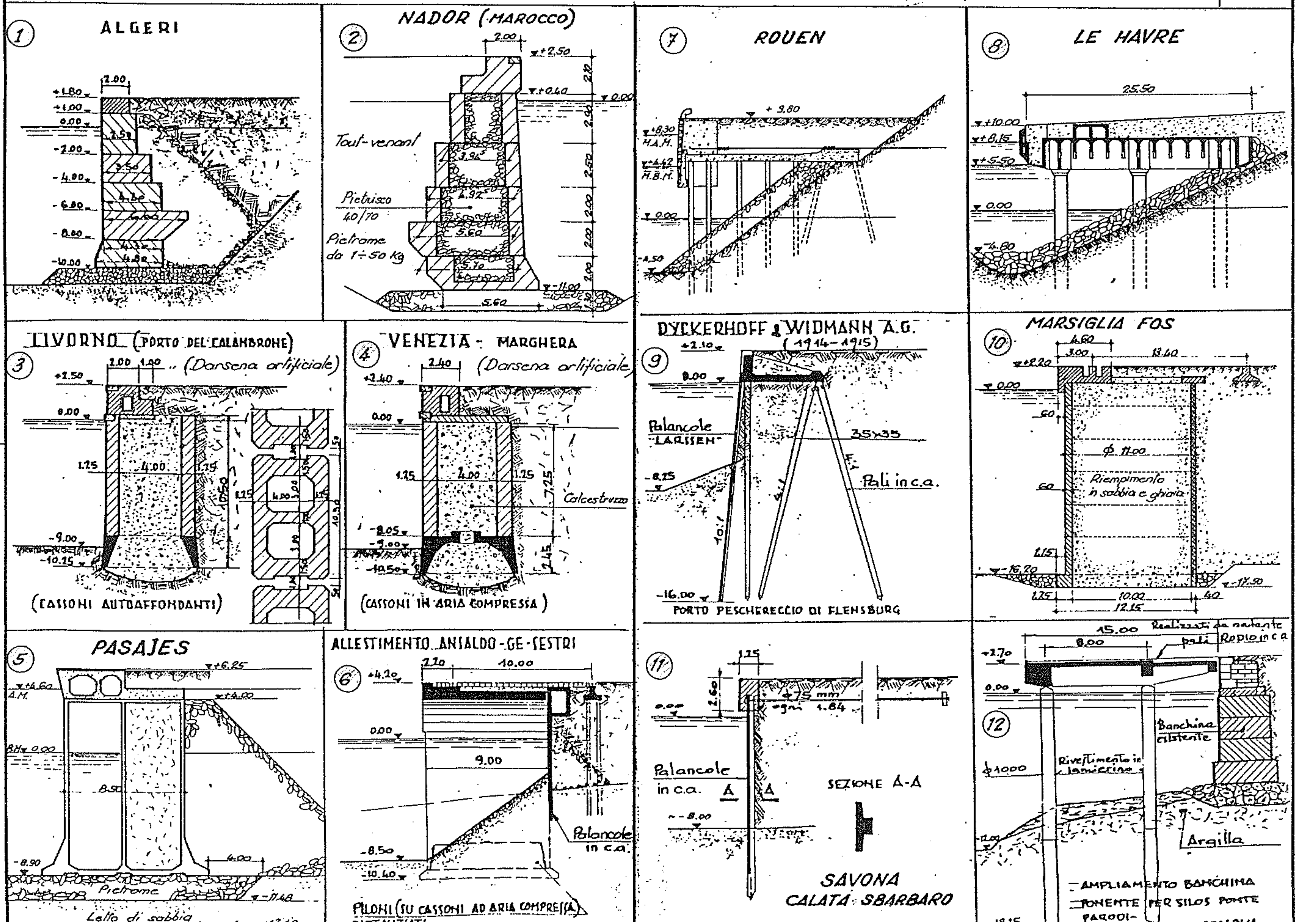
- a scarpata
  - elementi di sostegno
  - di tipo pesante (a gravità) con fond. sup. o "profonda" ma poco sotto la superficie del fondo
    - a speroni di massi con impalcato ad arco (tipologia in disuso) (6)
    - a speroni di cassoni di c.a. con imp. a solettone
  - su sostegni isolati (a giorno)
    - su pali infissi nel terreno (frequenti) (4)
    - su piloni con fondazione diretta o su pali con (se il terreno discreto è poco profondo)



- a gabbioni (di palancole metalliche piatte)

28/02/99

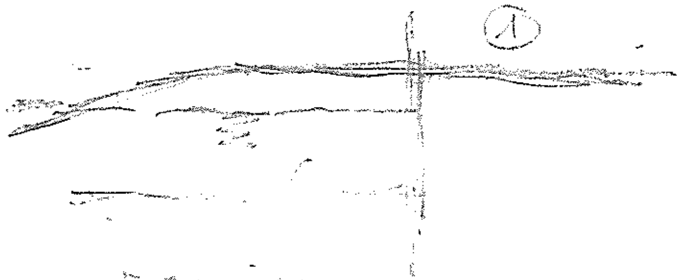
P. 2082



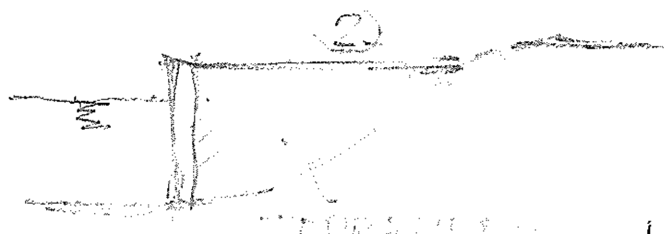
1 ALGERI
2 NADOR (MAROCCO)
Tout-venant
Pietrisco 40/70
Pietrame da 1÷50 kg
7 ROUEN
8 LE HAVRE
3 LIVORNO (PORTO DEL CALAMBRONE) (Darsena artificiale)
(CASSONI AUTOAFFONDANTI)
4 VENEZIA - MARGHERA (Darsena artificiale)
Calcestruzzo
(CASSONI IN ARIA COMPRESSA)
9 DYCKERHOFF & WIDMANN A.G. (1914-1915)
Palancole "LARSSEN"
Pali in c.a.
PORTO PESCHERECCIO DI FLENSBURG
10 MARSIGLIA FOS
Riempimento in sabbia e ghiaia
5 PASAJES
Pietrame
Letto di sabbia
6 ALLESTIMENTO ANSALDO - GE - SESTRI
Palancole in c.a.
PILONI (SU CASSONI AD ARIA COMPRESSA)
11
Palancole in c.a.
SEZIONE A-A
SAVONA CALATA SBARBARO
12
Banchina esistente
Rivestimento in lamierino
Argilla
AMPLIAMENTO BANCHINA PONENTE PER SILOS PONTE PARODI

ARGOMENTI CHE CONDIZIONANO LA SCELTA DEL TIPO DI MURO DI BANCHINA

- Destinazione (tipo di nave o imbarcazione)
- Profondità fondale (e marea)
- Caratteristiche fondale (buone, scadenti, superficialmente scadenti e buone al di sotto ecc.)
- Modalità di realizzazione (costruite all'asciutto su un terrapieno naturale o artificiale successivamente dragato ① o costruite in acqua con terrapieno realizzato successivamente ②)
- Risposta idraulica (assorbimento di energia)
- Disponibilità di materiale, mezzi d'opera, mano d'opera ...



ESCAVAZIONE E DRAGAGGIO



TERRAPIENO [illegible]

26/02/96

3.2 Design Loads

3.2.1 General

The design load for a limit state is defined as the most unfavourable combination of the characteristic load multiplied by a load coefficient. The limits states are categorized as follows:

- The ultimate limit state (ULS) is related to the risk of failure or large inelastic displacements or strains of a failure character.
- The servicability limit state (SLS) is related to criteria governing normal use or durability.
- The fatigue limit state (FLS) is related to the risk of failure due to the effect of repeated loading.
- The limit state of progressive collapse (PLS) is related to the risk of failure of the structure under the assumption that certain parts of the structure have ceased to perform their load-carrying functions.

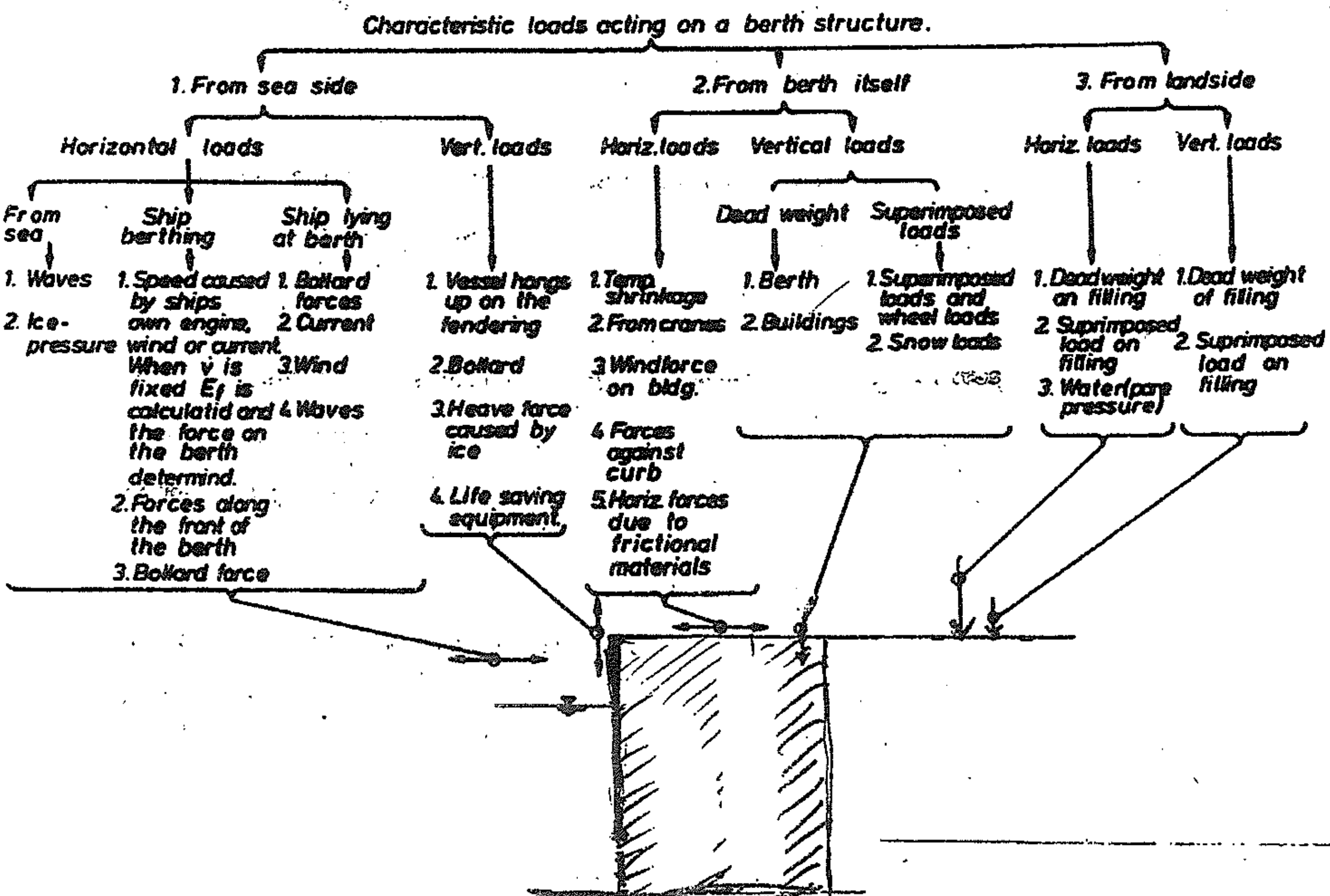
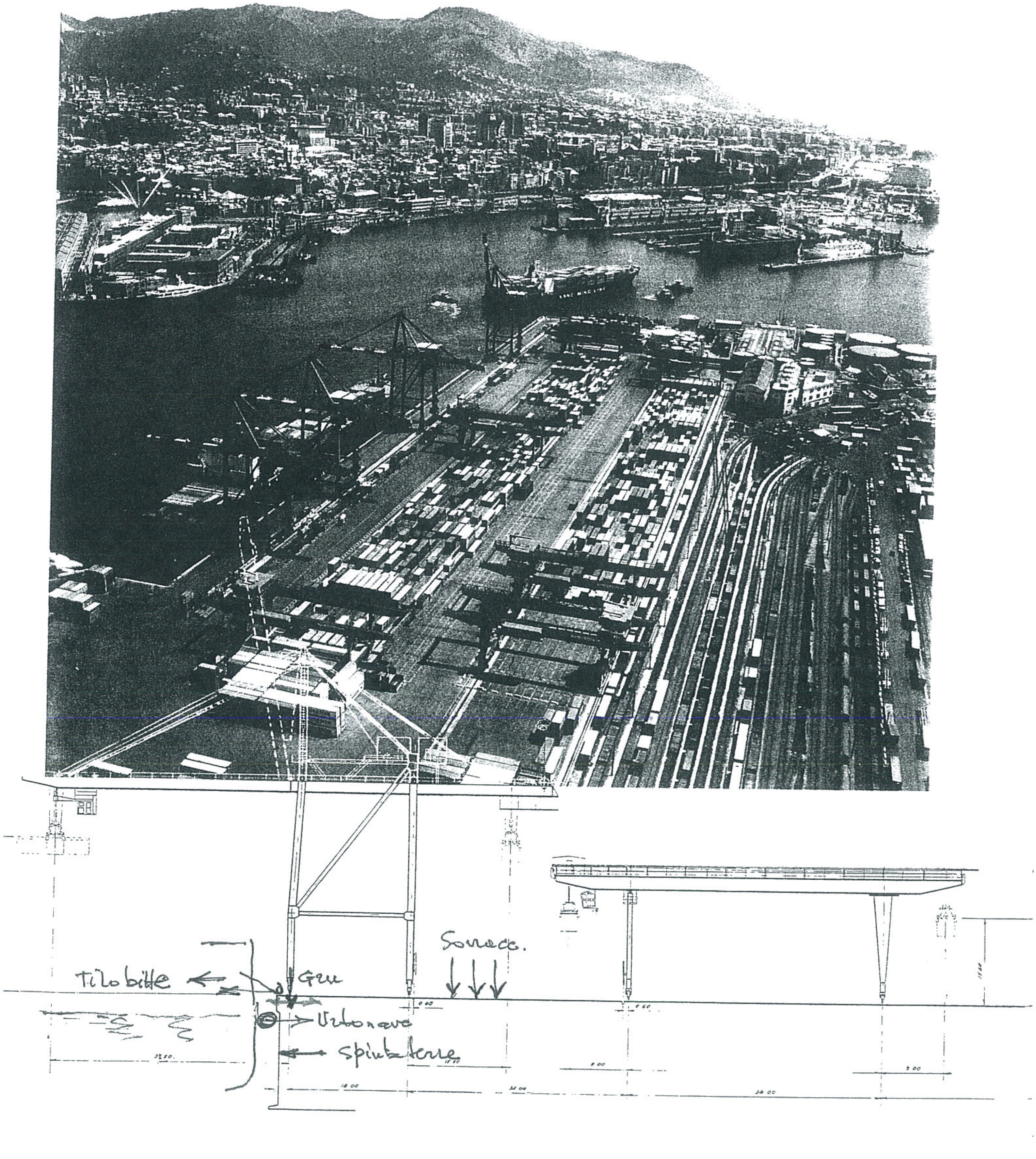


Fig. 3.2.1.A Characteristic loads acting on a berth structure

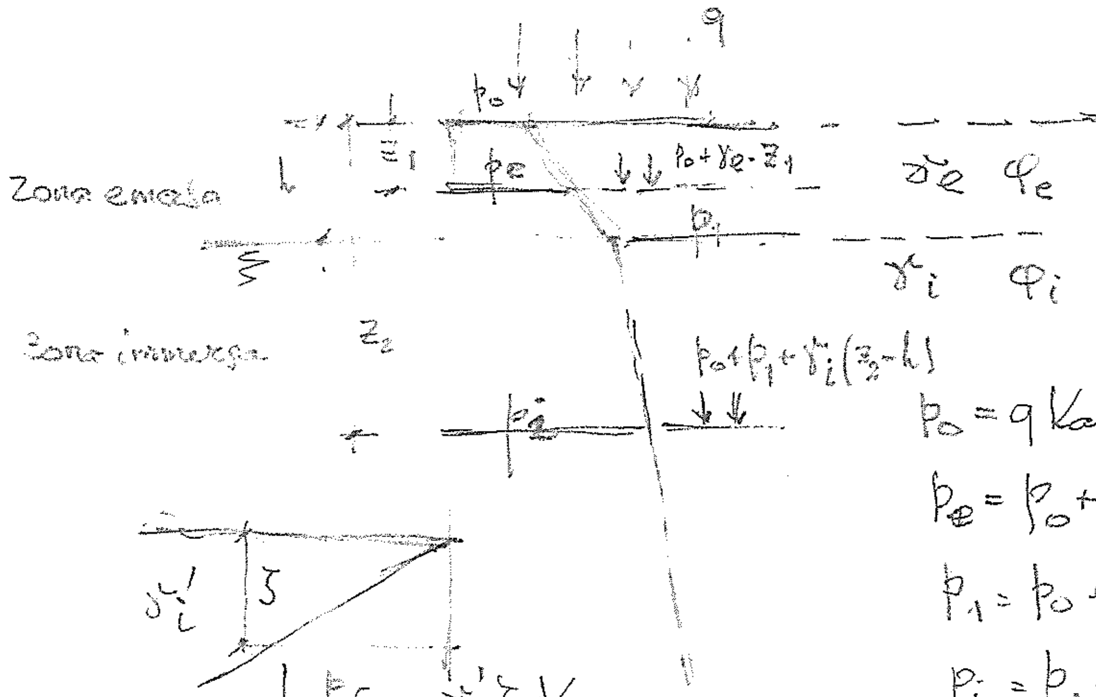
6



Sezione trasversale e, in alto, particolare del terminal visto da ponente.

Accanto, simulazione a video del posizionamento dei container, realizzata con il programma Set Graph della Sistemi e Telematica.

LATO TERRA — SPINTA TERRE



peso specifico e angolo d'attrito interno

$\varphi_e \simeq \varphi_i \simeq 30 \div 35°$

$$p_0 = q K_a$$

$$p_e = p_0 + \gamma_e z_1 K_a$$

$$p_1 = p_0 + \gamma_e h K_a$$

$$p_i = p_1 + \gamma_i (z_2 - h) K_a$$

$$= (q + \gamma_e h + \gamma'_i (z_2 - h)) K_a$$

$$p_F = \gamma'_i \zeta K_p$$

press. passiva che si instaura quando il muro si sposta

γ_e = peso specifico del rinfianco [illegible] asciutto

ad es $26,5 \times 0.65 = 17.2$ kN/m³ (≈35% vuoti)

γ'_i = peso specifico del rinfianco immerso (acqua 10 kN/m³)

ad es $26.5 \times 0.65 + 10 \times 0.35 - 10 = 10.7$ kN/m³

γ'_i idem per materiale in situ

$K_a = \tan^2(45 - \varphi/2)$ coeff. spinta attiva

$K_p = \tan^2(45 + \varphi/2)$ coeff. spinta passiva

NB. La pressione esercitata dalla terra contro il muro ad una data quota è pari alla pressione sul corrispondente piano orizzontale moltiplicata per il coefficiente di spinta locale del terreno

1.3. Caratteristiche fisiche. — Per caratterizzare le tre o le due fasi costituenti la roccia o la terra in sede o a rilevato si possono adottare le caratteristiche qui di seguito definite (fig. 1.2):

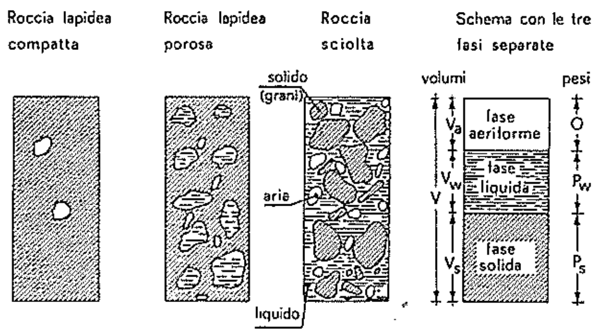


Fig. 1.2. — Elemento di volume di roccia lapidea o di roccia sciolta.

Peso specifico della sostanza solida	$\gamma_s = \dfrac{P_s}{V_s}$ [2.6 t/m³ ≅ 26 kN/m³]
Peso specifico dell'acqua	γ_w [~ 10 kN/m³] (.6 ÷ 16)
Peso secco dell'unità di volume	$\gamma_d = \dfrac{P_s}{V} = \gamma_s(1-n) = \gamma_s \dfrac{1}{1+e}$ [26 × .7 ≅ 18 kN/m³]
Peso dell'unità di volume	$\gamma = \dfrac{P_s + P_w}{V} = \gamma_d(1+w) = \gamma_s(1-n)(1+w)$ Sabbie
Porosità	$n = \dfrac{V_a + V_w}{V} = 1 - \dfrac{\gamma_d}{\gamma_s} = \dfrac{e}{1+e}$ [0.3] (0.4)
Indice dei pori	$e = \dfrac{V_a + V_w}{V_s} = \dfrac{\gamma_s}{\gamma_d} - 1 = \dfrac{n}{1-n}$
Contenuto d'acqua	$w = \dfrac{P_w}{P_s}$
Contenuto d'acqua del materiale saturo	$w_{max} = w_{sat} \dfrac{\gamma_w}{\gamma_s} e$
Peso dell'unità di volume del materiale saturo	$\gamma_{sat} = \gamma_s(1-n) + n \cdot \gamma_w$ [26 × 0.7 + 10 × 0.3] [= 21 kN/m³] (20)
Peso dell'unità di volume del materiale immerso in acqua	$\gamma' = \gamma_{sat} - \gamma_w$ [21 − 10 = 11 kN/m³] (10)
Grado di saturazione	$S_r = \dfrac{V_w}{V_a + V_w} = \dfrac{w}{w_{max}}$

Valori orientativi delle caratteristiche fisiche di alcuni terreni in sede sono riportati nelle tabb. 1.VI e 1.VII.

1.4. Identificazione delle terre.

1.4.1. Tessitura. — La fase solida di una terra è costituita da particelle di materia inorganica; a volte è presente anche materia organica (v. par. 1.4.2).

Tab. 1.VI. — Valori orientativi delle caratteristiche fisiche di rocce.

Rocce	γ_s (tm⁻³)	γ_d (tm⁻³)	n (%)
Graniti	2,6 ÷ 2,8	2,6 ÷ 2,8	1 ÷ 4
Dioriti	2,8 ÷ 3,0	2,8 ÷ 3,0	1 ÷ 2
Porfidi	2,6 ÷ 2,8	2,6 ÷ 2,8	1 ÷ 2
Arenarie	2,6 ÷ 2,7	2,6 ÷ 2,7	1 ÷ 2
Calcari compatti	2,7 ÷ 2,9	2,7 ÷ 2,9	1 ÷ 2
Travertini	2,6 ÷ 2,7	2,4 ÷ 2,5	5 ÷ 12
Calcareniti molto compatte	2,7	2,2	20
Trachiti	2,7	2,1	20
Tufi molto compatti	2,4	1,3 ÷ 1,4	40 ÷ 50
Tufi di media porosità	2,4	1,0 ÷ 1,2	50 ÷ 60
Calcareniti	2,7	1,3 ÷ 1,9	30 ÷ 50

Tab. 1.VII. — Valori orientativi delle caratteristiche fisiche delle terre in sede.

Terre	γ_s (tm⁻³)	γ (tm⁻³)	γ_d (tm⁻³)	n (%)	w
Argille estremamente dure	2,7	2,1 ÷ 2,3	1,9 ÷ 2,1	0,20 ÷ 0,30	0,1 ÷ 0,2
Argille dure e molto duro	2,7	1,8 ÷ 2,1	1,4 ÷ 1,8	0,30 ÷ 0,50	0,2 ÷ 0,4
Argille molli e molto molli	2,6 ÷ 2,7	1,4 ÷ 1,7	0,7 ÷ 1,1	0,55 ÷ 0,75	0,5 ÷ 1,0
Materiali torbosi	1,8 ÷ 2,2	0,9 ÷ 1,2	0,1 ÷ 0,4	0,80 ÷ 0,95	2,0 ÷ 6,0
Limi	2,7	1,6 ÷ 2,1	1,3 ÷ 1,9	0,30 ÷ 0,50	—
Sabbia fina uniforme	2,7	1,5 ÷ 2,1	1,4 ÷ 1,8	0,30 ÷ 0,50	—
Sabbia	2,6 ÷ 2,7	1,6 ÷ 2,1	1,3 ÷ 1,8	0,25 ÷ 0,45	—
Ghiaia sabbiosa	2,6 ÷ 2,7	1,8 ÷ 2,3	1,4 ÷ 2,1	0,20 ÷ 0,40	—
Pozzolane	2,3 ÷ 2,5	1,0 ÷ 1,7	0,8 ÷ 1,3	0,50 ÷ 0,60	0,1 ÷ 0,4
Pomici	2,0 ÷ 2,5	0,6 ÷ 1,2	0,5 ÷ 0,8	0,60 ÷ 0,80	0,2 ÷ 0,5
Pietrame rinfianco	2.6 t/m³ 26 kN/m³	2.1 sat 21	1,8 18	0.3	

Le caratteristiche geometriche (tessitura) delle particelle sono molto variabili e possono essere individuate con i seguenti parametri:

— granulometria;

— forma;

— grado di arrotondamento degli spigoli.

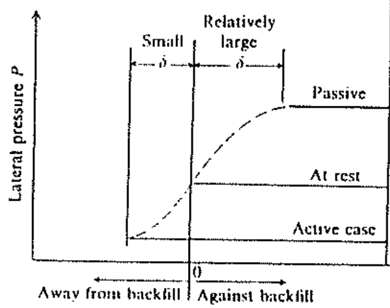
La *granulometria* viene definita in base alla curva granulometrica e cioè al diagramma di frequenze cumulate delle percentuali in peso della distribuzione del diametro equivalente d delle particelle (fig. 1.3): di norma la granulometria viene determinata sul passante a 60 mm.

Per le frazioni granulometriche si adotta la terminologia di fig. 1.3 (A.G.I., 1977).

Se in un terreno sono presenti le frazioni A, B e C in percentuali $p_A > p_B > p_C$ il terreno viene denominato (A.G.I. 1977) col nome della frazione A seguito dai nomi delle frazioni B o C preceduti dalla congiunzione «con» se la corrispondente percentuale p è compresa fra il 50 ed il 25 %, seguiti dal suffisso «oso» se p è compresa tra il 25 ed il 10 % ed infine seguiti dal suffisso anzidetto e preceduti da «debolmente» se p è compresa tra il 10 ed il 5 %.

Il *grado di uniformità* è dato dal rapporto fra il diametro che ha percentuale 60 ed il diametro che ha percentuale 10 (*diametro efficace*):

$$U = \frac{d_{60}}{d_{10}}$$



Usual range of earth pressure coefficients	
Cohesionless soils	Cohesive soils
3–14	1–2
0.4–0.6	0.4–0.8
0.33–0.22	1–0.5

FIGURE 11-3 Illustration of active and passive pressures with usual range of values for cohesionless and cohesive soil.

This discussion has been on a theoretical basis. We must now have a means to apply these principles in a general way to evaluate what the earth pressure will be for specific applications. There are currently two general procedures for soil masses and a theory of elasticity method for loads on the soil mass that is to be resisted by the wall. These methods will be considered in the following several sections.

11-4 COULOMB EARTH PRESSURE THEORY

One of the earliest methods for estimating earth pressures against walls is credited to C. A. Coulomb (ca 1776) which made a number of assumptions as follows:

1. Soil is isotropic, homogeneous, and has both internal friction and cohesion.
2. The rupture surface is a plane surface (as BC of Fig. 11-2b) and the backfill surface is planer (it may slope but is not irregularly shaped).
3. The friction resistance is distributed uniformly along the rupture surface and the soil-to-soil friction coefficient $f = \tan \phi$.
4. The failure wedge is a rigid body undergoing translation.
5. There is wall friction, i.e., as the failure wedge moves with respect to the backface of the wall a friction force is developed between soil and wall. This friction angle is usually termed δ.
6. Failure is a plane strain problem—consider a unit slice from an infinitely long wall.

The principal deficiencies in the Coulomb theory are in the assumption of an ideal soil and that the rupture zone is a plane (although for clean sand in the active pressure case photographs of model walls indicates the rupture zone is very nearly a plane as BC of Fig. 11-2b).

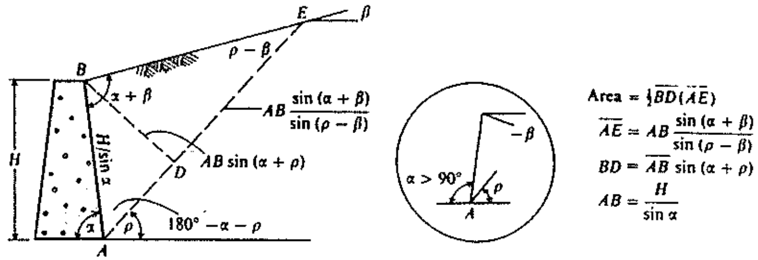


FIGURE 11-4 Failure wedge used in deriving the Coulomb equation for active pressure. Note β may be $\pm$ and $0 < \alpha < 180°$.

The equations based on the Coulomb theory for a cohesionless soil can be derived from Figs. 11-4 and 11-5 and using a substantial amount of trigonometric relationships. The weight of the soil wedge ABE of Fig. 11-4 is

$$W = \gamma A(1) = \frac{\gamma H^2}{2 \sin^2 \alpha}\left[\sin(\alpha + \rho)\frac{\sin(\alpha + \beta)}{\sin(\rho - \beta)}\right] \qquad (a)$$

The active force P_a is a component of the weight vector as illustrated in Fig. 11-5c. Applying the law of sines, we obtain

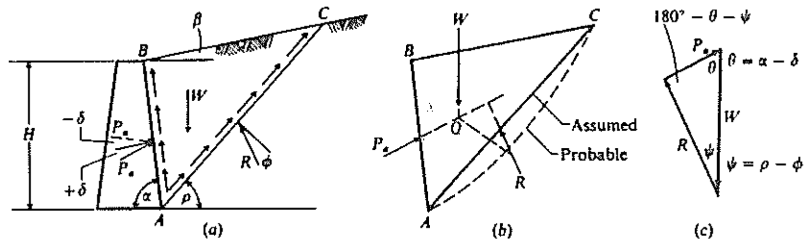
$$\frac{P_a}{\sin(\rho - \phi)} = \frac{W}{\sin(180 - \alpha - \rho + \phi + \delta)}$$

or

$$P_a = \frac{W \sin(\rho - \phi)}{\sin(180 - \alpha - \rho + \phi + \delta)} \qquad (b)$$

From Eq. (b) it can be seen that the value of $P_a = f(\rho)$; that is, all other terms for a given problem are constant, and the value of P_a of primary interest is the largest

FIGURE 11-5 (a) Assumed conditions for failure; (b) indicates all force vectors may not pass through point O; hence static equilibrium is not satisfied; (c) force triangle to establish P_a.



possible value. Combining Eqs. (*a*) and (*b*), we obtain

$$P_a = \frac{\gamma H^2}{2\sin^2\alpha}\left[\sin(\alpha+\rho)\frac{\sin(\alpha+\beta)}{\sin(\rho-\beta)}\right]\frac{\sin(\rho-\phi)}{\sin(180-\alpha-\rho+\phi+\delta)} \tag{c}$$

the maximum active wall force P_a is found from setting $dP_a/d\rho = 0$ to give

$$P_a = \frac{\gamma H^2}{2}\frac{\sin^2(\alpha+\phi)}{\sin^2\alpha\sin(\alpha-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2} \tag{11-1}$$

If $\beta = \delta = 0$ and $\alpha = 90°$ (a smooth vertical wall with horizontal backfill), Eq. (11-1) simplies to

$$P_a = \frac{\gamma H^2}{2}\frac{(1-\sin\phi)}{(1+\sin\phi)} = \frac{\gamma H^2}{2}\tan^2\left(45-\frac{\phi}{2}\right) \tag{11-2}$$

which is also the Rankine equation for active earth pressure considered in the next section. Equation (11-2) takes the general form

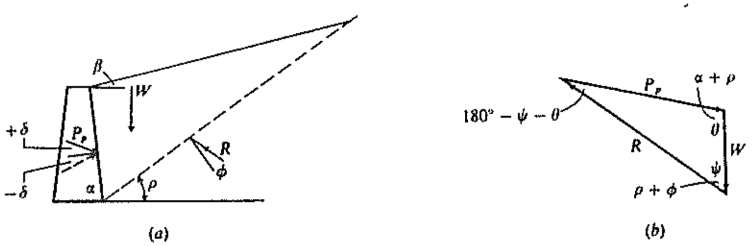
$$P_a = \frac{\gamma H^2}{2}K_a$$

where
$$K_a = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha\sin(\alpha-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2} \tag{11-3}$$

and is a coefficient which considers α, β, δ, and ϕ, but is independent of γ and H. Table 11-1 gives values of K_a for selected angular values, and a computer program can easily be written to solve for values of K_a for other angle combinations.

Passive earth pressure is derived similarly except that the inclination at the wall and the force triangle will be shown as in Fig. 11-6.

FIGURE 11-6 (*a*) Failure wedge and acting forces for passive pressure; (*b*) force polygon for computation of passive pressure.



From Fig. 11-6 the weight of the assumed failure mass is

$$W = \frac{\gamma H^2}{2}\sin(\alpha+\rho)\frac{\sin(\alpha+\beta)}{\sin(\rho-\beta)} \tag{d}$$

and from the force triangle, using the law of sines

$$P_p = W\frac{\sin(\rho+\phi)}{\sin(180-\rho-\phi-\delta-\alpha)} \tag{e}$$

Setting the derivative $dP_p/d\rho = 0$ gives the minimum value of P_p as

$$P_p = \frac{\gamma H^2}{2}\frac{\sin^2(\alpha-\phi)}{\sin^2\alpha\sin(\alpha+\delta)\left[1-\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi+\beta)}{\sin(\alpha+\delta)\sin(\alpha+\beta)}}\right]^2} \tag{11-4}$$

For a smooth vertical wall with horizontal backfill ($\delta = \beta = 0$ and $\alpha = 90°$), Eq. (11-4) simplifies to

$$P_p = \frac{\gamma H^2}{2}\frac{1+\sin\phi}{1-\sin\phi} = \frac{\gamma H^2}{2}\tan^2\left(45+\frac{\phi}{2}\right) \tag{11-5}$$

Equation (11-4) can also be written

$$P_p = \frac{\gamma H^2}{2}K_p$$

where
$$K_p = \frac{\sin^2(\alpha-\phi)}{\sin^2\alpha\sin(\alpha+\delta)\left[1-\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi+\beta)}{\sin(\alpha+\delta)\sin(\alpha+\beta)}}\right]^2} \tag{11-6}$$

Table 11-2 gives values for K_p for selected angular values of ϕ, α, δ, and β.

Figure 11-1 displays that earth pressure is dependent on the *effective* stresses in the soil and not total stresses. It necessarily follows that the wall pressure below the water table is the sum of the hydrostatic pressure and the effective lateral earth pressure from using the effective unit weight γ' of the soil.

Example 11-1. What is the total active force per meter of wall for the soil-wall system shown in Fig. E11-1, using the Coulomb equations? Where does P_a act?

Solution. Take wall friction $\delta = 2\phi/3 = 20°$ (a common estimate). For $\phi = 30°$ obtain $K_a = 0.34$ from Table 11-1.

$$p_a = \gamma z K_a$$

$$P_a = \int_0^H \gamma z K_a\,(dz) = \frac{1}{2}\gamma H^2 K_a$$

$$P_a = \tfrac{1}{2}(17.52)(5)^2(0.34) = 74.5\ \text{kN/m}$$

TABLE 11-1

Coulomb active earth pressure coefficients K_a using Eq. (11-3)

ALPHA = 90   BETA = -10

δ	$\phi = 26$	28	30	32	34	36	38	40	42
0	0.354	0.328	0.304	0.281	0.259	0.239	0.220	0.201	0.184
16	0.311	0.290	0.270	0.252	0.234	0.216	0.200	0.184	0.170
17	0.309	0.289	0.269	0.251	0.233	0.216	0.200	0.184	0.169
20	0.306	0.286	0.267	0.249	0.231	0.214	0.198	0.183	0.169
22	0.304	0.285	0.266	0.248	0.230	0.214	0.198	0.183	0.168

ALPHA = 90   BETA = -5

δ	$\phi = 26$	28	30	32	34	36	38	40	42
0	0.371	0.343	0.318	0.293	0.270	0.249	0.228	0.209	0.191
16	0.328	0.306	0.284	0.264	0.245	0.226	0.209	0.192	0.176
17	0.327	0.305	0.283	0.263	0.244	0.226	0.208	0.192	0.176
20	0.324	0.302	0.281	0.261	0.242	0.224	0.207	0.191	0.175
22	0.322	0.301	0.280	0.260	0.242	0.224	0.207	0.191	0.175

ALPHA = 90   BETA = 0

δ	$\phi = 26$	28	30	32	34	36	38	40	42
0	0.390	0.361	0.333	0.307	0.283	0.260	0.238	0.217	0.198
16	0.349	0.324	0.300	0.278	0.257	0.237	0.218	0.201	0.184
17	0.348	0.323	0.299	0.277	0.256	0.237	0.218	0.200	0.183
20	0.345	0.320	0.297	0.276	0.255	0.235	0.217	0.199	0.183
22	0.343	0.319	0.296	0.275	0.254	0.235	0.217	0.199	0.183

ALPHA = 90   BETA = 5

δ	$\phi = 26$	28	30	32	34	36	38	40	42
0	0.414	0.382	0.352	0.323	0.297	0.272	0.249	0.227	0.206
16	0.373	0.345	0.319	0.295	0.272	0.250	0.229	0.210	0.192
17	0.372	0.344	0.318	0.294	0.271	0.249	0.229	0.210	0.192
20	0.370	0.342	0.316	0.292	0.270	0.248	0.228	0.209	0.191
22	0.369	0.341	0.316	0.292	0.269	0.248	0.228	0.209	0.191

ALPHA = 90   BETA = 10

δ	$\phi = 26$	28	30	32	34	36	38	40	42
0	0.443	0.407	0.374	0.343	0.314	0.286	0.261	0.238	0.216
16	0.404	0.372	0.342	0.315	0.289	0.265	0.242	0.221	0.201
17	0.404	0.371	0.342	0.314	0.288	0.264	0.242	0.221	0.201
20	0.402	0.370	0.340	0.313	0.287	0.263	0.241	0.220	0.201
22	0.401	0.369	0.340	0.312	0.287	0.263	0.241	0.220	0.201

TABLE 11-2

Coulomb passive earth pressure coefficients K_p using Eq. (11-6)

ALPHA = 90   BETA = -10

δ	$\phi = 26$	28	30	32	34	36	38	40	42
0	1.914	2.053	2.204	2.369	2.547	2.743	2.957	3.193	3.452
16	2.693	2.956	3.247	3.571	3.934	4.344	4.807	5.335	5.940
17	2.760	3.034	3.339	3.679	4.062	4.493	4.983	5.543	6.187
20	2.980	3.294	3.645	4.041	4.488	4.997	5.581	6.255	7.039
22	3.145	3.490	3.878	4.317	4.816	5.389	6.050	6.819	7.720

ALPHA = 90   BETA = -5

δ	$\phi = 26$	28	30	32	34	36	38	40	42
0	2.223	2.392	2.577	2.781	3.004	3.250	3.523	3.826	4.163
16	3.367	3.709	4.094	4.529	5.024	5.591	6.243	7.000	7.883
17	3.469	3.828	4.234	4.694	5.218	5.820	6.516	7.326	8.277
20	3.806	4.226	4.704	5.250	5.879	6.609	7.462	8.468	9.665
22	4.064	4.532	5.067	5.684	6.399	7.236	8.222	9.397	10.809

ALPHA = 90   BETA = 0

δ	$\phi = 26$	28	30	32	34	36	38	40	42
0	2.561	2.770	3.000	3.255	3.537	3.852	4.204	4.599	5.045
16	4.195	4.652	5.174	5.775	6.469	7.279	8.229	9.356	10.704
17	4.346	4.830	5.385	6.025	6.767	7.636	8.661	9.882	11.351
20	4.857	5.436	6.105	6.886	7.804	8.892	10.194	11.771	13.705
22	5.253	5.910	6.675	7.574	8.641	9.919	11.466	13.364	15.726

ALPHA = 90   BETA = 5

δ	$\phi = 26$	28	30	32	34	36	38	40	42
0	2.943	3.203	3.492	3.815	4.177	4.585	5.046	5.572	6.173
16	5.250	5.878	6.609	7.464	8.474	9.678	11.128	12.894	15.076
17	5.475	6.146	6.929	7.850	8.942	10.251	11.836	13.781	16.201
20	6.249	7.074	8.049	9.212	10.613	12.321	14.433	17.083	20.468
22	6.864	7.820	8.960	10.334	12.011	14.083	16.685	20.011	24.352

ALPHA = 90   BETA = 10

δ	$\phi = 26$	28	30	32	34	36	38	40	42
0	3.385	3.712	4.080	4.496	4.968	5.507	6.125	6.840	7.673
16	6.652	7.545	8.605	9.876	11.417	13.309	15.665	18.647	22.497
17	6.992	7.956	9.105	10.492	12.183	14.274	16.899	20.254	24.633
20	8.186	9.414	10.903	12.733	15.014	17.903	21.636	26.569	33.270
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

- other variable actions, combination and action factor $\gamma_{Q,i}\psi_{0,i}$; $1.5 \times \psi_{0,i}$
- combination factor $\psi_{0,i}$ should be taken in the range 0.8–1.0.

6.4 Material factors

The design strength of a material is determined by dividing the structural material strength by a material factor, which shall account for uncertainties in material strength, execution and calculations. The material factor must also take into account the consequences of damage. The consequences of damage can be divided into three classes:

(a) Less serious: failure that involves little risk of injury to people or small economic or other consequences.
(b) Serious: failure that involves risk of injury to people or significant economic or other consequences.
(c) Very serious: failure that involves large risk of injury to people or very large economic or other consequences.

The values of the material factors are designed to provide a level of safety appropriate to the purpose of maritime structures. In the maritime environment, considerations of damage to material objects rather than to human life generally predominate. It is therefore necessary for a rational design to weigh the ascertainable cost of providing additional strength against the probable costs of repair, consequential damage and economic loss during the life of the structure.

In, for example, the Norwegian Standard, the material factors for concrete and reinforcement are 1.4 and 1.25 respectively for concrete works executed under an extended or ordinary inspection work standard. The standards allow the use of reduced material factors in cases where the tolerances are strictly controlled and where the maximum deviations in the most unfavourable direction are considered in the design. For berth and harbour structures where the durability is a major concern, the use of such reduced values is however not encouraged. On the basis of experience with maintenance and deterioration of structures in marine environments it is recommended that a material factor at least equal to the following, if the design life of the structure is more than about 20 years, should be used:

(a) steel piles filled with reinforced concrete: 1.25
(b) reinforced concreted piles, etc. concreted under water: 1.60
(c) all other harbour structures: 1.40.

If the design life of the structure is less than 20 years a material factor of 1.25 can be used.

In the new European suite of design standards, the *Eurocode*, the material factors are defined in a slightly different manner. The *Eurocode* is a common set of standards, although matters related to both safety and durability are within the competence of the member states. Parameters that affect safety and durability are therefore open for the member states to determine as NDPs. The *Eurocode* gives recommended values, but the various European countries are free to give other values that have to be used on their territory. These NDPs shall be given in a NA. The recommended values for reinforced concrete structures are for the concrete $\gamma_c = 1.5$ and for the reinforcement $\gamma_s = 1.15$. Reduced values may be permitted under certain conditions.

The design strength of the concrete is determined as $f_{cd} = \alpha_{cc} f_{ck}/\gamma_c$ where α_{cc} is a factor taking account of long-term effects on the compressive strength of the concrete resulting from the duration of the load and the way it is applied. The *Eurocode* specifies that α_{cc} shall be taken between 0.8 and 1.0 with 1.0 as the recommended value. For berth structures it may well be that a value of 0.9 should be considered for α_{cc}.

6.5 Characteristic loads on berth structures

6.5.1 Temperature and shrinkage forces

In the design of, for example, berth decks of reinforced concrete, allowances must be made for temperature and shrinkage forces in the transverse as well as the longitudinal directions of the deck.

6.5.2 Live loads and wheel loads

It is difficult to lay down guidelines for live loads on aprons as a function of the ship's size. The loads on the apron deck are determined by the type of traffic utilizing the berth, and not so much by the size of the ships. Special berths like oil piers accommodate ships of several hundred thousands of tons displacement but have live loadings of say 10 kN/m^2. On the other hand, berths accommodating supply ships for the offshore oil industry of only, say, two thousand tons displacement must be designed for a live load of between 50–200 kN/m^2. Berths for heavy industry should be designed for a live load of between 40–100 kN/m^2. In fishing harbours the berth structures should be designed for a live load of at least 15 kN/m^2. The loads are therefore essentially dependent on the type of cargo, on the

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Table 6.2. Recommended live loads

Type of traffic and cargo	Loading in kN/m^2
Light traffic or small cars	5
Heavy traffic or trucks	10
General cargo	20
Palletized general cargo	20–30
Multi-purpose facility	50
Offshore feeder bases	50–200
Heavy vehicles, heavy crane, crawler crane, etc. that operate from the berth front and 3 m inboard	60
Heavy vehicles, heavy crane, crawler crane, etc. that operate from 3 m behind the berth front and further inwards	40–100
Containers	
Empty and stacked 4 high	15
Full and stacked 2 high	35
Full and stacked 4 high	55
General ro/ro loads	30–50

handling equipment, local practices, etc., so that uniformity can only be achieved to a limited extent.

As a general guideline Table 6.2 shows recommended live loads for the apron and the terminal area.

In the case of a very exposed open berth structure, the possibility of uplifting of the deck structure due to waves passing under should be considered.

It is strongly recommended that the same live load for the whole terminal as the live load used at the apron be used, in order to achieve maximum flexibility in cargo-handling techniques. The berth structure should also be designed to carry the maximum live loading that might be imposed during the life of the structure due to handling, transport and storage of the cargo or other activities.

Most public berths (multi-purpose berths), accommodating ocean-going dry-cargo ships, should be designed for container loads. Twenty-foot containers stacked two high imply a load of 25–35 kN/m^2 depending on the cargo they are loaded with. The sizes of a 20-ft and 40-ft container is respectively $6.06 \times 2.44 \times 2.44$ m and $12.12 \times 2.44 \times 2.44$ m. The empty weight of a 20-ft container range between 19–22 kN and the maximum total weight permitted by ISO (the international container standard) is 240 kN. For a 40-ft container the empty weight ranges between 28–36 kN and with a maximum total weight of 305 kN.

Aprons and ramps for container traffic should be designed for a useful load of at least 40 kN/m^2.

Wheel loads from trailers, fork-lift trucks, mobile cranes, container cranes and other cranes on rails, railways, etc. should be evaluated in each case, because there is, in the market nowadays, a spectrum of types and makes of such equipment with individual loading specifications. Fork-lift trucks for handling 40-ft containers can have axle loads of up to 1200 kN. In order to highlight the relatively big damaging effect of fork-lifts on pavements, it is significant to note that an axial load up to 1200 kN on a fork-lift will give a wheel load slightly higher than the maximum wheel loads transmitted to the pavement during take-off by a Boeing 747 B.

Where mobile cranes may operate in the area behind the berth line, then provision should be made for the outrigger reactions and bearing pressures which may be imposed by the maximum size of a crane anticipated. The outrigger reactions are largely dependent on the crane lifting capacity and the radius of the jib length. If no information on the mobile crane can be obtained, the berth structure or the apron should be designed for a concentrated point load of at least 700 kN on an area of $1.0\,m \times 1.0\,m$ in the least favourable position. It should be mentioned that wheel loads for railways would be increased by 10 per cent and for fork-lift trucks and cranes by 20 per cent due to dynamic impacts. Both the berth apron and the whole container yard have to be designed in a homogeneous way and must be able to carry the heaviest combination of wheel or static loads for all handling equipment that may be present in the areas, i.e. container cranes, trailers, fork-lifts, straddle carriers, etc. In Chapter 13 about container terminal equipment, the different types of container handling equipments are in shown in principle.

To reduce the effect of a concentrated point load acting directly on a concrete deck slab, or to increase the loading area on which a concentrated point load is acting, one can, as shown in Fig. 6.3, put a layer of sand and asphalt on top of the concrete slab.

Berth structures which have direct road connection to the public highway network, should at least be designed for loads in accordance with the Highway Department's regulations. The loads should be at least a concentrated load of 150 kN on an area of $0.2\,m \times 0.2\,m$ in the most unfavourable position, or a live load of 20 kN/m^2.

The horizontal load transmitted to the apron, due to braking or wind forces, from cranes is about $\frac{1}{7}$ of the wheel load on the braked wheels in the direction of the rails. The horizontal load in the direction

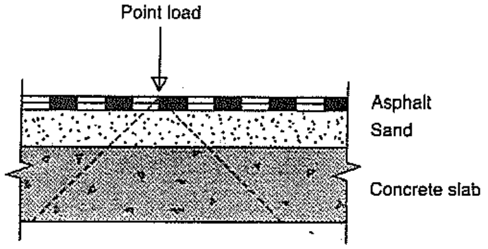

Fig. 6.3. To increase the loading area

perpendicular to the rails is about $\frac{1}{10}$ of the wheel load. For rubber-tyre mounted equipment a factor of $\frac{1}{10}$ is also applied.

Storing of frictional material will imply a horizontal load component being transmitted to the apron deck in addition to the vertical load. This component, acting as a tensile force in the top of the deck, is equal to the maximum static friction in the stored material.

In order to prevent vehicles from rolling over the berth line into the water, a front curb should be installed along the berth front. This curb should be designed for a horizontal point load of 15–25 kN depending on the type of traffic and should be about 0.20 m high.

Useful loads in transit sheds and warehouses depend to a great extent on the height to which palletized cargo can be stacked with fork-lift trucks. Design loads vary between 20 and 50 kN/m^2 (or more) over the whole floor area, depending on types of cargo.

To prevent overloading of the berth structure the allowable load should be marked in clear letters and figures on a signboard at the apron.

6.5.3 Seismic loads

Seismic or earthquake loads on the berth structure should be considered if the structures are in an area of seismographic disturbance. The seismic loads will act at the centre of gravity of the structure as a horizontal force equal to the design coefficient times the weight of the structure. The weight to be used for the berth structure itself should be the total dead load plus one-half of the live load. The design seismic coefficient is equal to the regional seismic coefficient times the factor for the subsoil condition times the coefficient of importance. The design seismic coefficient will usually be between 0.05–0.25. For cargo-handling equipment, the seismic load is the product of a horizontal seismic coefficient and the deadweight of the cargo-handling equipment.

The actual seismic load due to an earthquake will depend on the magnitude of the earthquake, the type of structure, type of equipment and the soil conditions in the area. Generally, unless the berth structure is of a massive or gravity type, the seismic effect on the design will usually be small. This applies to both the transverse and the longitudinal direction of the berth structure.

The seismic performance requirements for a particular berth structure should be established in accordance with international standards and guidelines based on acceptable risk procedures. The requirements for a berth structure should be based on the importance of the berth structure, the acceptable levels of risk to life safety, the port operations, etc.

6.6 Characteristic loads from the landside

The weight of the fill behind the berth structure and the useful load on top of the fill will serve as a stabilizing load, for example on berth anchoring friction plates. The weight of the fill may also cause horizontal loading to the berth structure, for instance in connection with water pressure due to a blockage of the drainage system behind the berth structure. The magnitude of such forces must be evaluated in each separate case.

A further discussion of forces acting from the landside is considered to be outside the scope of this book, but EAU 1996 and ROM 0.2-90 give useful recommendations.

6.7 Summary of loads acting from the seaside

From the above discussions not only are static and dynamic conditions involved when design loads on the berth structure being established, the human factor during, for example, manoeuvring the ship to the berth also comes in. Therefore this suggests the assumption of more conservative load values than those which are strictly necessary according to detailed calculations.

As an example, if the contractors carrying out a tender are allowed to give alternative designs for example, berthing structure for gas tankers, the following minimum design loads should be given.

Ship size maximum	137 000 m^3
Ship size maximum fully loaded displacement	100 000 t
Ship size minimum	20 000 m^3

YOLTRI
20 (1)
20 (2)
35 (3)

(1) pavimentaz.

(2) stabilizza

600 kN/m

(3) stabilizzaz.

URTO

Approach ship velocity normal to jetty front	0.15 m/sec
Approach ship velocity parallel to jetty front	0.02 m/sec
Ship angle of approach to jetty front fully loaded	5°
Hull pressure between the fenders and the ship, max.	0.20 MPa
Friction coefficient between ship hull and fender, both horizontal and vertical	0.2
QRH with capstan, minimum capacity of each hook	1000 kN
Loading platform and access road:	
Point load at any point at loading platform and access road on area 1 m × 1 m	700 kN
Vertical live load, general	20 kN/m^2
Pipeline:	
Vertical live load	2.5 kN/m^2
Walkways:	
Vertical live load	3.0 kN/m^2
Horizontal load top handrail	0.8 kN/m
Mooring dolphins:	
Vertical live load	3.0 kN/m^2

Earthquake: There is little risk of seismic activity. For design purpose the values of z according to Uniform Building Code (USUBC) are assumed to be as for zone 2B, where $z = 0.2$.

References and further reading

British Standard BS 6349, Part 1 (2000) *Code of Practice for Maritime Structures*, London: BSI.

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Ministerio de Obras Publicas y Transpotes (1990) *ROM Recomendaciones para Obras Maritimas* (*Maritime Works Recommendations, Actions in the Design of Maritime and Harbour Works ROM 0.2-90. English version*). Madrid.

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7

Safety consideration

7.1 General

The safety measures, which have to be considered in a harbour project, will be safety related to the specification, the design, the construction, the personnel and the operation. The safety related activities are important activities in the work of the Consulting Engineer and all these aspects should therefore be given the highest priority during all his consulting work.

7.2 Specification safety

For a harbour project, the primary activity of securing satisfactory implementation of safety should be considered in the specification or the start-up phase. In this phase all the engineering standards, design codes, governmental laws and regulations have to be defined and listed as the project engineering specifications.

The safety routines for all the work to be performed by the Consulting Engineer should be implemented through quality assurance and the control system for the project, and through the project coordination and engineering procedures. The project quality assurance manual, project coordination and engineering procedures should give detailed regulations for review and approvals, both internal for the project team, and in relation to Client and external interfaces.

7.3 Design safety

The design aspect for the berth structures should be based on common and proven design methods and technology. In order to sort out the

3.2.2.4 Bollard Loads

A ship coming alongside is usually stopped partly by reversing the engine and partly be retarding by the spring hawser, so that the total design force transmitted to the berth structure through the bollard will at least be equal to the breaking load of the spring hawser. Materials for hawsers are steel wire, manila rope, nylon rope, etc., i.e. different materials implying great variations in the breaking loads and ductility of the various hawsers. Figure 3.2.2.4.A shows a normal mooring arrangement for a ship to a quay via bollards.

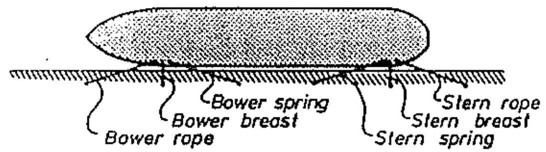


Fig. 3.2.2.4.A Mooring by hawsers and bollards

It has therefore been necessary, as has been done in most port engineering standards, to specify minimum loadings that the bollards shall be able to resist for ships of various tonnages. Thus the bollards, their dimensions and anchoring, and the berth structure itself shall be designed for a certain minimum loading. The idea is that if a ship has a too strong hawser compared to the design load of the bollard, only the latter will break at its footing without the berth structure itself being much affected.

Bollard should be provided at intervals of approximately 5-30 meters depending of the size of the ship along the berthing face, and the bollard load capacity should be as shown in the table below.

The bollard load P and approximate spacing between bollards shall be:

Ships of displacements in tons up to	Bollard load P, kN	Appr. spacing in m	Bollard load from the berth kN pr. lin m berth	Bollard load along the berth, kN per lin. m berth
2,000	100	5-10	15	10
5,000	200	10-15	15	10
10,000	300	15	20	15
20,000	500	20	25	20
30,000	600	20	30	20
50,000	800	20-25	35	20
100,000	1000	25	40	25
200.000	1500	30	50	30

For larger ships, specific calculation must be carried out to determine the maximum bollard load, taking into account the type of ship and the environmental loading. Bollard loads are assumed to act in any direction within 180° around the bollard at the sea side, and from horizontally to 60° upwards, as shown in Figure 3.2.2.4.B.

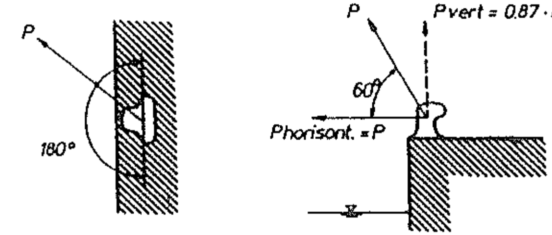


Fig. 3.2.2.4.B Bollard load directions

If the berth structure is much exposed to wind and currents, the above bollard loads should be increased by 25%. When the ship is moored, the bollard will be loaded with a vertical force of 0.87 · P, as shown in Figure 3.2.2.4.B.

Mooring dolphins should be designed for the same loads as the bollards. In addition to the usual quay bollards, storm bollards are often installed behind the apron, designed for twice the above bollard loadings.

If one and the same bollard accommodates more than one hawser, the German standard recommends that the bollard should still be designed for the tabulated load only. This because it is most unlikely that all the hawsers are fully loaded and pulling in the same direction at the same time.

Figure 3.2.2.4.C shows the six main types of movements of a ship in unsettled waters. The mooring forces most difficult to predict, are those caused by waves acting along the berthlines. Such forces are probably also the most common reason for broken moorings.

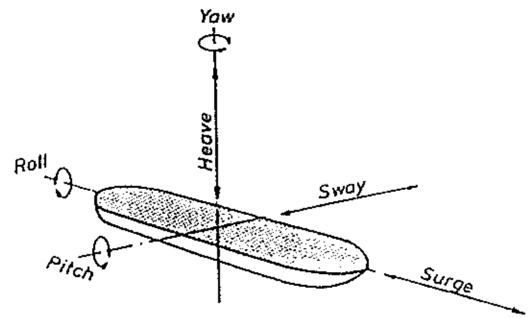


Fig. 3.2.2.4.C Types of ship's movements

GRU DI BANCHINA CONTAINER:
(PORTAINER)

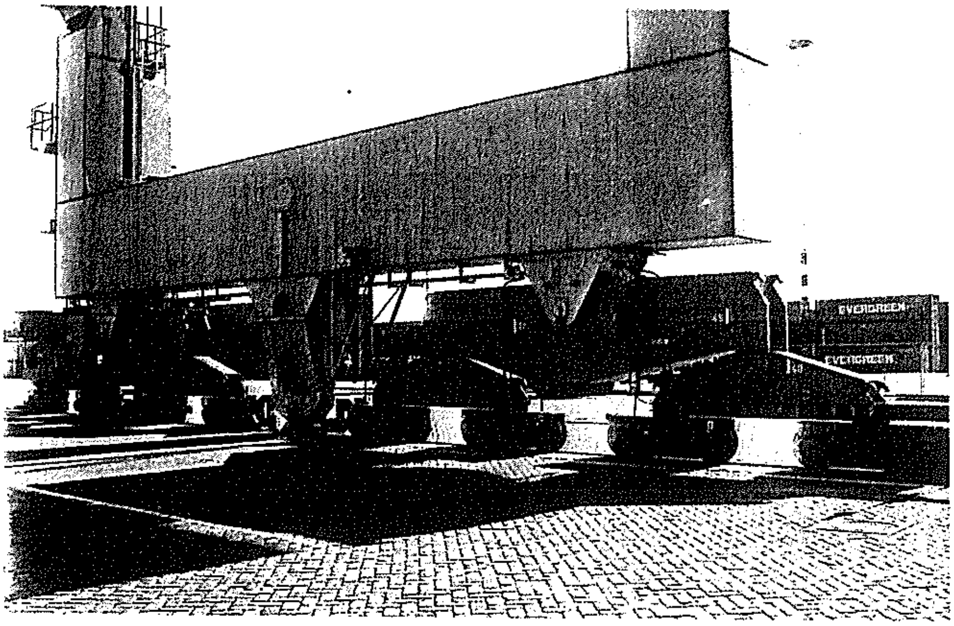


Fig. 5.12 Bogie of a portainer crane.

Table 5.3 Portainer cranes with eight wheels on each leg.

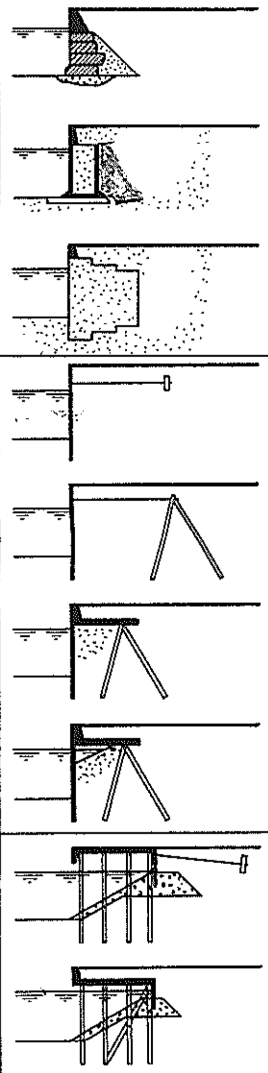
Rail gauge	Lifting capacity on the water side with an out reach of	Lifting capacity on the land side with an backreach of	Self weight kN	Max. wheel load water side kN	Max. wheel load land side kN	Distance between wheels m
15.24 m	410 kN–36 m	410 kN–13 m	5150	293	274	1.75
15.24 m	500 kN–38 m	500 kN–12 m	8100	474	433	1.20
20.00 m	500 kN–43 m	500 kN–16 m	9770	568	542	1.00
100ft → 30.48 m	500 kN–40 m	500 kN–18 m	8970	408	609	1.24
35.00 m	670 kN–52 m	670 kN–25 m	12122	691	691	1.05
48.00 m	450 kN–30 m	450 kN–20 m	7350	420	383	1.50

2 gru di banchina: 400/1.60 = 250 kN/m
+20% f. dinam.

2m 1.60

~400 kN ad es:

MURI DI BANCHINA (QUAY WALL)



	Type of structure	Static Principle	Construction materials	Remarks
VERTICAL FACE	BLOCK WALL	Gravity structures	Blocks: Mass concrete Bedding: Stones, quarry run Backfill: Stones	Reasonably good subsoil needed. Preloading of structure normally required. Some types susceptible to settlements
	CAISSON		Caissons: Reinforced concrete Bedding: Stones, quarry run Capping: Reinforced or mass concrete	Reasonably good subsoil needed. Calm sea while placing required. Somewhat sensitive to settlements
	CELLULAR BULKHEAD		Cell: Flat web steel piles Fill: Sand Capping: Reinforced or mass concrete	Sensitive to horizontal loads until cell has been filled. Considerable settlements acceptable until *in situ* construction of capping wall
	SHEET PILE WALL	Flexible retaining structures. The wall distributes earth pressure by bending of sheet piles. The load is absorbed by tie rods anchored to slabs or batter piles and by passive earth pressure in front of the toe of the wall	Wall: Steel, reinforced concrete Tie rods: Steel Anchor slabs: Reinforced concrete or steel ~~Fill: Sand~~	Structure itself absorbs settlements, which may however not be acceptable for quay apron. Often the least cost solution
	SHEET PILE WALL			
	SHEET PILE WALL WITH RELIEVING PLATFORM (*) (Banchine danesi)		Wall: Steel, reinforced concrete Platform: Reinforced concrete Piles: Reinforced concrete, prestressed concrete, steel	
	SHEET PILE WALL WITH RELIEVING PLATFORM			
OPEN	OPEN PILED STRUCTURE con tirante	Vertical loads are carried by piles, while horizontal loads are absorbed by anchors or by batter piles	Deck: Reinforced concrete, cast *in situ* or precast Piles: Reinforced concrete, steel Slope protection: Stones	
	OPEN PILED STRUCTURE WITH RELIEVING PLATFORM che consente pali inclin. senza tirante			

Fig. 8.4 (a) Types of Structures

(*) RELIEVING PLATFORM : piattaforma collaborante che sopporta il carico del fronte banchina e del terreno