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Luxembourg

Practical Design of Sheet Pile Bulkheads

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Foreword

The non-homogeneity of soils and the uncertainty of the factors governing the equilibrium of forces in a mass of earth, present problems of earth pressure and resistance that cannot be solved with mathematical precision.

It must never be overlooked that, in practice, one cannot expect to obtain from computations in soil mechanics, results of more than approximate accuracy in relation to reality. For that reason, it is necessary to proceed very frequently by trial and error and to adopt several cases of possible stresses in order to value the effect of variation in results in terms of entertained assumptions.

To deal with a bulkhead problem it is therefore necessary to have straightforward methods of design that are rapid and accurate enough to reveal the possible solutions of the problem in view within a relatively short time. These methods should avoid lengthy computations and should render results that can be easily verified.

This handbook endeavours to summarize some of the rational graphical methods, based on simple hypotheses generally accepted today.

It is addressed primarily to those who require an introduction to the design of sheet pile bulkheads; it tries to present simply certain intricate phenomena that take place in a mass of earth and of which the thorough study is impossible without a fundamental knowledge of soil mechanics.

This book then is not a scientific treatise, it aims merely at outlining some of the problems in order to facilitate a closer study of sufficient accuracy for practical application.

The first chapter recalls certain elementary notions concerning the properties of soils of which knowledge is essential in a bulkhead design.

We have then shown how one can picture and assess numeric-

ally the pressures exercised by the soil on a plane wall in accordance with the theory of the linear distribution of pressures.

Some remarks on the pressure distribution on the buried part of the bulkhead lead to the determination of the different depths of penetration to be adopted for an earth retaining sheet pile wall.

The second chapter reviews numerical examples of the application of sheet pile bulkheads, cantilevered or anchored, the latter being freely supported or fixed in the ground.

Several pages are devoted to the checking of retaining walls subjected to exceptional stresses. Such checking alone can give a true idea of the reliability of the structure in a given case.

The third chapter deals with the simplification of the existing methods of anchorage design. For the standard anchor wall, these have always led to laborious exercises in trial and error. The two improved methods presented here are capable of easing appreciably the evaluation of the dimensions of the anchorage system with a predetermined factor of safety.

The accessories of the anchorage system, which possess only a subordinate interest in a sheet pile bulkhead design, are treated only summarily. The engineer in charge will have no difficulty in selecting dimensions of these parts by following the rules usually applicable in structural steel work.

In the Appendix, reference is given to the assumptions made and to the developments in calculations which have led to the establishment of various diagrams.

At the end, the reader will find an index to the notations used in the text as well as a list of publications by various authors cited in this handbook.

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CONTENTS

CHAPTER ONE

INTRODUCTION

1. PROPERTIES OF SOIL

The fundamental properties of soil serving as the basis for a bulkhead design are:

a) Its unit weight γ ;
b) Its angle of internal friction ρ ;
c) Its cohesion *c*.

1.a *UNIT WEIGHT OF SOIL*

Soils are formed from elements derived from the decomposition of rocks having in their natural compact state an average specific gravity Γ of about 2,65 t/m^3. These elements are separated by voids which occupy a relative volume v.

The dry weight of a unit volume of soil is consequently a function of the volume $(1-v)$ of its solid elements and of their specific gravity Γ. This can be shown as

$$\boxed{\gamma_s = \Gamma(1-v)}$$

A soil having a void ratio of 35 % (= 0,35) has a dry weight, if $\Gamma = 2{,}65$ t/m^3

$$\gamma_s = 2{,}65\,(1-0{,}35) = 1{,}72 \text{ t/m}^3.$$

In their natural state all soils occlude a certain quantity of water.

This natural moisture content w, expressed as a percentage of the dry unit weight, increases the actual unit weight of the soil to

$$\boxed{\gamma_n = \gamma_s (1 + w)}$$

For sands, w varies between 2 and 8 % and for silt and clay between 10 and 40 % of γ_s. Thus, a soil having a natural moisture content of $w = 4$ % (0,04) has a natural unit weight of

$$\gamma_n = 1{,}72 (1 + 0{,}04) = 1{,}79 \text{ t/m}^3.$$

At the foot of a horizontal layer of $h = 5$ m this soil exercises a vertical stress equal to

$$p_h = \gamma_n \cdot h = 1{,}79 \cdot 5 = 8{,}95 \text{ t/m}^2.$$

If the same soil is immerged, the water fills all the voids. From Archimedes' principle, the specific gravity of the solid particles is diminished in terms of their volume and of the unit weight γ_0 of the water, practically equal to 1,0 t/m³.

The submerged solid phase of a soil weighs therefore only:

$$\boxed{\gamma_i = \gamma_s - \gamma_0 (1 - v)}$$

or $\gamma_i = 1{,}72 - 1{,}0 (1 - 0{,}35) = 1{,}07$ t/m³ and the vertical stress exercised on a horizontal surface of a layer of soil of thickness $h = 5$ m is:

$$p_h = \gamma_i \cdot h$$

or
$$p_h = 1{,}07 \cdot 5 = 5{,}35 \text{ t/m}^2.$$

To the stress of the solid phase is obviously added the hydrostatic pressure of the liquid phase, equal to

$$p_h = \gamma_0 \cdot h = 1{,}0 \cdot 5 = 5 \text{ t/m}^2.$$

Thus if a soil is immerged, two pressures are distinguishable, one due to the solid and the other to the liquid phase. We shall see later that the pressures exerted by the solid phase of a soil on the face of a sloping wall are proportional to the vertical stresses multiplied by a factor λ more or less variable, while the hydrostatic pressure has the same intensity in all directions.

1.b *ANGLE OF INTERNAL FRICTION*

The angle of internal friction of a soil is a function of the rugosity of the particles of which it is composed and of its state of compaction. It is the decisive factor of the pressures that the soil can exert on an obstacle. Being of the order of 30-40° for sands or gravels, dry or immerged, it has relatively low values in the case of immerged coherent soils.

1.c *COHESION c*

The cohesion of a soil is the property of its constituent parts to cling together. Even fine sands always show some degree of cohesion due to the clayey impurities they generally contain.

Cohesion is a fairly uncertain property. It can diminish or even disappear entirely as, for example, in consequence of vibrations. The presence or absence of water can influence cohesion considerably. For this reason it is usually disregarded in most cases of practice. It is taken into account only in the case of unreservedly coherent soils and here again it is well to be cautious and not to overestimate its permanent effects.

A thorough knowledge of these three fundamental characteristics is essential in all conclusive bulkhead design.

Thus, the average values shown in Table I are recorded only as indications. These approximate values can, at the best, serve merely as the basis for estimating and summary examinations of a structure.

Table I

	γ_n (t/m³)	γ_i (t/m³)	ρ (°)	c (t/m²)
Sands and gravels	1,8	1,1	30–40	0
Silty sands	1,8	1,1	25–35	Slight
Clays and silt	1,8	1,0	10–25	0–10

In Items [1] to [7] of the publications cited in the appendix, the reader will find a great deal of information concerning the properties and characteristics of soils.

2. PRESSURE AND RESISTANCE OF SOIL

A mass of earth retained by a bulkhead exerts upon it lateral pressures that are related to:

1. Slopes of the free surface β of the back-fill and of the bulkhead α;
2. Unit weight γ, angle of internal friction ρ and the cohesion c of the back-fill;
3. Angle of friction δ of the earth on the wall.

Let us assume that the bulkhead is infinitely rigid and can be neither displaced nor deformed. In that case it is subjected by the back-fill to a pressure "at rest" proportional to a factor λ_0 more or less known (Fig. 1).

A slight yield of the flexible screen effected by pressure Q_a reduces the pressure on the rear face to a minimum value characterized by the coefficient λ_a of "active pressure".

On the contrary, a displacement of the screen towards the back-fill will produce soon a maximum of passive earth pressure Q_p, characterized by the coefficient of passive earth pressure or resistance λ_p.

The importance of yield, capable of developing in soils their forces of pressure or of resistance, is not well known. But experience has shown that the flexibility of steel sheet piling is sufficient (except perhaps in the case of sheet pile caissons of very high rigidity) to allow ground fully to develop completely either its minimum active pressure or its maximum passive pressure.

There will therefore be the two values λ_a and λ_p that determine the loading of a steel bulkhead.

The pressure exerted on a section of the surface of a retaining wall is given by the general expression:

$$q_a = p_h \cdot \lambda_a$$

in the case of active pressure and by:

$$q_p = p_{t'} \cdot \lambda_p$$

in the case of passive pressure.

p_h and $p_{t'}$ are the vertical stresses in the soil at the considered level. They are proportional to the sum of a possible surcharge of the ground and of the weight of the volume of earth above the considered level.

$$p_h = S + \gamma \cdot h$$

$$p_{t'} = S + \gamma \cdot t'$$

γ can be equal to γ_n or γ_i as the case may be.

Determination of the factors λ_a and λ_p may be achieved by admitting rectilinear or curvilinear equilibria, that is to say, by

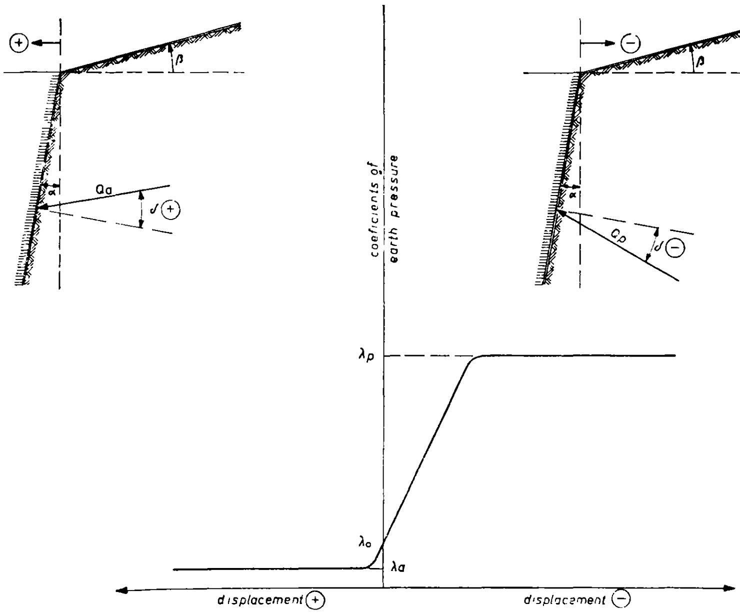
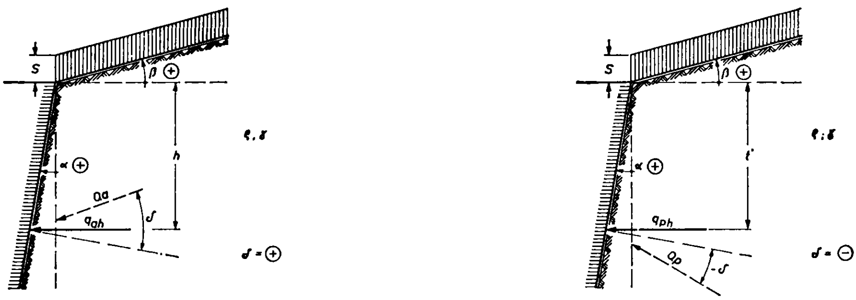


FIG. 1

assuming the sliding surfaces of the earth being straight or curved like circles or logarithmic spirals.

All these hypotheses, the development of which does not appear within the compass of this handbook, give almost identical values for λ_a but very divergent ones for λ_p, above all for high values of the angle of internal friction ρ combined with significant values for earth-wall friction δ. The hypothesis of plane sliding surfaces in this case gives exaggerated values for λ_p, even reaching infinity, which is in conflict with reality. However in normal conditions one can follow this hypothesis in design, for it has the advantage of being easier than the others, even if it must be rejected for high values of ρ and δ.



$$q_{ah} = p_h \lambda_{ah} = (s + \gamma h) \cdot \lambda_{ah}$$

$$\lambda_{ah} = \frac{\cos^2(\rho+\alpha)}{\cos^2\alpha \left[1 + \sqrt{\frac{\sin(\rho+\delta)\,\sin(\rho-\beta)}{\cos(\alpha-\delta)\,\cos(\alpha+\beta)}}\right]^2}$$

$$q_{ph} = p_t \lambda_{ph} = (s + \gamma t)\, \lambda_{ph}$$

$$\lambda_{ph} = \frac{\cos^2(\rho-\alpha)}{\cos^2\alpha \left[1 - \sqrt{\frac{\sin(\rho-\delta)\,\sin(\rho+\beta)}{\cos(\alpha-\delta)\,\cos(\alpha+\beta)}}\right]^2}$$

FIG. 2

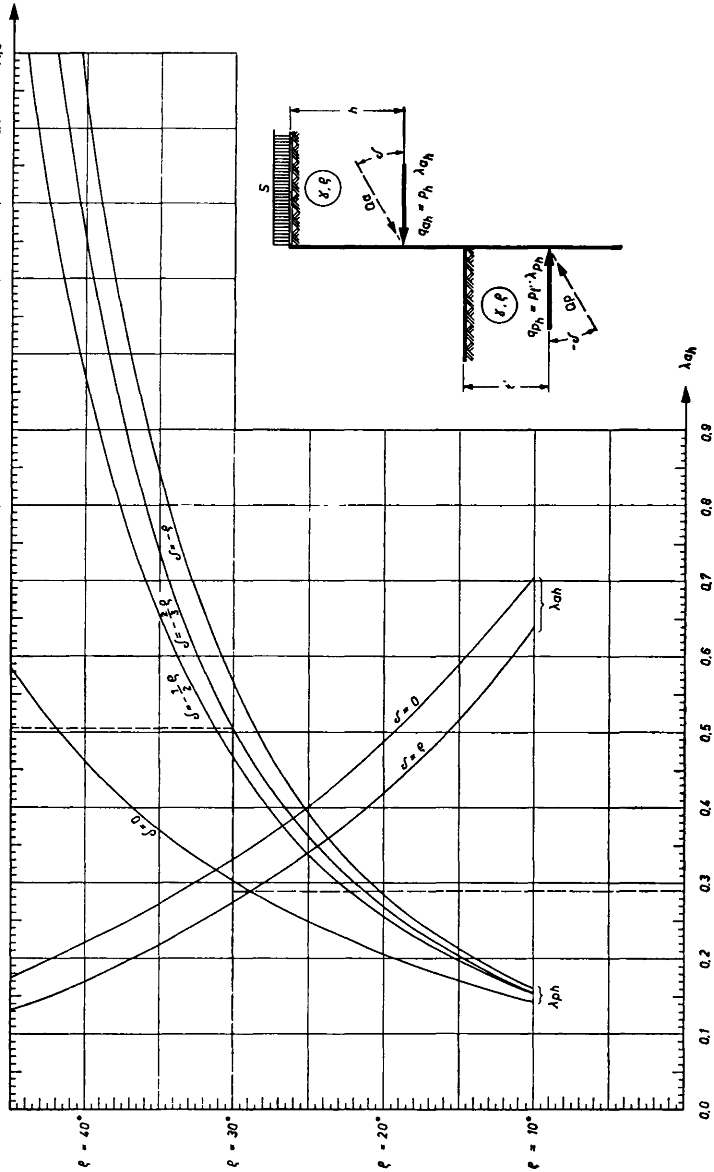
As in the design of sheet pile bulkheads one seeks first the horizontal component of the active and passive earth pressure we have shown in Figure 2 the formulae for estimating λ_{ah} and λ_{ph} in the hypothesis of plane sliding surfaces. In these formulae the angle of earth-wall friction δ must be regarded as positive in the case of active, and negative in the case of passive earth pressure.

For the friction of earth on steel sheet piling an angle δ can be allowed equal to about

$$\pm \tfrac{2}{3} \cdot \rho . \qquad [8]$$

As in most cases of bulkheads the wall is vertical and the ground is horizontal. Figure 3 shows for this case a diagram indicating with sufficient accuracy the values of λ_{ah} and λ_{ph} for the various values of ρ and of δ.

FIG. 3



This diagram has been prepared with the aid of tables of active and passive earth pressure drawn up by Messrs. A. Caquot and J. Kérisel [9] following a method that assumes curvilinear equilibria.

Let us examine the agreement of the formulae of Figure 2 with the values given in Figure 3, in a practical illustration.

Assuming

$$\rho = 30^\circ\,; \qquad \delta = \tfrac{2}{3}\cdot 30 = 20^\circ\,; \qquad \alpha = 0^\circ\,; \qquad \beta = 0^\circ\,.$$

In substituting these values in the formulae of Figure 2, we obtain

$$\lambda_{ah} = \frac{\cos^2(30+0)}{\cos^2 0\left[1+\sqrt{\dfrac{\sin(30+20)\sin(30-0)}{\cos(0-20)\cos(0+0)}}\right]^2}$$

$$= \frac{0{,}75}{1\left[1+\sqrt{\dfrac{0{,}765\cdot 0{,}5}{0{,}94\cdot 1{,}0}}\right]^2}$$

$$\lambda_{ah} = \frac{0{,}75}{[1+\sqrt{0{,}407}]^2} = \frac{0{,}75}{|1{,}64|^2} = 0{,}28$$

$$\lambda_{ph} = \frac{\cos^2(30-0)}{\cos^2 0\left[1-\sqrt{\dfrac{\sin(30+20)\sin(30+0)}{\cos(0+20)\cos(0+0)}}\right]^2}$$

$$= \frac{0{,}75}{1\left[1-\sqrt{\dfrac{0{,}765\cdot 0{,}5}{0{,}94\cdot 1{,}0}}\right]^2}$$

$$\lambda_{ph} = \frac{0{,}75}{|1-\sqrt{0{,}407}|^2} = \frac{0{,}75}{[0{,}36]^2} = 5{,}8$$

(It is to be observed that in the calculation of λ_{ph}, δ should be replaced by $-\delta$.)

Thus,

$$\lambda_{ah} = 0{,}28, \qquad \lambda_{ph} = 5{,}8.$$

In the graph of Figure 3, we obtain

$$\lambda_{ah} = 0{,}29, \qquad \lambda_{ph} = 5{,}05.$$

The agreement of the two methods is reasonably good, but nevertheless we observe that the formulae of Figure 2 already give more favourable results, a tendency that progresses with increasing values of ρ. It is for this reason that the plane sliding surface hypothesis must be rejected for values of ρ exceeding 35° combined with values over $\frac{1}{2}\rho$ for the angle δ of friction.

From the graph of Figure 3 we notice the considerable influence of earth-wall friction on the passive earth pressure whereas the effect of that same friction is far less in the case of the active pressure.

On the assumption that $\delta = 0$, and for horizontal back-fill surface and a vertical bulkhead, the coefficients of active and passive earth pressure are those given by the Rankine formula.

$$\lambda_{ah} = \mathrm{tg}^2\left(\tfrac{1}{4}\pi - \tfrac{1}{2}\rho\right)$$
$$\lambda_{ph} = \mathrm{tg}^2\left(\tfrac{1}{4}\pi + \tfrac{1}{2}\rho\right)$$

which have, in the case of the above illustration, the respective values:

$$\lambda_{ah} = \mathrm{tg}^2(45 - 15) = \mathrm{tg}^2\, 30° = 0{,}33$$

$$\lambda_{ph} = \mathrm{tg}^2(45 + 15) = \mathrm{tg}^2\, 60° = 3{,}0\,.$$

3. PREPARING A DIAGRAM OF ACTIVE EARTH PRESSURES

We have seen that the lateral pressure of a soil on a bulkhead at a point situated at depth h is given by the expression

$$q_a = p_h \cdot \lambda_a$$

and more specially by

$$q_{ah} = p_h \cdot \lambda_{ah}$$

in the case of horizontal pressure.

Knowing the vertical stress p_h at the considered level one may deduce the horizontal active pressure on the bulkhead by simple multiplication by the coefficient of horizontal active pressure λ_{ah}.

In Figure 4*a* a practical example is shown, neglecting the earth-wall friction. (That friction has not much influence on the active pressures, as was seen in paragraph 2 above.)

The bulkhead retains a back-fill consisting of two horizontal layers of different characters. The free water level and the water table in the back-fill are both at 4,0 m above the dredge line and the back-fill is assumed to bear a surcharge of $S = 2{,}0$ t/m².

At the level ⓪ the vertical stress is equal to the surcharge S

$$p_0 = S = 2{,}0 \text{ t/m}^2.$$

The horizontal pressure at that level is thus equal to

$$q_0 = p_0 \cdot \lambda_{ah} = 2{,}0 \cdot 0{,}27 = 0{,}54 \text{ t/m}^2.$$

At the level ① where there are changes in the characteristics of the soil, there prevails a vertical stress owing to the surcharge and to the volume of the upper layer of earth.

$$p_1 = p_0 + \gamma_n \cdot h_{01} = 2{,}0 + 1{,}65 \cdot 2{,}5 = 6{,}12 \text{ t/m}^2.$$

In the upper layer ($\rho = 35°$; $\lambda_{ah} = 0{,}27$) the horizontal pressure attains

$$q_1 = p_1 \cdot \lambda_{ah} = 6{,}12 \cdot 0{,}27 = 1{,}65 \text{ t/m}^2$$

and in the lower layer ($\rho = 30°$; $\lambda_{ah} = 0{,}33$)

$$q_1 = p_1 \cdot \lambda_{ah} = 6{,}12 \cdot 0{,}33 = 2{,}02 \text{ t/m}^2.$$

Beneath water level ②, the unit weight of the solid phase

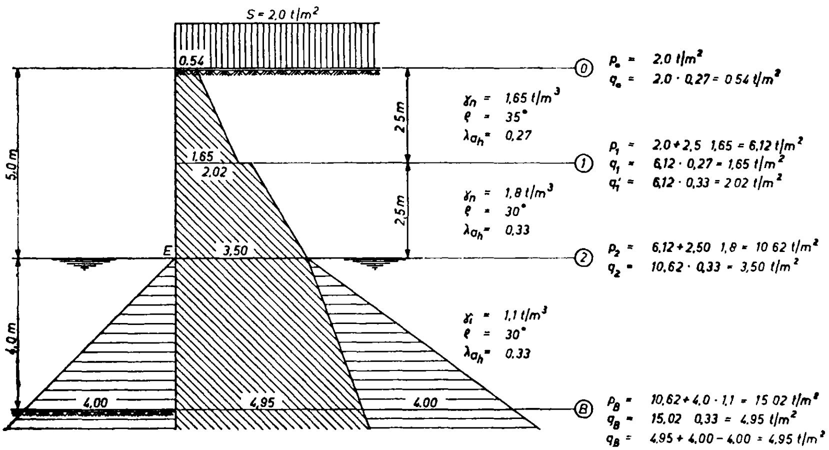


FIG. 4*a*

decreases from $\gamma_n = 1{,}8$ t/m³ to $\gamma_1 = 1{,}1$ t/m³ but there is, superimposed to the pressure of the solid phase, the hydrostatic pressure of the water in the back-fill, so that at level Ⓑ of the dredge line, the following pressures exist:

a) solid phase:

$$p_B = p_2 + \gamma_1 \cdot h_{2B} = 10{,}62 + 1{,}1 \cdot 4{,}0 = 15{,}02 \text{ t/m}^2,$$
$$q_B = p_B \cdot \lambda_{ah} = 15{,}02 \cdot 0{,}33 = 4{,}95 \text{ t/m}^2\,;$$

b) liquid phase:

$$p'_B = \gamma_0 \cdot h_{2B} = 1{,}0 \cdot 4{,}0 = 4{,}0 \text{ t/m}^2\,.$$

This hydrostatic pressure is transmitted entirely in all directions, for the coefficient of hydrostatic pressure λ_e is always equal to unity. We have therefore:

$$q'_B = p'_B \cdot \lambda_e = 4{,}0 \cdot 1{,}0 = 4{,}0 \text{ t/m}^2\,.$$

But the free water likewise has an influence on the pressure diagram. At dredge line level Ⓑ it exerts in its turn a horizontal pressure on the bulkhead equal to

$$q'_B = 4{,}0 \text{ t/m}^2$$

opposed to the pressure of the water in the back-fill.

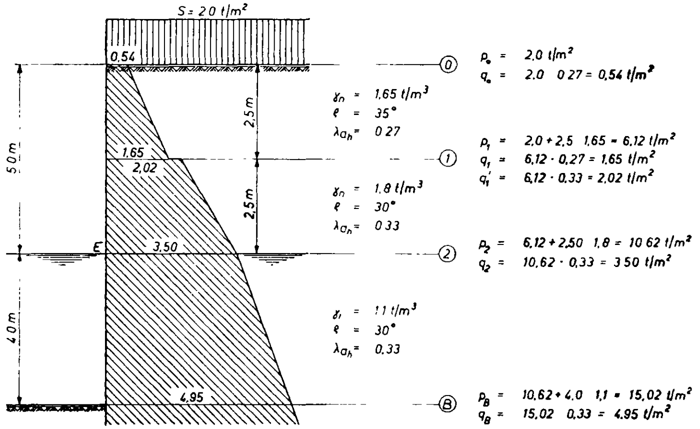


FIG. 4*b*

Thus the water pressures on the bulkhead cancel themselves out and the position is as if the bulkheads only load were the solid phase of the earth with submerged unit weight (Fig. 4*b*).

In the diagram shown in Figure 4*b* it is observed that a change in the angle of internal friction (therefore of the coefficient λ_{ah}) induces, at the considered level ①, a jump in the pressure diagram.

A change in the unit weight of the earth alters the slope of the linear distribution of pressures.

The example in Figure 4*c* indicates the distribution of pressures if, for one reason or another, water is held at different levels in front and on the rear side of the bulkhead.

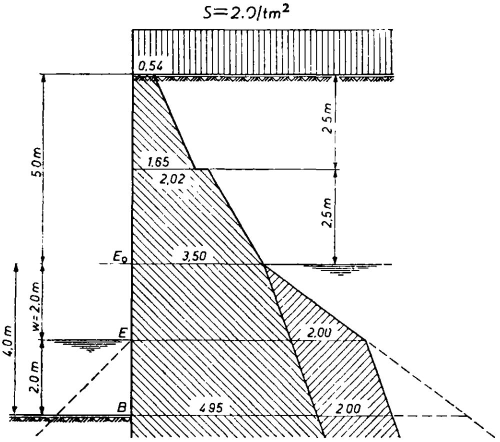


FIG. 4*c*

In this case the water pressures counteract each other only partly and only beneath the level of the free water in front of the bulkhead. Therefore, superimposed on the pressure of the solid phase, there is an unbalanced water pressure rising from zero at level E_0 and reaching a maximum value of $\gamma_0 \cdot w$ at level E. Below level E the unbalanced water pressure remains constant and equal to its maximum value $\gamma_0 w$, if γ_0 is the unit weight of the water and w the total hydraulic head.

4. PRESSURE DIAGRAM IN SOIL OF COHESION c

Cohesion diminishes the active pressure and increases the passive pressure of a soil.

In Figure 5 it is seen clearly that the effects of cohesion may be superimposed, in a way, on the effects derived from the non-cohesive medium alone.

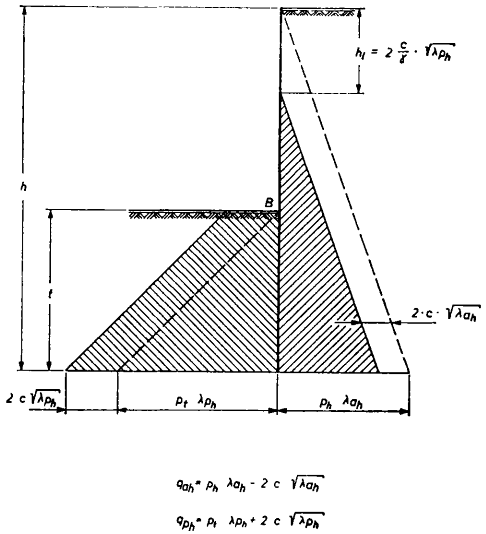


FIG. 5

On neglecting the earth-wall friction, the pressures exerted by the medium regarded as non-cohesive are:

$$q_{ah} = p_h \cdot \lambda_{ha} = p_h \cdot \mathrm{tg}^2 \left(\tfrac{1}{4}\pi - \tfrac{1}{2}\rho\right)$$
$$q_{ph} = p_t \cdot \lambda_{ph} = p_t \cdot \mathrm{tg}^2 \left(\tfrac{1}{4}\pi + \tfrac{1}{2}\rho\right).$$

Under the influence of cohesion the active pressures diminish by

$$2c\sqrt{\lambda_{ah}} = 2c \cdot \mathrm{tg}\left(\tfrac{1}{4}\pi - \tfrac{1}{2}\rho\right)$$

and the passive pressures increase by

$$2c\sqrt{\lambda_{ph}} = 2c \cdot \mathrm{tg}\left(\tfrac{1}{4}\pi + \tfrac{1}{2}\rho\right).$$

It is seen in Figure 5 that no pressure at all is exerted on the unsupported height h_l equal to $2 \cdot c/\gamma \cdot \mathrm{tg}\,(\frac{1}{4}\pi + \frac{1}{2}\rho)$. It is at that height h_l that the earth remains firm to a vertical wall without falling away.

It should be borne in mind however that the effects of cohesion are much less certain than those of internal friction and one should always be very cautious when assigning pressures of cohesive soils.

5. PRESSURE DIAGRAM ON THE BURIED PART OF A BULKHEAD

Consider an anchored bulkhead (Fig. 6*a*) driven into an assumedly homogeneous soil.

On the rear face of the bulkhead the back-fill exerts lateral pressures, the evaluation of which has been discussed in paragraph 3.

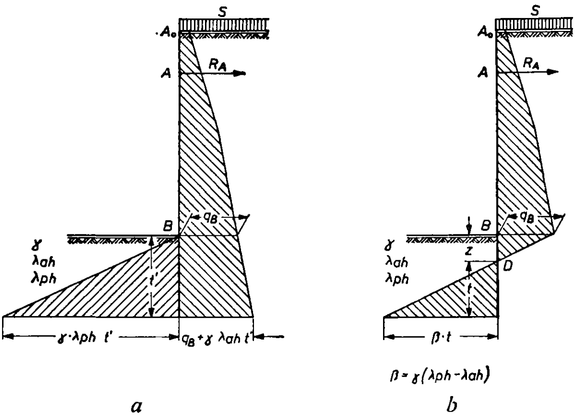


FIG. 6

Below the dredge line B, this pressure increases, in a homogeneous soil, linearly with the depth t', following the relation

$$q_B + \gamma \cdot \lambda_{ah} \cdot t'$$

where q_B is the lateral pressure at dredge line level B;

γ the unit weight of the soil, equal to γ_n or γ_i according to circumstances of the case;

λ_{ah} coefficient of horizontal active pressure;
t' the depth below dredge line level B.

Downstream the wall the yield of the flexible bulkhead gives rise to passive earth pressures that increase linearly with the depth and which can be assessed as

$$\gamma \cdot \lambda_{ph} \cdot t'$$

where λ_{ph} is the coefficient of horizontal passive pressure.

The superposition of these two diagrams of active and passive pressures provides the resultant diagram of pressures on the part embedded in the ground (Fig. 6*b*).

This diagram is characterised by a point D where the pressures, on both sides of the retaining wall, balance and result in zero pressure. The depth z of this point below dredge line level B is given by the relation

$$\boxed{z = \frac{q_B}{\beta}} \tag{1}$$

where q_B is the horizontal active earth pressure at B and

$$\boxed{\beta = \gamma\,(\lambda_{ph} - \lambda_{ah})} \tag{2}$$

the effective coefficient of horizontal passive earth pressure.

γ may be equal to γ_n or γ_i.

Below this point D of zero pressure the passive pressures of the soil are given by

$$\boxed{q_t = \gamma\,(\lambda_{ph} - \lambda_{ah})\,t = \beta \cdot t} \tag{3}$$

where t is the depth of the considered level below the point D.

It is this linear diagram that we shall adopt later in determining the depth of penetration of a bulkhead into the ground.

6. DEPTH OF PENETRATION OF A BULKHEAD

In Figure 7 we have sketched the loads and elastic lines of anchored bulkheads in relation to their depth of penetration.

The lateral active pressure on the retaining wall cannot be counterbalanced by the anchor pull alone. The bulkhead must find in the sub-soil a supplementary support which is now to be determined.

In case *a* of Figure 7 the passive pressures that the earth develops on the driven part of the bulkhead are too weak to hold it in place. It will fail due to dislocation of its toe C. There will thus occur at *C* a yield ΔC and an angular distortion γ_C of the bulkhead.

By increasing the penetration, a point is found where enough passive pressures are brought into play to maintain the toe C of the bulkhead in its place. It is the minimum depth of penetration that is shown in Figure 7*b*. The yield of the toe is zero but there remains an appreciable angular distortion γ_C. This penetration is absolutely necessary in order to ensure the stability of the bulkhead in the given conditions. But it is easy to conceive that any increase in active pressure due for example to an increase in the surcharge, will cause collapse of the bulkhead which, in these new conditions, will lack support.

Hence care must be taken to lengthen the penetration beyond the minimum in order to be secure under any possible temporary overstress.

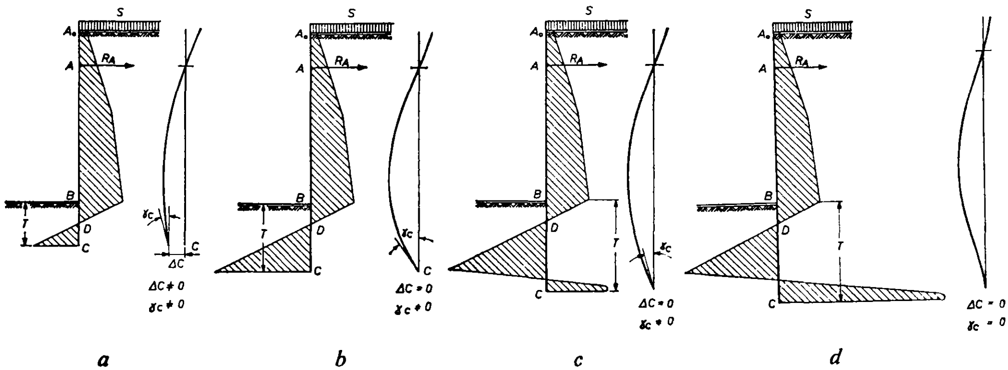


FIG. 7

On further prolonging the penetration, there appears counter-resistance of the soil on the rear side of the bulkhead, that has tendency to relieve the angular distortion of its toe. Thus one may speak, in the case of Figure 7*c*, of a partially fixed toe C in the ground. The angular distortion γ_C of the toe prevails but it is less pronounced than in the case of 7*b*. We have in this case a certain reserve of soil resistance that ensures stability of the bulkhead against the incidence of supplementary loading.

The ideal case of penetration of a bulkhead is that shown in Figure 7*d*; this disposes not only of the angular distortion γ_C but also of the yield ΔC of the toe C of the bulkhead. In this case, one may refer to a bulkhead with fixed earth support. An extension of penetration beyond this fixed end is not necessary.

7. METHODS OF BULKHEAD DESIGN

There are methods of bulkhead design derived from large scale model tests, but their proper application to other requirements is often difficult [10]. Other methods are purely empirical and their only justification is that structures, based upon those design methods, are still in good condition.

We restrict ourselves to deal here with the two so-called "classical" methods of design, based on the laws of elementary statics, namely:

a) The American method of bulkheads with free earth support;

b) The European method of bulkheads with fixed earth support.

a) The American method of the free earth support consists in finding the minimum depth of penetration of the bulkhead to assure its stability. As that penetration is the exact minimum required and to provide protection against possible supplementary stresses, a considerable safety factor is assigned to the coefficients of active pressure λ_{ah} and passive earth pressure λ_{ph} by neglecting the friction of earth on wall. The coefficients of pressure and resistance then have the respective values:

$$\lambda_{ah} = tg^2 \left(\tfrac{1}{4}\pi - \tfrac{1}{2}\rho\right)$$
$$\lambda_{ph} = tg^2 \left(\tfrac{1}{4}\pi + \tfrac{1}{2}\rho\right)$$

in the general case of vertical walls.

By way of additional security a supplement of some 20 % is added to this driving depth.

Experience has shown that the maximum bending moments and the anchor pulls, computed by this method, are higher than the values measured on site and are thus on the safe side.

Certain investigations and large scale model tests [11], in this respect have led to the diminution of bending moments and of anchor pulls in terms of the flexibility of the employed sections and of the degree of compaction of the back-fill.

It is possible that in the case where the bending moments and the anchor pulls have been found to be lower than the computed values, the penetration is such that we find it to be already in the region of what would be a partially fixed support. These considerations are outside the scope of this book. That is why we restrict ourselves to only mentioning the American method and giving in Chapter II an example of the design for the minimum depth of penetration of a bulkhead, which plays an essential part when designing the length of the anchor-rods in Chapter III.

b) The European method consists in computing the penetration of the fixed toe of the bulkhead.

This method was proposed by Dr. Blum [12] and it is the same author [13] who has demonstrated its validity, and above all its simplicity in comparison with other methods, perhaps more accurate but much more complicated.

Dr. Blum allows a distribution of pressure on the embedded portion as is shown in Figure 8*a*, the pressures in front q_l and behind the bulkhead q_r being of different intensities.

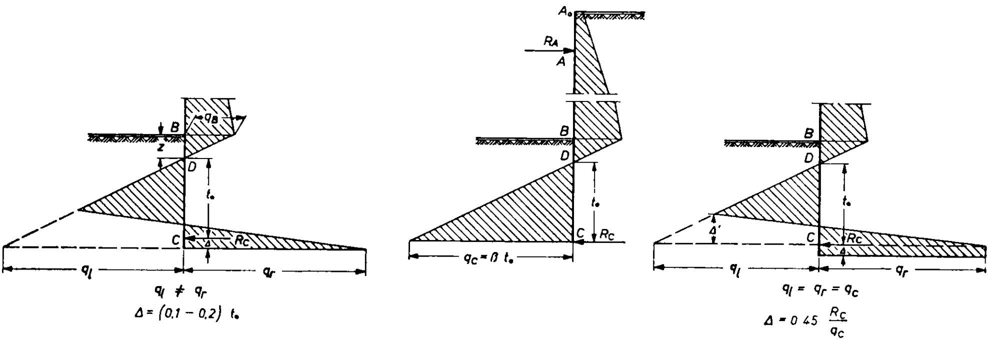
He replaces this diagram with an idealized one (Figure 8*b*) saying that the passive pressures in front of the bulkhead increase linearly with the depth down to the theoretical fixed toe C. The counter-resistance of the soil is replaced by a single force vector R_c which is assumed to originate at C.

The conditions governing the fixed end are:

a) The sum of all the horizontal forces must be zero;

b) The moment about C, caused by all the pressures acting upon the bulkhead, must likewise counterbalance.

It is this idealized diagram that we shall use later to compute the penetration for the fixed end condition of anchored bulkheads. To allow the full development of the counter-resistance R_c Dr. Blum proposes to add to the theoretical penetration t_0 a supplement Δ varying from 10 to 20 % of t_0 in so far as the rate of pressure q_r/q_l varies from 2 to 1.



a *b* *c*

FIG. 8

L. Descans [14] starts from another hypothesis to determine the value of Δ.

Assuming (Fig. 8*c*) that the passive pressures at the theoretical toe C, q_r and q_l, are of equal intensity q_c, and furthermore that the intensity of the counter-resistance below C remains constant and equal to q_c, he obtains the counter-resistance R_c at C as the force vector of a triangle and a rectangle.

The equations of translation and rotation equilibrium about C give as extra length

$$\boxed{\Delta = \frac{R_c}{(1+\sqrt{1{,}5})\cdot q_c}} \quad \text{or} \quad \boxed{\Delta = \frac{0{,}45\, R_c}{q_c}} \qquad (4)$$

That value of Δ is about 15 % of the theoretical penetration t_0 below D.

CHAPTER II

NUMERICAL EXAMPLES OF BULKHEAD DESIGN

1. GENERAL CONSIDERATIONS ON THE DESIGN

The static design of a continuous bulkhead is generally made over a unit portion of bulkhead that may be taken for example equal to 1 m.

The computation can be undertaken analytically. That method is a long and tedious one and embraces many sources of error that are difficult to detect, even for a trained calculator.

Graphical design, on the contrary, has the advantage of being fairly rapid and to present results in the form of a simple and clear drawing from which all the necessary elements for the choice of sections can be taken by simple measurement.

The accuracy of results obtained by graphical design is usually of greater precision than that of the data available for the design.

a) The first step in the design consists in setting out the diagram of pressures acting upon the bulkhead.

Having established the diagram of active pressures (see Chapter I.3) the depth z of the point D of zero pressure below the dredge line is evaluated following the formula

$$z = \frac{q_B}{\beta}, \tag{1}$$

if q_B is the horizontal active pressure at the dredge line level B and

$$\beta = \gamma\,(\lambda_{ph} - \lambda_{ah}), \tag{2}$$

the residual coefficient of the horizontal passive pressure, γ may be equal to γ_i or γ_n whether the ground is submerged or not.

Below D, the passive pressures increase in proportion to $\beta \cdot t$ if t is the depth of the considered level below D.

In order to obviate cumbersome drawings, it is recommended to reduce the scale, representing passive pressures below D, in comparison with that of the active pressures.

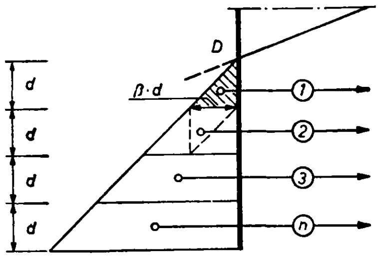
b) In the second place, the pressure diagram is split into triangular and trapezoidal areas and these are replaced by their force vectors acting at the gravity centres of the areas.

By the reduction of the scale representing passive earth pressures precision of the pointing is lessened. It is therefore expedient to proceed in the following way in determining the force vectors in the passive pressure diagram.

From the point of zero pressure D, the triangular diagram is split into a triangle and trapezes of the same height d.

The intensity of the first force vector is in this case:

$$① = \beta \cdot \tfrac{1}{2} d^2 .$$



It is easy to show that the intensities of the force vectors ②, ③, etc., are respectively equal to

$$② = 3 \times ①$$

$$③ = 5 \times ①$$

. . . .

$$ⓝ = (2n - 1) \times ①$$

Thus, calculating the first force vector of the soil resistance it remains only to multiply that force vector by the sequence of the odd numbers to obtain the succeeding force vectors of the passive earth pressure. This procedure is far more accurate than any graphical pointing.

For greater ease we have admitted the gravity centres of the trapezes at their midheight. The approximation of this short cut is all the greater as the subdivisions are smaller and generally the error involved is compatible with the accuracy of graphical design.

c) In collecting these force vectors to a string polygon, it is easy to compute the line of moments of the bulkhead.

d) The closing line of the polygon of moments varies according to the support conditions in the soil. The determination will be imparted in the examples of this chapter. Let us assume that it has been found.

e) We now in turn break up the moment area into elastic weights that are applied anew, in the direction of their action, on a beam that is assumed to replace the bulkhead.

f) Finally one draws up the elastic line of the bulkhead which is none other than the funicular polygon of moments of the beam acted upon by the elastic weights.

It is not out of place to add a few words on the comparatively new designation of scales that is introduced here.

The scales are defined for the original drawing (but not for the reduced size blocks of the following examples) by the ration

$$\frac{\text{size of unit}}{\text{representation of unit}}$$

If, always on the original drawing, one metre for example is represented by 2 cm on the drawing, the scale of length is given by the ratio

$$e_L = \frac{1\ \text{m}}{2\ \text{cm}}.$$

In splitting up the pressure diagram, force vectors are obtained for which, according to their magnitude, for example, the following scale is chosen

$$e_F = \frac{2\,t}{1\,cm}.$$

By means of this scale the force vectors are assembled in a string polygon for which a polar distance $\mathcal{H}$ is chosen arbitrarily, let us assume, for example, 5 cm. The scale of moments is thus given by the simple expression.

$$e_M = e_L \cdot e_F \cdot \mathcal{H},$$

which, with the scales adopted above, gives

$$e_M = \frac{1\,m}{2\,cm} \cdot \frac{2\,t}{1\,cm} \cdot 5\,cm = \frac{5\,tm}{1\,cm}.$$

One cm on the original drawing therefore represents a bending moment of 5 tm.

After splitting up the diagram of moments delimited by the line of moments and the closing line, a scale is chosen appropriate to the elastic weights expressed in tm^2.

Let us assume the choice of

$$e_{E-W} = \frac{10\,tm^2}{1\,cm},$$

With the string polygon of the elastic weights having a polar distance $\mathcal{H}$, the elastic line of the bulkhead is computed, of which the scale is given by:

$$e_t = e_L \cdot e_{E-W} \cdot \mathcal{H}$$

or, with the values adopted above:

$$e_t = \frac{1\,m}{2\,cm} \cdot \frac{10\,tm^2}{1\,cm} \cdot 5\,cm = \frac{50\,tm^3}{2\,cm}.$$

The yield is thus given in tm^3, a purely relative value of which the real yield F of the elastic line in cm is deduced by dividing the value given in tm^3, or rather in $kg \cdot cm^3$ ($1\,tm^3 = 10^9\,kg \cdot cm^3$) by the expression

$$E \cdot I \text{ in } kg \cdot cm^2$$

elastic constant of the chosen section.

E is the modulus of elasticity of the steel equal practically to $E = 2,1 \cdot 10^6$ kg/cm².

I is the moment of inertia of the sheet piles in cm^4/m, value given in the catalogue and in the attached table.

2. THE CANTILEVER WALL

2.a *GENERAL CONSIDERATIONS*

Back-fill of no great height can be retained by a bulkhead driven simply into the soil and having no other supplementary support above the dredge line.

In that case the pressures exerted on the cantilevered part of the bulkhead should be counterbalanced by the reaction of the earth on the embedded part. Such a bulkhead has to be fixed in the soil.

It is easy to understand that the penetration of cantilevers must be far greater than that of the anchored bulkheads and that the stress on the buried part can reach an excessive value, when the height of the retained back-fill becomes excessive while at the same time the top of the cantilever begins to show severe yield which is not permissible. That is why the use of the cantilever bulkhead is not very economical and for that reason it is fairly restricted in application.

2.b *GRAPHICAL DESIGN METHOD*

In Figure 9 we deal with the case of a cantilever bulkhead retaining the bank of a canal. In preparing the pressure diagram we have neglected the earth-wall friction.

Having chosen a convenient scale for the force vectors of the pressures acting upon the wall, the line (or rather the polygon) of moments is computed by use of the string polygon. "The exact theoretical penetration of the cantilever is cut off by the intersection of the line of moments with the tangent at its origin, in this case the vertical line at the top of the wall."

In the case examined the theoretical depth of penetration beneath D is equal to $t_0 = 3{,}75$ m.

The determination of the extended length Δ needs a little intermediate calculation:

$$\beta = \gamma_i (\lambda_{ph} - \lambda_{ah}) = 2{,}94 \text{ t/m}^3.$$

Unit pressure at C:

$$q_c = \beta \cdot t_0 = 2{,}94 \cdot 3{,}75 = 11{,}02 \text{ t/m}^2.$$

Total horizontal soil resistance from D to C:

$$Q_{ph} = q_c \cdot \frac{t_0}{2} = 11{,}02 \cdot \frac{3{,}75}{2} = 20{,}67 \text{ t}.$$

Total active earth pressure:

$$Q_{ah} = 4{,}65 \text{ t}.$$

Counter-resistance:

$$R_c = Q_{ph} - Q_{ah} = 20{,}67 - 4{,}65 = 16{,}02 \text{ t}.$$

Foreseen extension of length

$$\Delta = \frac{0{,}45\, R_c}{q_c} = 0{,}45 \frac{16{,}02}{11{,}02} = 0{,}65 \text{ m}.$$

In the string polygon of the force vectors of the soil resistance it is seen that a parallel to the closing line of the moment area, outlines the same counter-resistance $R_c = 16{,}02$ t.

By splitting up the diagram of moments and on applying the elastic weights to a beam representing the bulkhead one computes likewise the elastic line of the bulkhead assumed as being fixed at C.

The representative scales of the moments and of the elastic line are set up with the help of the known formulae (see under II.1)

$$e_M = e_L \cdot e_F \cdot \mathcal{H},$$
$$e_t = e_L \cdot e_{E-W} \cdot \mathcal{H},$$

where $\mathcal{H}$ is always the polar distance of the corresponding string polygon.

The problem is resolved.

The choice of section is a function of the maximum bending moment of the wall.

$$M_{max} = 13{,}85 \text{ tm}.$$

In selecting a steel of 50/60 kg/mm² quality with a permissible working stress of

$$\sigma = 1\,800 \text{ kg/cm}^2$$

choice should be made of a section having a section modulus of

$$W = \frac{M_{max}}{\sigma} = \frac{13{,}85 \cdot 10^5}{1{,}8 \cdot 10^3} 770 \text{ cm}^3/\text{m}$$

which corresponds to that of the section

BZ I RA with W = 850 cm³/m.

The maximum yield at the top of the cantilever is equal to

$$f_A = 260 \text{ tm}^3/\text{m}.$$

The moment of inertia of BZ IRA being $I = 7\,100$ cm⁴/m and the modulus of elasticity of the steel being $E = 2{,}1 \cdot 10^6$ kg/cm² the actual yield of the top A_0 of the cantilever amounts to:

$$F = \frac{f_A}{E \cdot I} = \frac{260 \cdot 10^9}{2{,}1 \cdot 10^6 \cdot 7{,}1 \cdot 10^3} = 17{,}5 \text{ cm}.$$

The more rigid section BZ II N ($I = 13\,200$ cm⁴/m) will reduce that yield to 9,4 cm.

This large yield of a cantilever bulkhead under load is its major drawback which, according to circumstances, may demand the use of so heavy and rigid a section that it would be more economical to provide anchorage of the top of the wall in so far as local conditions will permit.

It remains to be added that in the region of the fixed toe C the actual line of moments will rather follow the dotted line in Figure 9. But that is not important in the progress of the calculation.

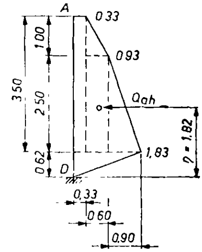
We have likewise indicated in Figure 9 the origin of the force vector Q_{ah} of the total active pressures which is situated at

$$\eta = 1{,}82 \text{ m}$$

above level D. The origin of Q_{ah} is obtained from the intersection of the tangent at its origin with the tangent at the point of counterflexure of the line of moments, that point of counterflexure being found at level D of zero pressure.

The value of η plays a part in the following paragraph.

ΣQ_{ah}	η	$\Sigma M = Q\ \eta$
$0,33 \cdot 3,50 = 1,15$	2,37	2,73
$0,60 \ \frac{1\,00}{2} = 0,30$	3,45	1,04
$0,60 \ 3,50 = 1,50$	1,87	2,80
$0,90 \ \frac{2\,50}{2} = 1,12$	1,45	1,63
$1,83 \ \frac{0\,62}{2} = 0,57$	0,41	0,23
$Q_{ah} = 4,64$		$8,43 = M_D$



$$\eta = \frac{M_D}{Q_{ah}}$$

$$r = \frac{6\ Q_{ah}}{\beta\ \eta^2} \longrightarrow \vartheta,\ \mu$$

$$t_0 = \vartheta\ \eta$$

$$M_{max} = \mu\ M_D$$

$\beta = 2,94\ t/m^3$
$\eta = 1,82\ m$
$r = 2,85$
$\vartheta = 2,05$
$\mu = 1,64$
$t_0 = 3,74\ m$
$M_{max} = 13,80\ tm$

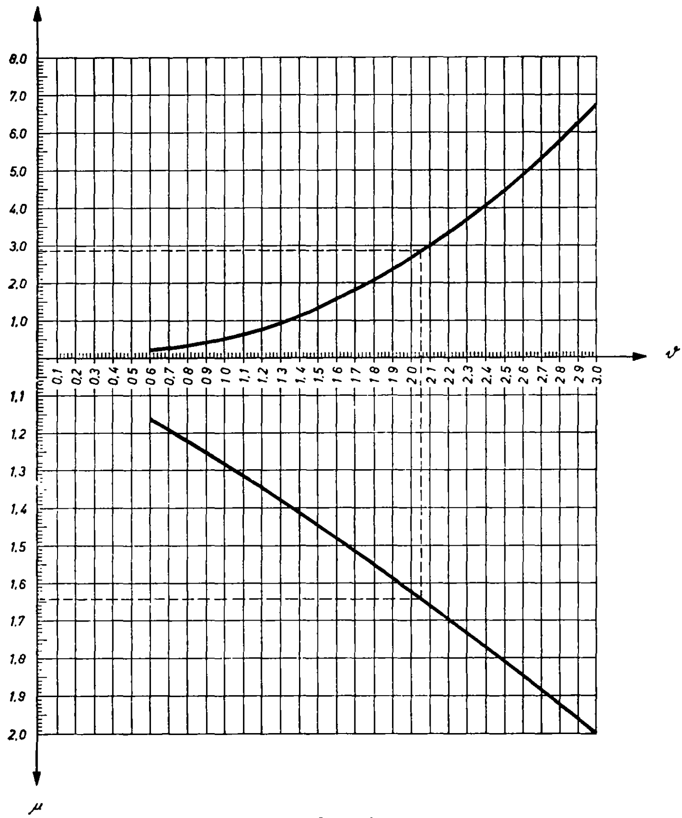


FIG. 10

2.c *ANALYTICAL METHOD COMBINED WITH GRAPH*

The peculiarity of the cantilever wall allows the immediate determination of the theoretical depth of embedment beneath D and of the maximum bending moment of the wall by means of a graph, if the magnitude and the origin of the force vector Q_{ah} of the total active pressures is known.

In splitting up the diagram of active pressures into rectangles and triangles, the force vector of the active pressures Q_{ah} and the cantilever moment M_D caused by this force vector is calculated following the scheme shown in figure 10. In the column headed η there is entered each time the distance of the level of the gravity centre of the partial area above the point D.

The height of origin of Q_{ah}, that is the distance between the level of the gravity centre of the active pressure diagram and the point D is given by

$$\eta = \frac{M_D}{Q_{ah}} = \frac{8{,}43}{4{,}64} = 1{,}82\ \text{m}\ .$$

Bearing in mind that $\beta = \gamma_1 (\lambda_{ph} - \lambda_{ah}) = 2{,}94\ \text{t/m}^3$, the constant r is given by:

$$r = \frac{6 \cdot Q_{ah}}{\beta \cdot \eta^2} = \frac{6 \cdot 4{,}64}{2{,}94 \cdot 1{,}82^2} = 2{,}85\ .$$

With that value of $r = 2{,}85$ we read at once on the graph of Figure 10,

$$\vartheta = 2{,}05 \qquad \text{and} \qquad \mu = 1{,}64$$

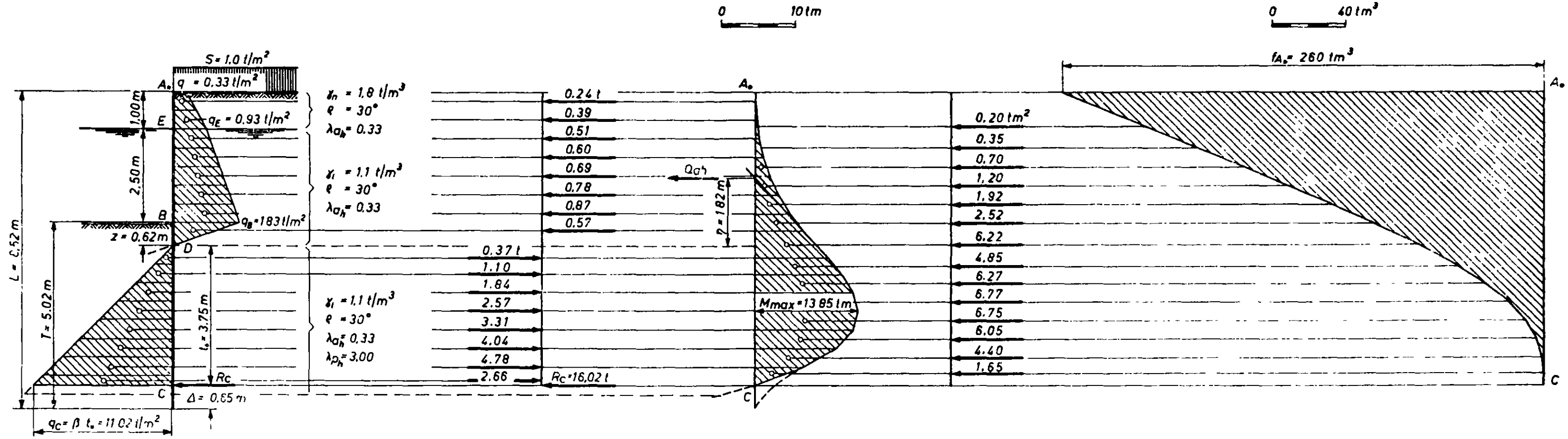
which gives the theoretical depth of embedment beneath D and the maximum bending moment by the relations:

$$t_0 = \eta \cdot \vartheta = 1{,}82 \cdot 2{,}05 = 3{,}74\ \text{m};$$

$$M_{max} = \mu \cdot M_D = 1{,}64 \cdot 8{,}43 = 13{,}80\ \text{tm}.$$

The problem is completely solved if Δ is calculated as is shown under 2.*b*.

The interested reader will find in the Appendix the assumptions made to lead to the graph in Figure 10.



$p_0 = S = 1.0\ t/m^2$

$q_0 = 1.0 \cdot 0.33 = 0.33\ t/m^2$

$p_E = 1.0 + 1.8 \cdot 1.0 = 2.8\ t/m^2$

$q_E = 2.8 \cdot 0.33 = 0.93\ t/m^2$

$p_B = 2.8 + 1.1 \cdot 2.5 = 5.55\ t/m^2$

$q_B = 5.55 \cdot 0.33 = 1.83\ t/m^2$

$\beta = \gamma_1 (\lambda p_h - \lambda a_h) = 1.1\,(3.0 - 0.33) = 2.94\ t/m^3$

$z = \frac{q_B}{\beta} = \frac{1.83}{2.94} = 0.62\ m$

$Q_{p_h} = q_C \frac{t_0}{2} = 11.02 \cdot \frac{3.75}{2} = \mathbf{20.67\ t}$

$\Delta = \frac{R_C}{2.22 \cdot q_C} = \frac{16.02}{2.22 \cdot 11.02} = 0.65\ m$

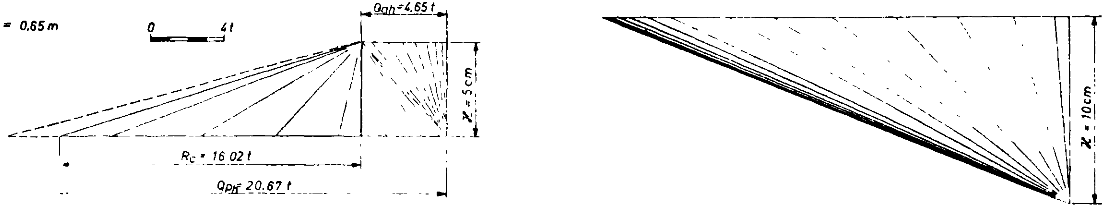


FIG. 9

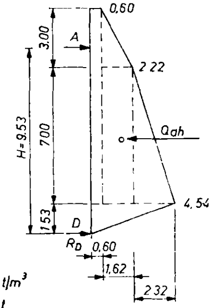
3. THE ANCHORED BULKHEAD

3.a *DETERMINATION OF THE MINIMUM DEPTH OF PENETRATION*

The minimum depth of penetration is of interest while designing the lengths of the tie-rods (Chapter III*b*).

We give hereafter two methods, analytical and graphical, that can be combined to determine this minimum penetration.

$\Sigma\ Q_{ah}$	η	$\Sigma M = Q\ \eta$
0,60 10,00 = 6 00	6.53	39,20
1,62 $\frac{3.00}{2}$ = 2.43	9 53	23 20
1,62 7 00 = 11.34	5,03	57,10
2.32 $\frac{7.00}{2}$ = 8,11	3,86	31.30
4,54 $\frac{1.53}{2}$ = 3.47	1,02	3.50
Q_{ah} = 31,35		154 30 = M_D



$$R_D = Q_{ah} - \frac{M_D}{H}$$

$$r = \frac{6\ R_D}{\beta\ H^2} \longrightarrow \vartheta$$

$$t_{min} = \vartheta\ H$$

β = 2,97 t/m³

R_D = 15,15 t

r = 0 336

ϑ = 0 308

t_{min} = 2.94 m

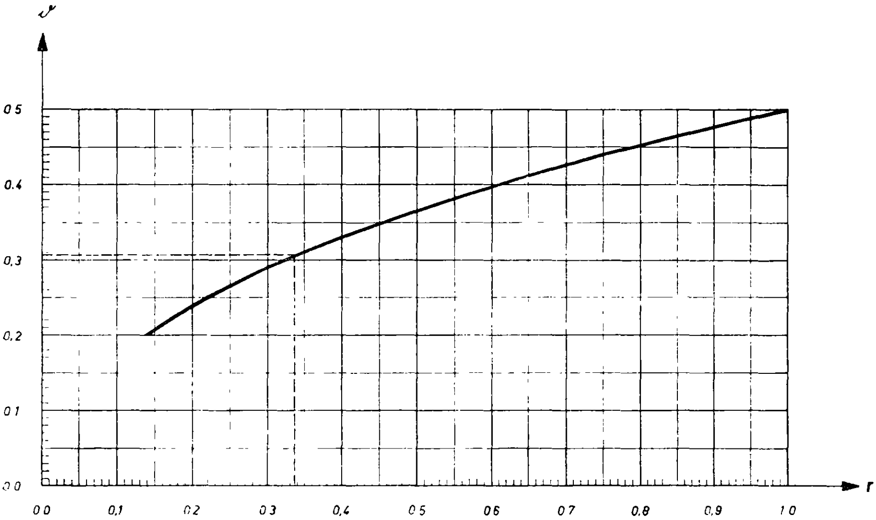


FIG. 11

3.a.a Analytical method combined with graph (Figure 11)

After having prepared the pressure diagram for the beam AD (see example under 3.*a.b*), it is split up into rectangular and triangular areas and the stress on the beam is calculated with the aid of the scheme of Figure 11.

The first column of the scheme contains the calculation of the force vectors of the partial areas, the second, the distances of the gravity centres of the partial areas above point D and the third, the cantilever moment in D caused by all the partial areas. By addition one obtains the force vector of the total lateral pressure $Q_{ah} = 31{,}35$ t and the cantilever moment in D: $M_D = 154{,}30$ tm.

It is easily shown (for proof, see Appendix) that the minimum depth of penetration beneath D of a bulkhead may be ascertained by a very simple relation if the reaction of support R_D of the beam A_0D, freely supported at A and D, at a distance of H metres, is known.

The reaction R_D is deduced from available data by

$$R_D = Q_{ah} - \frac{M_D}{H} = 31{,}35 - \frac{154{,}30}{9{,}53} = 15{,}15\,\text{t}\,.$$

Knowing also that $\beta = \gamma_1 (\lambda_{ph} - \lambda_{ah}) = 2{,}97$ t/m^3 one calculates the constant

$$r = \frac{6 \cdot R_D}{\beta \cdot H^2} = \frac{6 \cdot 15{,}15}{2{,}97 \cdot 9{,}53^2} = 0{,}336\,.$$

In the graph of Figure 11, we deduce starting from the abcissa $r = 0{,}336$, the value of

$$\vartheta = \frac{t_{min}}{H} = 0{,}308$$

from which we obtain

$$t_{min} = \vartheta \cdot \text{H} = 0{,}308 \cdot 9{,}53 = 2{,}94\ \text{m}.$$

For our needs knowledge of the penetration is all that is necessary. The complete design of the bulkhead is carried out more advantageously by the graphical method given, for information, in the following paragraph.

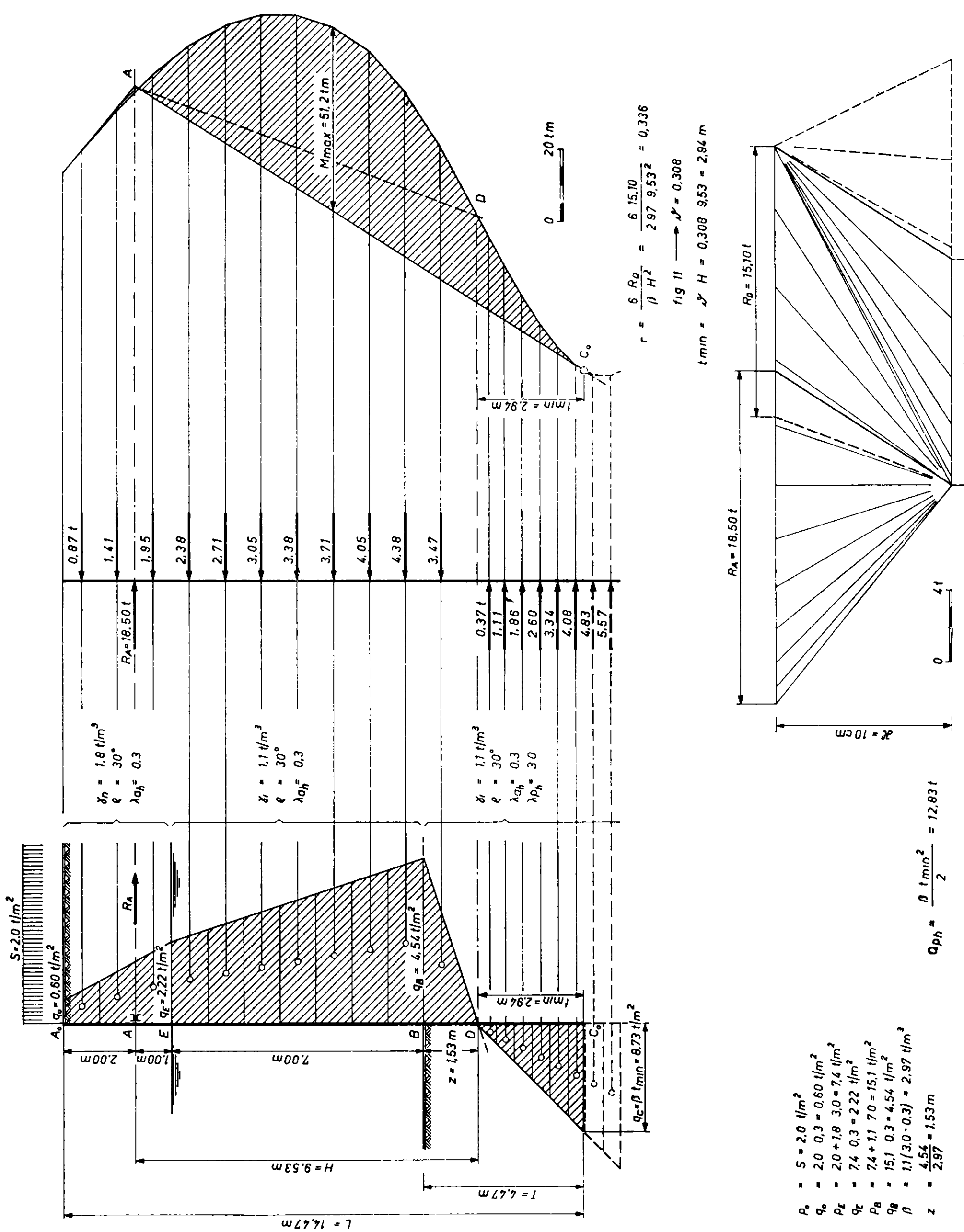

S = 2.0 t/m²
q₀ = 0.60 t/m²
q_E = 2.22 t/m²
q_B = 4.54 t/m²
R_A
R_A = 18.50 t
M_max = 51.2 tm
0 20 tm
t min = 2.94 m
0.87 t
1.41
1.95
2.38
2.71
3.05
3.38
3.71
4.05
4.38
3.47
0.37 t
1.11
1.86
2.60
3.34
4.08
4.83
5.57
2.00 m
1.00 m
7.00 m
z = 1.53 m
H = 9.53 m
T = 4.47 m
L = 14.47 m
q_c = β t_min = 8.73 t/m²
r = 6 R_a / β H² = 6 15.10 / 2.97 9.53² = 0.336
fig 11 → ν = 0.308
t min = ν H = 0.308 9.53 = 2.94 m
R_a = 15.10 t
Q_ph = 12.83 t
R_A = 18.50 t
ℋ = 10 cm
0 4 t
Q_ph = β t min² / 2 = 12.83 t
p₀ = S = 2.0 t/m²
q₀ = 2.0 0.3 = 0.60 t/m²
p_E = 2.0 + 1.8 3.0 = 7.4 t/m²
q_E = 7.4 0.3 = 2.22 t/m²
p_B = 7.4 + 1.1 7.0 = 15.1 t/m²
q_B = 15.1 0.3 = 4.54 t/m²
β = 1.1 (3.0 - 0.3) = 2.97 t/m³
z = 4.54 / 2.97 = 1.53 m

FIG. 12

3.a.b **Graphical method** (Figure 12)

The course to follow in order to prepare the pressure diagram is given in Figure 12. With the string polygon one computes the polygon of moments which presents no particular difficulty. The closing line of the moment area should satisfy two conditions:

a) To intersect the tangent at the origin of the line of moments at the level of anchorage A

b) To be tangent to the line of moments at the toe C_0.

The first condition is easy to achieve. But it is difficult to draw a tangent to a polygon that replaces the "line" of moments. So a more accurate process must be adopted.

In the preceding paragraph we indicated such a step which consists in taking the value of

$$\vartheta = \frac{t_{min}}{H}$$

from a graph if one knows R_D, the reaction of support at D of the beam A_0D freely supported at A and at D, with a span H. However the quest for that reaction R_D is easy if one has already computed the line of moments.

The line AD, intersecting the line of moments at the level D of zero pressure is drawn (indicated as a dotted line in Figure 12). A parallel to that line outlines the reaction R_D in the string polygon of the active force vectors

$$R_D = 15{,}10 \text{ t.}$$

That value of R_D enables calculation of the constant:

$$r = \frac{6 \cdot R_D}{\beta \cdot H^2} = 0{,}336$$

which in turn, provides immediately a reading from the graph in Figure 11 of $\vartheta = 0{,}308$.

The minimum penetration below D therefore is

$$t_{min} = \vartheta \cdot \text{H} = 0{,}308 \cdot 9{,}53 = 2{,}94 \text{ m}$$

which allows the definite drawing of the line AC_0, closing the area of moments.

In the string polygon of active force vectors a parallel to this closing line AC_0 outlines the anchor pull $R_A = 18{,}50$ t and another parallel gives the total passive earth pressure $Q_{ph} = 12{,}83$ t in the string polygon of the passive force vectors.

The maximum bending moment of the bulkhead equals

$$M_{max} = 51{,}2 \text{ tm}.$$

The scale of moments having been calculated according to formula,

$$e_M = e_L \cdot e_F \cdot \mathcal{H}$$

(on this point see II.1).

We do not dwell on the determination of the elastic line of the bulkhead as it is of no particular interest.

3.b *DETERMINATION OF THE PENETRATION FOR THE FIXED END CONDITION OF AN ANCHORED BULKHEAD*

The fixed earth support assures the best conditions to the anchored bulkhead in normal service. At the same time it is also the maximum depth of penetration which it is useless to increase.

We describe hereafter two methods used for determining the penetration for fixed bulkhead toe, namely:

— A graphical method, approximate but often sufficiently exact;

— A graphical method that uses "the error" of the elastic line of the bulkhead to correct the already computed driving depth.

3.b.a **Dr. Blum's "equivalent beam"**

In dealing with numerous examples, Dr. Blum [12] has established that the bending moment of an anchored bulkhead with fixed earth support is reduced to zero in immediate proximity to level D of zero pressure. From this he has derived a very simple method that is rapid and often sufficiently accurate for determining the penetration for fixed earth support of an anchored bulkhead.

Since the moment about D is nil (Fig. 13) the wall portion A_0D may be considered as a beam supported at A and at D and loaded by active pressures.

Having prepared the diagram of active pressures on the part A_0D of the wall the line of moments can be computed without difficulty. The closing line should satisfy two conditions:

— To intersect the tangent to the origin at anchorage level A;
— To intersect the line of moments on level D.

A parallel to the line AD cuts off in the string polygon the reactions

$$R_A = 15{,}05 \text{ t} \qquad \text{and} \qquad R_D = 14{,}93 \text{ t}.$$

The moment at D and at C being zero the lower part DC of the wall can be regarded as a beam freely supported at D and at C and loaded with the triangular passive earth pressures.

An equation of moments about the support C gives

$$R_D \cdot t'_0 - \beta \cdot t'_0 \cdot \frac{t'_0}{2} \cdot \frac{t'_0}{3} = 0$$

from which one obtains the value of the depth of penetration below D

$$t'_0 = \sqrt{\frac{6 \cdot R_D}{\beta}}$$

equal to 4,25 m in the example given in Figure 13.

The counter-resistance R_c at the toe of the bulkhead is equal to

$$R_c = \beta \cdot \frac{t'^2_0}{3} \quad \text{or also} \quad R_c = 2 \cdot R_D = 29{,}86 \text{ t}.$$

The extension Δ to be added to this theoretical depth of embedment beneath D to allow the development of the counter-resistance is equal to

$$\Delta = 0{,}45 \frac{R_c}{q_c}$$

if q_c is the unit pressure at the theoretical fixed toe C

$$\Delta = 0{,}45 \frac{29{,}86}{21{,}04} = 0{,}64 \text{ m}.$$

The maximum bending moment is equal to

$$M_{max} = 32{,}4 \text{ tm}.$$

The problem is solved, accurately enough as can be verified by comparing results with those obtained in the example dealt with in 3.*b.b.*

The rapidity of this method can facilitate considerably the examination of several solutions under consideration.

It goes without saying that the penetration t_0' may also be deduced by an analytical computation of R_D.

3.b.b **Hedde's correction** [15]

We have separated in Figure 14 the two phases of the design. The method of preparing the pressure diagram is given in Figure 13.

After splitting up the diagram of lateral pressures acting upon the bulkhead, one computes the line of moments with the aid of the string polygon.

A first approximation to the penetration of fixed earth support is obtained from the closing line ① which intersects the moment at the level D of zero pressure (see 3.*b.a.*).

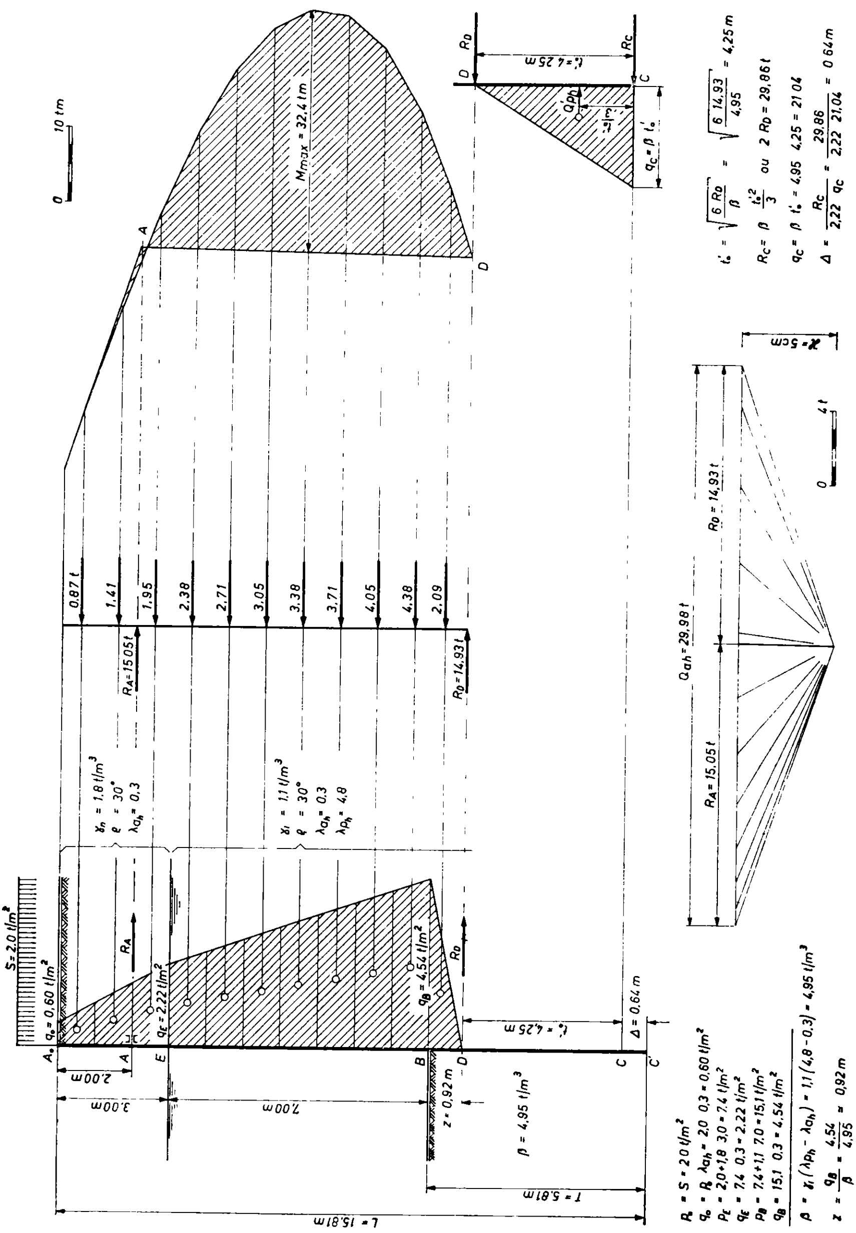
In splitting up the moment area into elastic weights applied to a beam representing the bulkhead, we compute the elastic line ① of the wall which generally will not pass through the point A, situated on a vertical rising from the fixed toe of the wall.

It is said that the "error" of the elastic line at A (level of anchorage) is Δ_{fA}, expressed as tm^3.

In the case of Figure 14, that error amounts to -80 tm^3.

The sign ⊖ indicates that the elastic line passes upstream of the point A. In the contrary event, the sign ⊕ should be applied.

Hedde has developed a formula that makes use of this peculiarity of the first elastic line to determine the supplement of the moment ΔM_C that should be added to the moment area at level C to counteract the error of the elastic line at A.



S = 2.0 t/m²
q₀ = 0.60 t/m²
q_E = 2.22 t/m²
q_B = 4.54 t/m²
γ_n = 1.8 t/m³
ρ = 30°
λa_h = 0.3
γ₁ = 1.1 t/m³
ρ = 30°
λa_h = 0.3
λp_h = 4.8
R_A = 15.05 t
R_D = 14.93 t
0.87 t
1.41
1.95
2.38
2.71
3.05
3.38
3.71
4.05
4.38
2.09
M max = 32.4 tm
Q_ah = 29.98 t
R_A = 15.05 t
R_D = 14.93 t
2.00 m
3.00 m
7.00 m
z = 0.92 m
β = 4.95 t/m³
t'₀ = 4.25 m
Δ = 0.64 m
T = 5.81 m
L = 15.81 m
Q_ph
q_C = β t'₀
p₀ = S = 2.0 t/m²
q₀ = p₀ λa_h = 2.0 0.3 = 0.60 t/m²
p_E = 2.0+1.8 3.0 = 7.4 t/m²
q_E = 7.4 0.3 = 2.22 t/m²
p_B = 7.4+1.1 7.0 = 15.1 t/m²
q_B = 15.1 0.3 = 4.54 t/m²
β = γ₁(λp_h − λa_h) = 1.1(4.8 − 0.3) = 4.95 t/m³
z = q_B/β = 4.54/4.95 = 0.92 m
t'₀ = √(6 R_D/β) = √(6 14.93/4.95) = 4.25 m
R_C = β t'₀²/3 ou 2 R_D = 29.86 t
q_C = β t'₀ = 4.95 4.25 = 21.04
Δ = R_C/(2.22 q_C) = 29.86/(2.22 21.04) = 0.64 m

FIG. 13

The formula is:

$$\boxed{\Delta M_c = \frac{3 \cdot \Delta f_A}{l_1^2}}$$

where Δ_{fA} is the error of the elastic line at the level A expressed in tm^3 and l_1 the distance in metres between the origine of anchorage A and the position given to the toe C in first approximation.

In the case of Figure 14 we obtain:

$$\Delta M_c = \frac{3 \cdot (-80)}{13{,}16^2} = -1{,}38 \text{ tm} .$$

In bringing ΔM_C (having regard to the sign ⊖) the closing line ① is pivoted about the point A in the position ② which is final and gives the theoretical penetration of fixed earth support by its intersection with the line of moments.

The elastic line ② obtained by splitting up the moment areas delimited by line ② passes through the point A.

A parallel to the closing line ② of the moment areas outlines in the string polygon of the active force vectors the anchor pull

$$R_A = 14{,}88 \text{ t}.$$

The unit pressure at the toe C of the bulkhead is equal to

$$q_c = \beta \cdot t_0 = 4{,}95 \cdot 4{,}30 = 21{,}30 \text{ t/m}^2.$$

The total horizontal passive pressure on the downstream face of the bulkhead equals

$$Q_{ph} = q_c \frac{t_0}{2} = 21{,}30 \cdot \frac{4{,}30}{2} = 45{,}80 \text{ t} .$$

The counter-resistance appears as

$$R_c = Q_{ph} + R_A - Q_{ah} = 45{,}80 + 14{,}88 - 29{,}88 = 30{,}70 \text{ t}.$$

The extension Δ is equal to

$$\Delta = 0{,}45 \frac{R_c}{q_c} = 0{,}45 \cdot \frac{30{,}70}{21{,}30} = 0{,}65 \text{ m} .$$

It is to be observed that a parallel to the line ② outlines in the string polygon of passive force vectors the counter-resistance

$$R_c = 30{,}70 \text{ t}.$$

It is interesting to compare results from the two methods described in 3.*b.a* and 3.*b.b*.

We obtain:

	Equivalent Beam	Exact Method
t_0 (m)	4,25	4,30
M_{max} (tm) . . .	32,40	31,70
R_A (t)	15,05	14,88
R_c (t)	29,86	30,70

The theoretical depth of penetration for fixed earth support of an anchored bulkhead obtained by Hedde's correction is sufficiently accurate in practice and one would be deluded in striving for greater precision.

Choice of sheet pile section

For an allowable stress $\sigma = 1\,800$ kg/cm² of a steel of 50/60 kg/mm² quality the section modulus of the sheet piles should be equal to:

$$W = \frac{M_{max}}{\sigma} = \frac{31{,}7 \cdot 10^5}{1{,}8 \cdot 10^3} = 1\,760 \, \text{cm}^3/\text{m}.$$

In chosing section BZ III N

$$W = 1\,750 \, \text{cm}^3/\text{m}, \qquad I = 22\,750 \, \text{cm}^4/\text{m}$$

the permissible stress of 1 800 kg/cm² will not be reached.

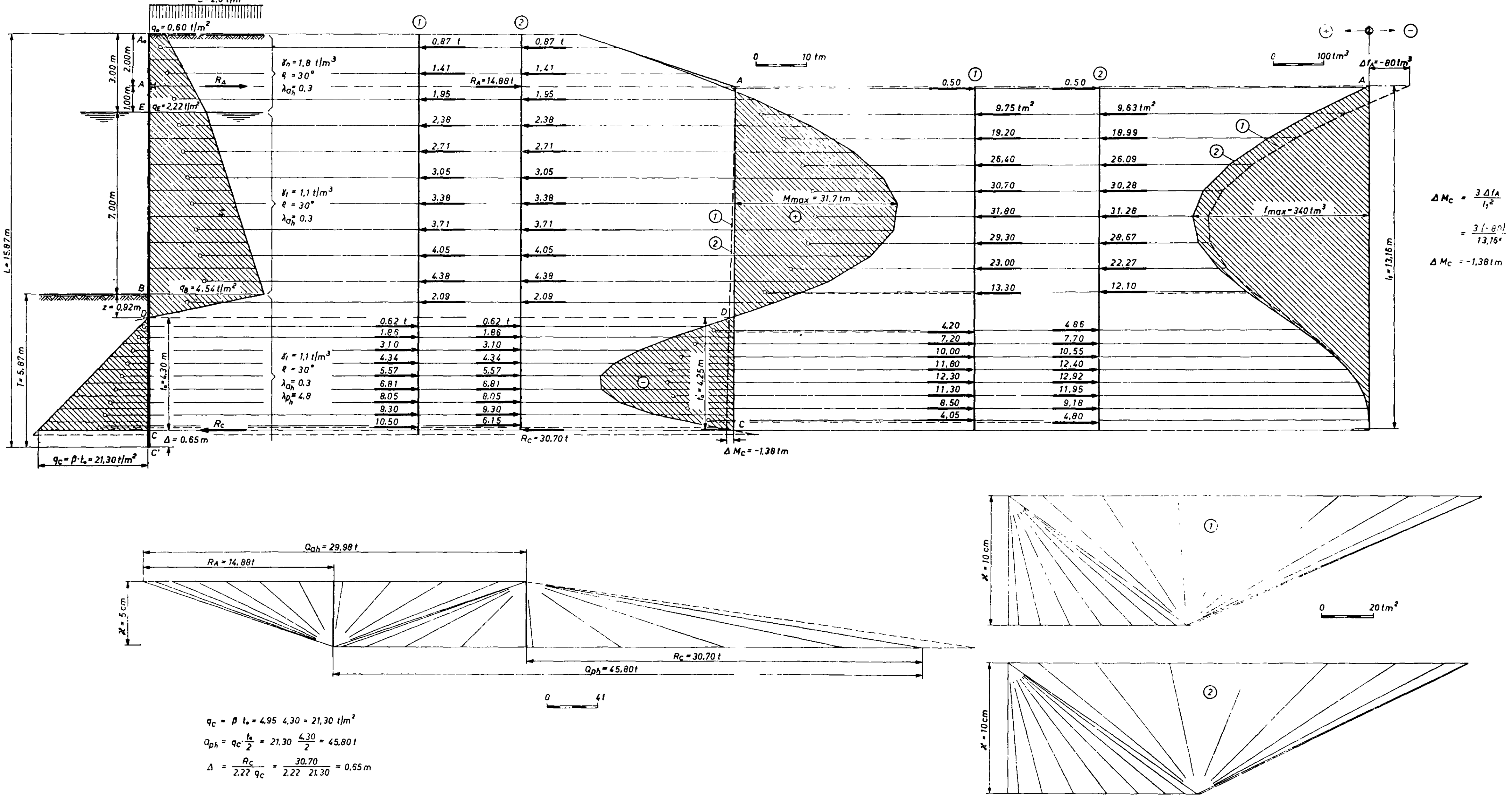
In this case the maximum yield of the bulkhead is at the level indicated in Figure 14 and its extent is

$$F = \frac{f_{max}}{EI} = \frac{340 \cdot 10^9}{2{,}1 \cdot 10^6 \cdot 22{,}75 \cdot 10^3} = 7{,}15 \, \text{cm}.$$

The total length of the sheet piles is equal to

$$L = 10 + 0{,}92 + 4{,}30 + 0{,}65 = 15{,}87 \, \text{m}$$

say $L = 15{,}90$ m.



S = 2.0 t/m²
q₀ = 0.60 t/m²
γn = 1.8 t/m³
ρ = 30°
λah = 0.3
γ1 = 1.1 t/m³
ρ = 30°
λah = 0.3
γ1 = 1.1 t/m³
ρ = 30°
λah = 0.3
λph = 4.8
qE = 2.22 t/m²
qB = 4.54 t/m²
z = 0.92 m
t₀ = 4.30 m
Δ = 0.65 m
L = 15.87 m
T = 5.87 m
qc = β·t₀ = 21.30 t/m²
RA
RC
RA = 14.88 t
RC = 30.70 t
Mmax = 31.7 tm
t₀' = 4.25 m
ΔMC = -1.38 tm
fmax = 340 tm³
ΔfA = -80 tm³
l₁ = 13.16 m
ΔMC = 3 ΔfA / l₁²
= 3 (-80) / 13.16²
ΔMC = -1.38 tm
0 10 tm
0 100 tm³
0 20 tm²
0 4 t
Qah = 29.98 t
RA = 14.88 t
RC = 30.70 t
Qph = 45.80 t
ϰ = 5 cm
ϰ = 10 cm
qc = β t₀ = 4.95 · 4.30 = 21.30 t/m²
Qph = qc · t₀/2 = 21.30 · 4.30/2 = 45.80 t
Δ = RC / (2.22 qc) = 30.70 / (2.22 · 21.30) = 0.65 m

FIG. 14

4. SPECIAL CASES

4.a *EXCEPTIONAL STRESSES*

Like all engineering works, a sheet pile bulkhead is exposed to temporary load increments, be it in consequence of an unforeseen increase of the surcharge or by a lowering of the level of the dredge line B where the first resistant forces become evident or, again, by an unbalanced water pressure caused by the immersion of the back-fill.

The stability of the bulkhead is assured, when its embedded portion is sufficient to set up sufficient soil resistance to hold the toe of the wall back and when, at the same time, the maxima stresses are such that they do not exceed at any point, the elastic limit of the material. The study of several possible, exceptional cases will show whether the bulkhead can resist within the limiting conditions of static equilibrium.

It is only such study as this that provides sound control of the reliability of the structure.

In Figure 15, following the example cited in section 3, three checks are made of exceptional stresses by assuming successively,

— an increase of 100 % of the surcharge;

— a lowering of the dredge line by 1 m;

— an unbalanced water pressure in the back-fill.

For each case of new stress a new pressure diagram is first established, modified as compared with that of normal load.

With the string polygon of the force vectors resulting from the splitting up of the pressure diagram the line of moments is computed.

The theoretical toe C was found in section 3 previously, at the depth

$$T - \Delta = 5{,}87 - 0{,}65 = 5{,}22 \text{ m}$$

below the dredge line.

The new closing line should cancel out the moment at this level of the theoretical toe and intersect the tangent at the origin at level A of the anchorage.

In the three cases treated in Figure 15 it is seen that the bulkhead remains partly fixed in the ground. In any event the closing line becomes tangent to the line of moments although in the case of a fall of the dredge line (case *b)* it becomes dangerously near.

The soil resistance induced by the penetration of fixed earth support is therefore sufficient to keep the bulkhead in position.

But it is noted that the bending moment of the bulkhead increases appreciably relatively to the $M_{max} = 31{,}7$ tm under normal load.

The bulkhead having been dimensioned with section BZ III N, (section modulus $W = 1\,750$ cm³/m) it is seen that the stresses reach respectively

$$\sigma_a = \frac{43{,}0 \cdot 10^5}{1\,750} = 2\,460 \text{ kg/cm}^2$$

$$\sigma_b = \frac{54{,}5 \cdot 10^5}{1\,750} = 3\,100 \text{ kg/cm}^2$$

$$\sigma_c = \frac{68{,}4 \cdot 10^5}{1\,750} = 3\,900 \text{ kg/cm}^2 .$$

The 50/60 kg/mm² quality of steel having a minimum elastic limite of 3 000 kg/cm², these stresses are reached by a fall of the dredge line and are greatly exceeded by an unbalanced water pressure.

The anchor pull increases from its normal value, $R_A = 14{,}88$ t

a) $R_A = 20{,}40$ t $(= +36\ \%)$;

b) $R_A = 19{,}50$ t $(= +31\ \%)$;

c) $R_A = 26{,}63$ t $(= +79\ \%)$.

The safety factor 1.5 generally adopted for designing the anchor wall will be no longer sufficient in the case of an unbalanced water pressure.

This comparison shows that:

— a local and temporary increase in surcharge is only to be feared if it assumes really exceptional proportions;

— a fall of the dredge line can quickly create a dangerous situation. The reduction in the support of the bulkhead in the ground is accompanied by a considerable increase of the stresses in the steel. The wall may in these circumstances yield as easily from movement of its toe as well as from fatigue of its material;

— an hydraulic head is particularly dangerous. It can cause complete destruction of the work through lack of resistance of the material.

Therefore if such stresses are possible, even temporarily, care should be taken to control the stability of the bulkhead or to find means to avoid their disastrous effects. A layer of ballast in front of the bulkhead provides a usefull protection against the lowering of the dredge line, due to the eddies caused by ships' propellers.

An hydraulic head can be avoided by arranging an efficient drainage system in the bulkhead.

4.b *ANCHORED WALL CARRYING VERTICAL LOAD*

The steel sheet pile bulkhead is often used to carry one of the runways of the cranes installed on the platform, if the loading due to those cranes is not too important.

In such cases, to avoid the task of having to ensure the stability of the top of the bulkhead by supplementary anchorage, the main anchorage is placed as high up as possible, even at the top of the bulkhead using the runway itself as a waling.

These vertical loads stress the bulkhead in addition to the lateral pressures set up by the back-fill.

The procedure then is as follows:

One begins by selecting a section of sheet pile capable of withstanding the lateral earth pressure. That section is checked for to resist the supplementary vertical load without exceeding a reasonable limit of stress. It is also important to verify that the bulkhead is able to transmit that load to the subsoil.

In Figure 16 we deal with the checks generally accepted in such a case.

Without reproducing all the intermediate computations we cite the principal characteristics of the stresses of the bulkhead and those of the selected sheet pile section:

Maximum bending moment: $M_{max} = 15{,}5$ tm;

Linear anchor pull: $R_A = 6{,}5$ t;

Total penetration of fixed earth support: $T = 4{,}67$ m;

Relative yield of the elastic line: $f_{max} = 118$ tm^3;

Vertical load: $P = 10$ t.

The selected section is BZ II N with following characteristics:

Moment of inertia: $I = 13\,200$ cm^4/m;

Section modulus: $W = 1\,200$ cm^3/m;

Section of steel: $s = 155{,}5$ cm^2/m;

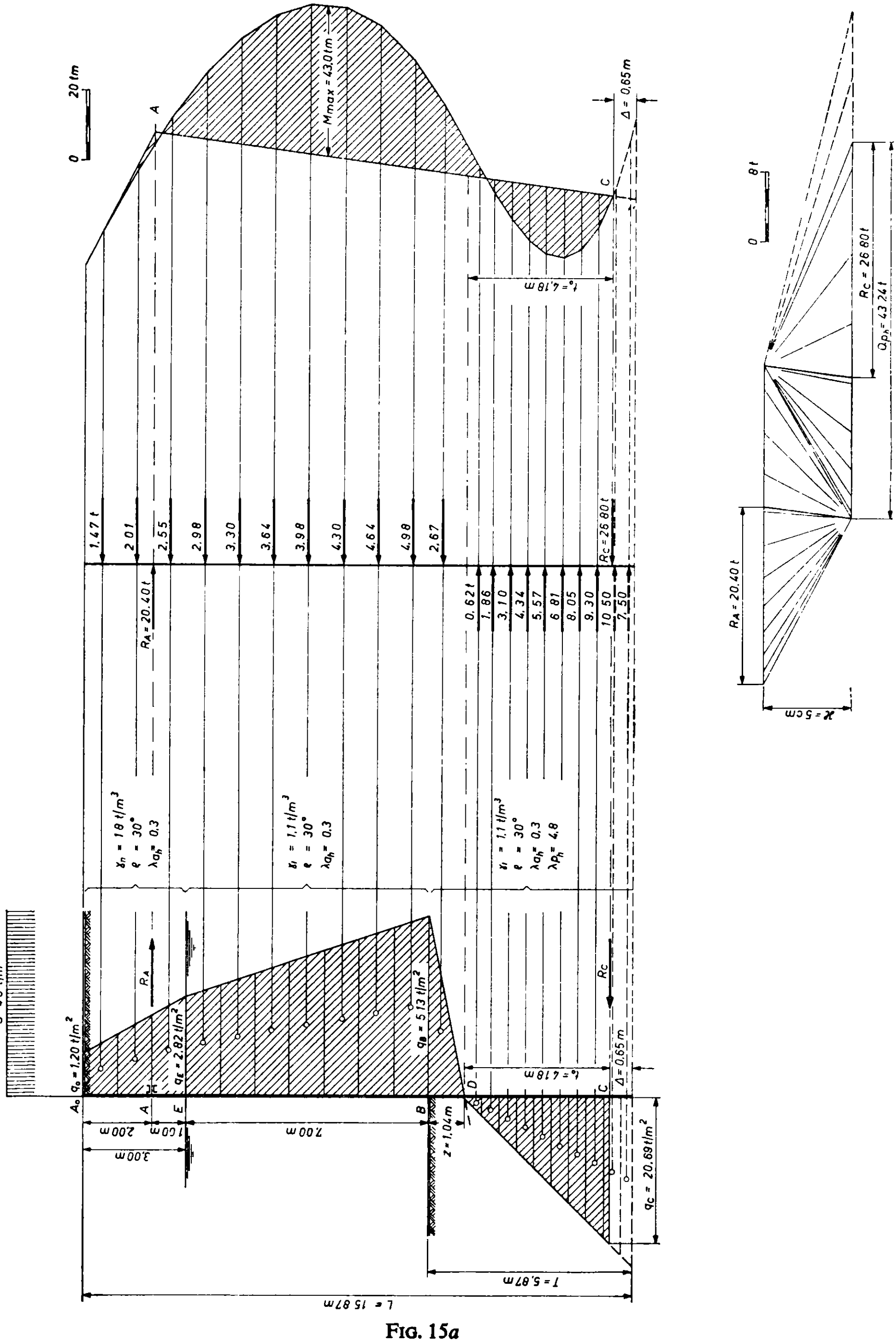
Perimeter of section: $p = 277$ cm/m;

Radius of gyration: $r = 9{,}21$ cm.

4.b.a Transmission of load to ground

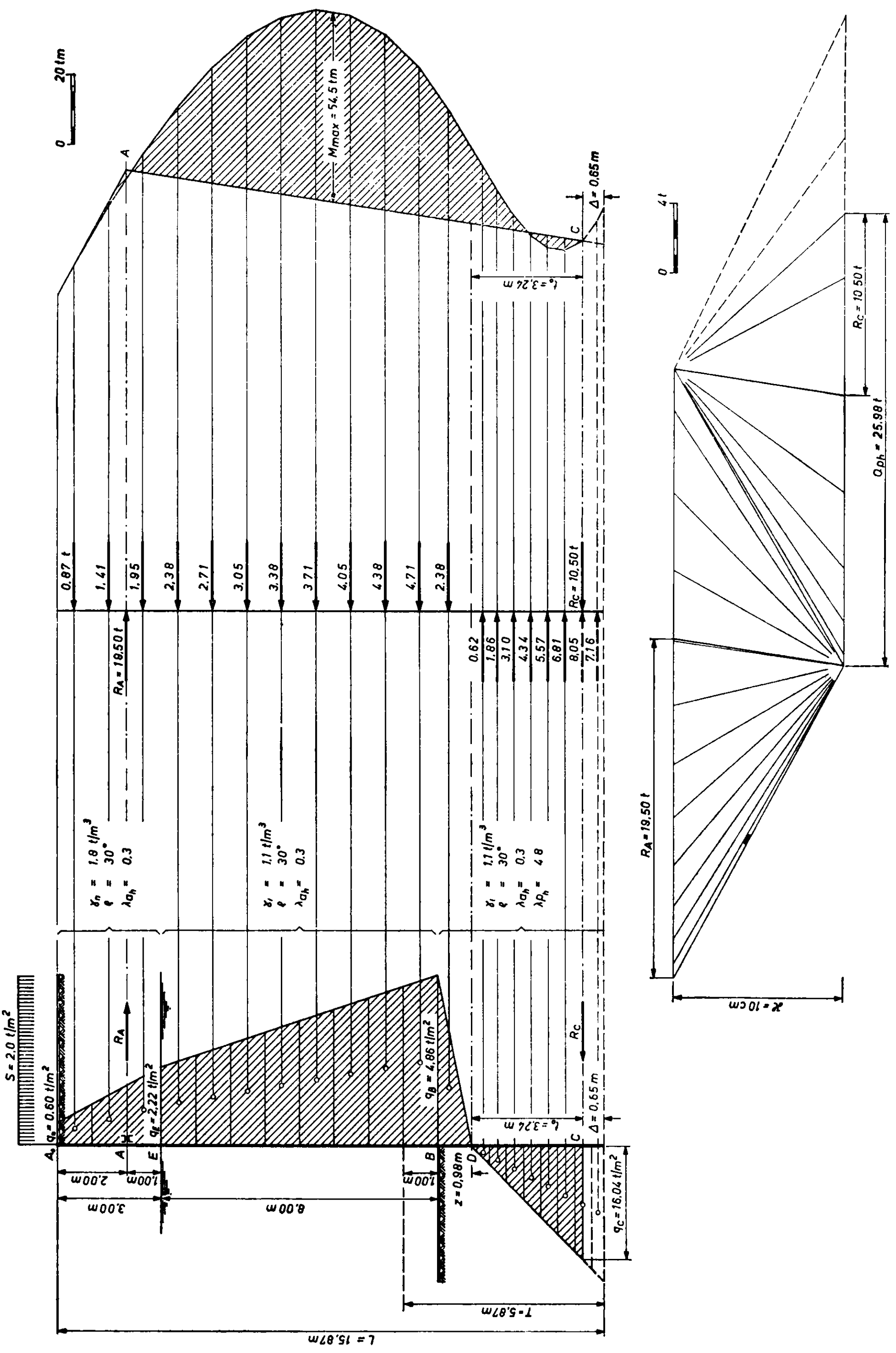
Because the steel section of the sheet piles is very small in relation to their surface $p \cdot T$ in contact with the ground, the point resistance of a sheet pile bulkhead can be neglected. The whole load P ought to be transmitted to the underground by the friction of the earth on the driven bulkhead surface.

As it is an issue, in many practical problems, to get an idea of the magnitude of the resistance that can be developed by friction between earth and wall, we give below a simple formula, deduced from the so-called "Dörr" formula for assessing the load that a bulkhead, driven to a depth of T m, can transmit to the ground. If the vertical load is less than the resultant of the frictional forces between earth and wall, its transmission is assured.



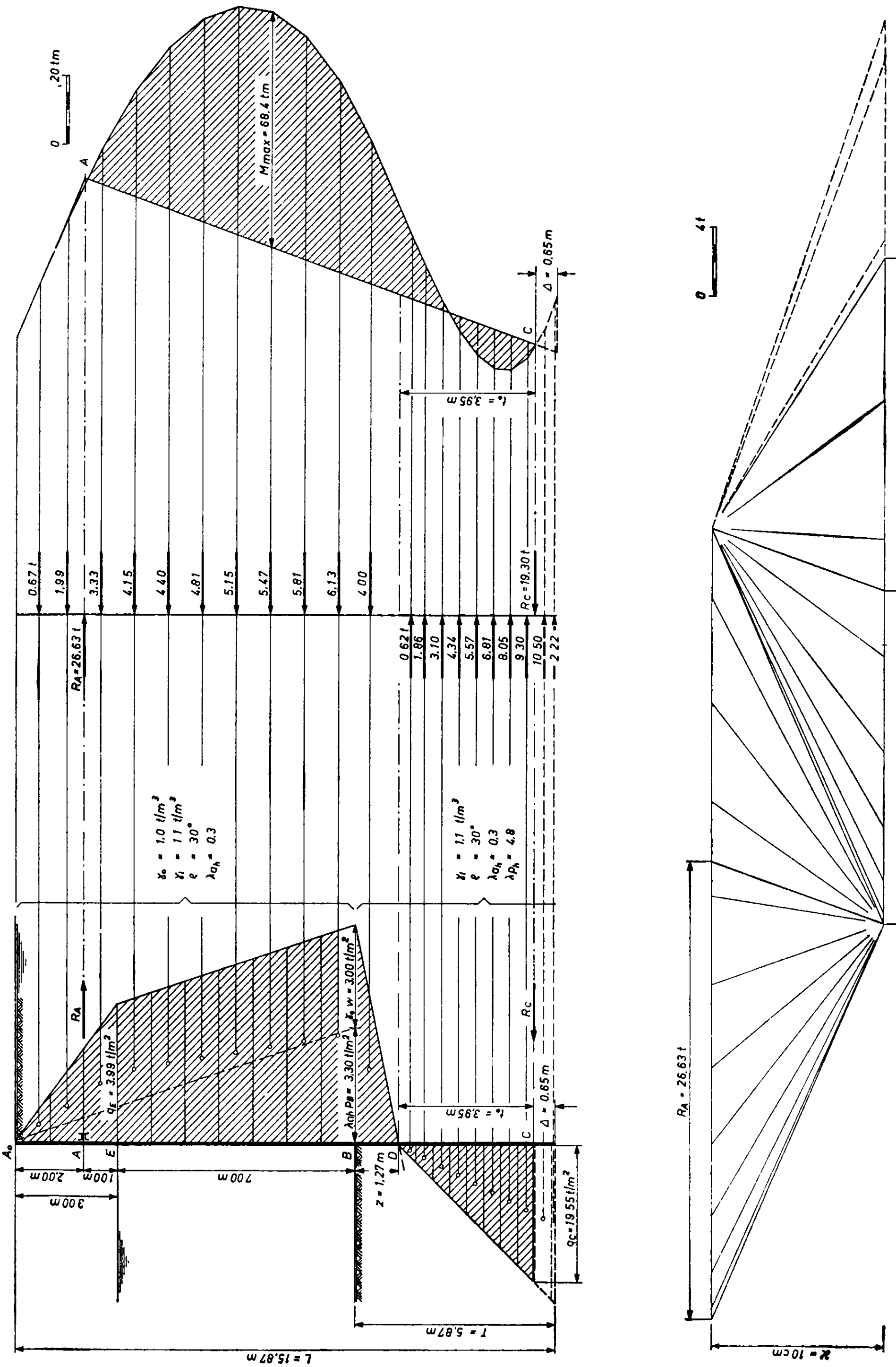
S = 4.0 t/m²
q₀ = 1.20 t/m²
q_E = 2.82 t/m²
q_B = 5.13 t/m²
q_C = 20.69 t/m²
R_A
R_C
2.00 m
1.00 m
3.00 m
7.00 m
z = 1.04 m
T = 5.87 m
L = 15.87 m
Δ = 0.65 m
t₀ = 4.18 m
M max = 43.0 tm
R_A = 20.40 t
R_C = 26.80 t
Q_Ph = 43.24 t
ϰ = 5 cm
1.47 t
2.01
2.55
2.98
3.30
3.64
3.98
4.30
4.64
4.98
2.67
0.62 t
1.86
3.10
4.34
5.57
6.81
8.05
9.30
10.50
7.50
0
20 tm
8 t

FIG. 15a



S = 2,0 t/m²
q₀ = 0,60 t/m²
q_E = 2,22 t/m²
q_B = 4,86 t/m²
q_C = 16,04 t/m²
R_A
R_C
2,00 m
1,00 m
3,00 m
8,00 m
1,00 m
z = 0,98 m
T = 5,87 m
L = 15,87 m
l₀ = 3,24 m
Δ = 0,65 m
γ_n = 1,8 t/m³
ϱ = 30°
λa_h = 0,3
γ_l = 1,1 t/m³
ϱ = 30°
λa_h = 0,3
γ_l = 1,1 t/m³
ϱ = 30°
λa_h = 0,3
λp_h = 4,8
0,87 t
1,41
1,95
2,38
2,71
3,05
3,38
3,71
4,05
4,38
4,71
2,38
R_A = 19,50 t
0,62
1,86
3,10
4,34
5,57
6,81
8,05
7,16
R_C = 10,50 t
M max = 54,5 tm
20 tm
4 t
Q_ph = 25,98 t
R_A = 19,50 t
R_C = 10,50 t
H = 10 cm

FIG. 15*b*



M max = 68.4 tm
0 20 tm
Δ = 0.65 m
l₀ = 3.95 m
0.67 t
1.99
3.33
4.15
4.40
4.81
5.15
5.47
5.81
6.13
4.00
R_C = 19.30 t
R_A = 26.63 t
0.62 t
1.86
3.10
4.34
5.57
6.81
8.05
9.30
10.50
2.22
γ₀ = 1.0 t/m³
γ₁ = 1.1 t/m³
ϱ = 30°
λa_h = 0.3
λp_h = 4.8
q_E = 3.99 t/m²
γ₀ w = 3.00 t/m²
λah p_B = 3.30 t/m²
z = 1.27 m
q_C = 19.55 t/m²
2.00 m
1.00 m
3.00 m
7.00 m
T = 5.87 m
L = 15.87 m
0 4 t
R_C = 19.30 t
Q_Ph = 38.62 t
R_A = 26.63 t
H = 10 cm

FIG. 15c

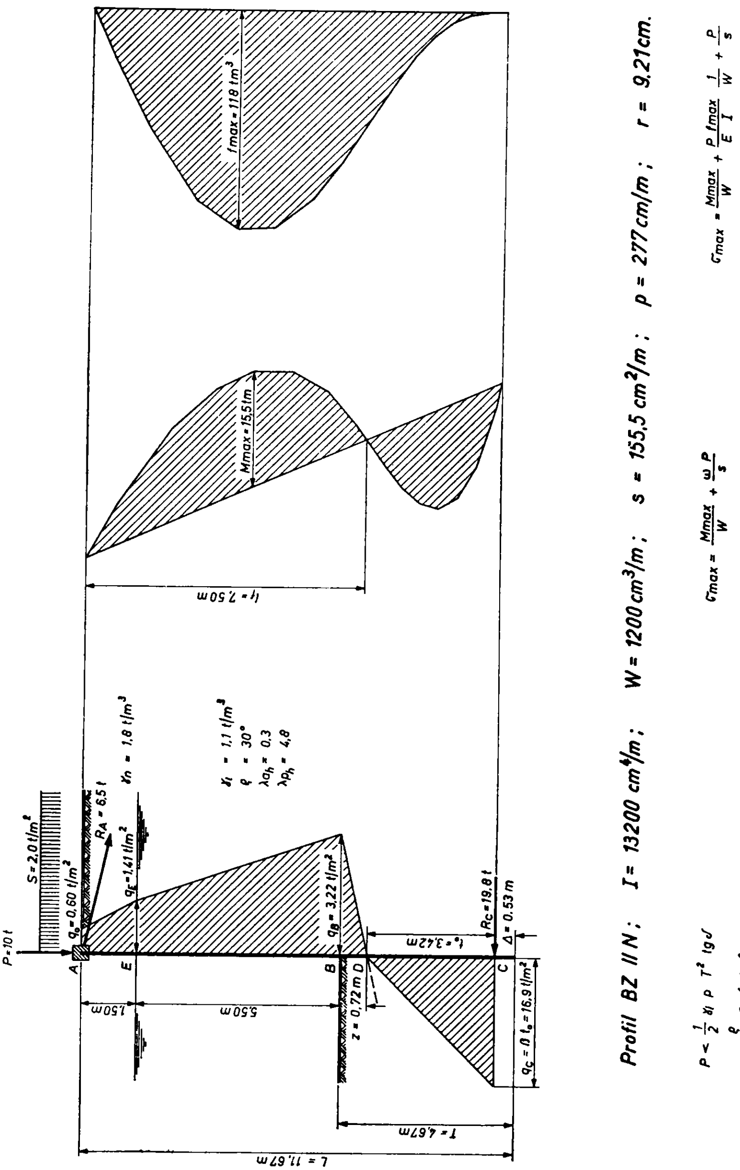

fmax = 118 tm³
Mmax = 15,51m
l₁ = 7,50 m
P = 10 t
S = 2,0 t/m²
q₀ = 0,60 t/m²
R_A = 6,5 t
q_E = 1,41 t/m²
γn = 1,8 t/m³
γi = 1,1 t/m³
ρ = 30°
λa_h = 0,3
λp_h = 4,8
q_B = 3,22 t/m²
R_C = 19,8 t
Δ = 0.53 m
l₀ = 3,42 m
z = 0,72 m
q_C = D l₀ = 16,9 t/m²
A
E
B
D
C
1,50 m
5,50 m
T = 4,67 m
L = 11,67 m
Profil BZ II N ; I = 13200 cm⁴/m ; W = 1200 cm³/m ; s = 155,5 cm²/m ; p = 277 cm/m ; r = 9,21 cm.
σmax = Mmax/W + ω P/s
σmax = Mmax/W + P fmax/E I · 1/W + P/s
P < 1/2 γi ρ T² tgδ
ρ/2 < δ < ρ

Fig. 16

The first check appears as

$$\boxed{P < \tfrac{1}{2}\gamma \cdot p \cdot T^2 \cdot \operatorname{tg}\delta}$$

where P is the lineal load to be transmitted to the ground in t/m;
γ the unit weight of the earth in t/m³;
p the perimeter of the sheet piles in m/m;
T the total penetration of the bulkhead in m;
$\operatorname{tg}\delta$ the coefficient of friction, δ being the angle of earth-wall friction.

This angle of friction δ may be assumed to be equal to about $\delta = \frac{1}{2}\rho$ in the case of loose unsettled earth and to $\delta = \frac{2}{3}\rho$ and even $\delta = \rho$ in the case of settled earth being in place for some time.

In the case of Figure 16, this inequality may be verified already for an angle δ equal to $\frac{1}{2}\rho$.

$$10 < \tfrac{1}{2} \cdot 1{,}1 \cdot 2{,}77 \cdot 4{,}67^2 \cdot 0{,}364 = 12{,}2 \text{ t.}$$

The transmission of the vertical load of 10 t/m is thus assured by the depth of penetration of fixed earth support of the bulkhead.

4.b.b Checking the steel stresses

To verify the supplementary stresses in the steel, due to the vertical load, two methods, which give approximately the same results, are available.

One of them, known as the "Omega" method, arises from principles of computing straight beams stressed by a load acting through their axis.

The other method proceeds from the supplementary stressing of the wall resulting from the eccentricity of the load P in relation to the deflected elastic line of the wall.

Omega method

We begin by calculating the ratio of slenderness λ_t of the beam which is given by the relation

$$\boxed{\lambda_t = \frac{l_t}{r}}$$

where λ_f is the ratio of slenderness of the considered beam, coefficient without dimensions;
l_f the buckling length of the beam, in cm;
r the radius of gyration of the selected section, in cm.

A certain degree of confusion prevails concerning the choice of the buckling length l_f of the bulkhead and its determination is more or less arbitrary. *We propose to consider as buckling length l_f of the bulkhead the part of the wall situated in the most highly stressed region (thus above the dredge line) included between the two points of zero moment.*

That corresponds most closely to the case of a compressed beam, loaded by a moment and articulated at its extremities of zero moment.

In the case of Figure 16 that buckling length would thus be equal to $l_f = 7{,}50$ m.

The radius of gyration of the selected section being equal to $r = 9{,}21$ cm, the ratio of slenderness of the yielded and compressed beam would thus be

$$\lambda_f = \frac{750}{9{,}21} = 81{,}5\,.$$

For that value of slenderness ratio, we find by linear interpolation in Table II below, the corresponding value of ω for the selected quality of steel.

TABLE II

λ_f	ω Steel 44/52	ω Steel 50/60	λ_f	ω Steel 44/52	ω Steel 50/60
0	1,00	1,00	80	1,76	1,85
10	1,01	1,01	90	2,21	2,39
20	1,03	1,03	100	3,07	3,55
30	1,06	1,07	110	3,72	4,29
40	1,12	1 13	120	4,43	5,11
50	1,20	1,22	130	5,20	6,00
60	1,32	1,35	140	6,03	6,95
70	1,49	1,54	150	6,92	7,98

In our case, for steel of 50/60 kg/mm², the value of ω is

$$\omega = 1{,}93.$$

This value is used to check the supplementary stress due to the vertical load by the relation

$$\boxed{\sigma_P = \frac{\omega \cdot P}{s}}$$

σ_P being the supplementary stress due to the load P in kg/cm²;
P the vertical load by running metre expressed in kg;
s the chosen steel section in cm²/m
and ω the factor obtained from Table II.

In the case of Figure 16 the supplementary stresses are equal to

$$\sigma_P = \frac{1{,}93 \cdot 10 \cdot 10^3}{155{,}5} = 124\ \text{kg/cm}^2\,.$$

Method using walls yield

In Figure 16 the elastic line of the wall shows a maximum yield of $f_{max} = 118$ tm³.

The actual value F of that yield expressed in cm is obtained by dividing the value of f_{max} expressed kgcm³ (1 tm³ = 10^9 kgcm³) by the elastic constant $E \cdot I$ of the wall

$$\boxed{F = \frac{f_{max}}{E \cdot I}}$$

E being the modulus of elasticity of the steel generally equal to $2{,}1 \cdot 10^6$ kg/cm²
and I the moment of inertia of the selected wall section expressed in cm⁴/m.

That yield of the elastic line in relation to the vertical line of action of the load P creates in the wall a supplementary moment equal to

$$\boxed{M_P = F \cdot P}$$

The supplementary stresses due to this moment are equal to

$$\boxed{\sigma_P = \frac{M_P}{W}}$$

These stresses go to increase those due to the direct action of the load P which is given by

$$\boxed{\sigma_P = \frac{P}{s}}$$

where s is the steel section of the bulkhead.

The supplementary stresses attributable to P are ultimately equal to

$$\boxed{\sigma_P = \frac{P}{s} + \frac{M_P}{W}}$$

σ_P are the supplementary stresses from the vertical load on the bulkhead expressed in kg/cm²;
P linear vertical load expressed in kg;
s steel section of the bulkhead in cm²/m;
M_P moment due to eccentricity of load P in relation to unyielded wall axis in kgcm;
W section modulus of the used sheet piles in cm³/m.

On the example given in Figure 16, the numerical calculation shows successively

$$F = \frac{f_{\max}}{E \cdot I} = \frac{118 \cdot 10^9}{2{,}1 \cdot 10^6 \cdot 13{,}2 \cdot 10^3} = 4{,}22 \text{ cm};$$

$$M_P = F \cdot P = 0{,}0422 \cdot 10 = 0{,}422 \text{ tm};$$

or

$$\frac{M_P}{W} = \frac{42{,}2 \cdot 10^3}{1{,}2 \cdot 10^3} = 35 \text{ kg/cm}^2;$$

$$\frac{P}{s} = \frac{10 \cdot 10^3}{155{,}5} = 65 \text{ kg/cm}^2;$$

$$\sigma_P = 35 + 65 = 100 \text{ kg/cm}^2.$$

It is seen that the second method, starting from the yield of the elastic line of the bulkhead gives a result slightly more favourable than that obtained from the "Omega method".

The total stress of the bulkhead is given by

$$\sigma_M = \frac{M_{max}}{W} = 1\,290 \text{ kg/cm}^2$$

owing to the maximum bending moment and the tension σ_P resulting from the vertical load.

Finally, we shall have

$$\sigma = \sigma_M + \sigma_P \,,$$

$$\sigma = 1\,290 + 124 = 1\,414 \text{ kg/cm}^2$$

or

$$1\,290 + 100 = 1\,390 \text{ kg/cm}^2.$$

It can be seen that the influence of a vertical load on the bulkhead stresses is rather insignificant. The principal check on a vertically loaded bulkhead consists in controlling the transmission of the load to the subsoil by the friction of the earth on the buried portion of the bulkhead.

CHAPTER III

THE ANCHORAGE SYSTEM

1. GENERAL

The stability of an anchored sheet pile bulkhead, driven into the ground, depends mainly on the stability of the device which has to guarantee the appearance of the anchor pull.

The wall may be supported, at its top, on a massive structure such as a right angle wall resting on piles. In that case the superstructure and the piles transfer the anchor pull to the subsoil.

The anchor pull can also be transmitted back to the retained fill, with the help of drilled in or driven batter piles that resist traction by the frictional forces that arise on their surfaces in contact with the earth.

Another anchorage system may be formed of trestles of raking piles set at some distance behind the bulkhead. These also resist the anchor pull by the frictional forces due to contact with the ground.

These systems of retention, consisting of piles, are within the design scope of drilled in or driven piles and need, more often than not, study and preliminary field investigation on their actual bearing capacity under given field conditions. The design of such structures is fairly critical and depends largely on the conception and the performance of the employed piles.

We propose to examine the classical type of anchorage which consists in transferring, by means of waling fixed to the bulkhead, and by tie-rods, the anchor pull to an anchor wall buried in the ground at a certain distance behind the bulkhead and acting by reason of the soil-resistance developed on its front face under the influence of the pull to which it is subjected.

The design itself of such a retaining system is dependent upon three distinct checks, namely:

— Determination of the required height of the anchor wall that should resist a given pull $\mu \cdot R_A$, μ being the factor of safety that it is wished to adopt;
— Computation of the stability of the whole, that is to say bulkhead and anchor wall or, in other words, the actual length of the tie-rods for necessary security;
— Selection of accessories such as walings or wales, tie-rods, plates and fixing bolts for a certain pre-determined stress.

2. THE HEIGHT OF THE ANCHOR WALL

The anchor wall (reinforced concrete slabs or pannels of sheet piles) stand the pull of the tie-rods by the soil-resistance that is developed on its front face.

Its dimensions are generally computed by a long and laborious trial and error.

To facilitate this design we give here a graphical method which uses the integral curve and the line of moments of the earth resistance, giving the solution of the problem in a few draughts.

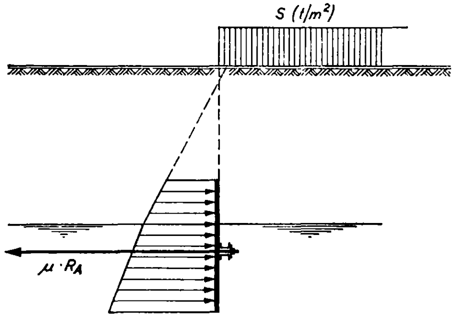


FIG. 17*a*

To obtain the height of the anchor wall we shall make the following assumptions (see Figure 17*a*):

— The anchor wall is vertical or slightly inclined. The coefficients of the earth pressures λ_{ah} and λ_{ph} are those for a vertical wall and can be read from the graph in Figure 3. The formulae in Figure 2 may serve to find the pressure coefficients in the case of a raked anchor wall;

— The anchor wall is continuous. The computation therefore is made just as for the bulkead, on the unit of length of, say 1 m;

— As an additional security the friction of earth on the anchor wall is not taken into account. That condition is by no means indispensable and in the case of rough blocks of reinforced concrete, such friction may well be taken into account so as to avoid anchorages that are too clumsy and awkward to handle;

— The surcharge is covering only the back-fill upstream of the anchor wall. The effect of this is to increase the active pressures on the back of the wall without influencing the resistance in front of it. We note however that this assumption is often negligible except in the case of already exceptional surcharges;

— The water in the back-fill maintains itself at the level of the water table in front of the bulkhead. However, it is quite possible, even if effective drainage has been foreseen in the bulkhead that, at a certain distance behind it, the back-fill may be waterlogged up to its free surface. It may therefore be assumed in practice that the soil in the neighbourhood of the anchor wall is completely immerged. This assumption in no way alters the principles of the method to be described.

METHOD FOR DETERMINING HEIGHT OF ANCHOR WALL

The preparatory work consists in establishing the soil resistance diagram for a fictitious anchor wall beginning at the free back-fill level and going down to an adequate depth. That depth can easily be increased if the computation shows it to be necessary.

In Figure 17*b*, the fictitious wall is represented by a vertical. The soil resistance diagram is easily prepared. Decrease in the soil

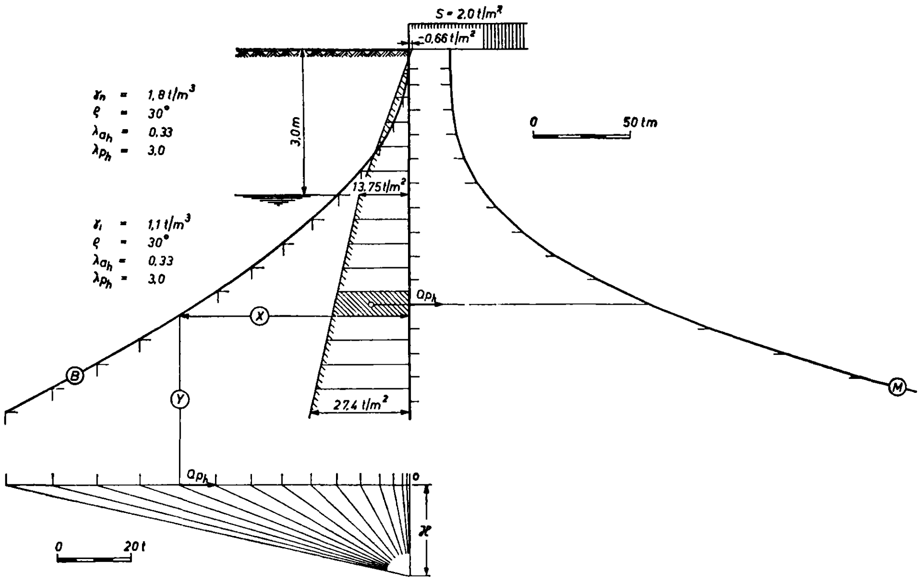


FIG. 17*b*

resistance from the effect of the surcharge behind the anchor wall is expressed as a negative pressure $-S \cdot \lambda_{ah}$ at the earth fill level. At each considered level, at (h) m below the free level, the effective soil resistance is therefore given by the expression

$$q_{ph} = p_{ph} \cdot (\lambda_{ph} - \lambda_{ah}) - S \cdot \lambda_{ah}$$

if p_h is the vertical stress at the considered level.

This soil resistance diagram is divided into horizontal trapezia represented by force vectors stemming from (o), below the vertical, in the direction of the anchor pull. This arrangement of the force vectors enables the integral curve of the soil resistance to be drawn, giving for each level the effective soil resistance available from the free back-fill level. This integral curve Ⓑ is in effect obtained by the intersection of a horizontal ⊗ drawn at the considered level (one selects the bases of the trapezia) and of a vertical Ⓨ issuing from the extremity of the force vector Q_{ph}, representative of the trapezium in question, hatched in the drawing.

The vector ⊗ comprised between the curve Ⓑ and the vertical represents, in each case, the intensity of the soil resistance between the level ⊗ and the upper surface of the back-fill.

An adequate number of points of intersection will be obtained to construct the curve Ⓑ with sufficient accuracy, if the height of the trapezia has not been overdrawn.

The force vectors of the partial soil resistances, now enable the line of moments Ⓜ to be computed after an appropriate polar distance $\mathcal{H}$ of the string polygon has been chosen. One may assume with reasonable approximation that the force vectors Q_{ph} originate at midheight of the trapezia.

This preparatory work, which at first sight appears lengthy, reduces the computation of plate dimensions to some draughts for all possible earth support conditions of the anchored wall.

The *computation of dimensions* according to the curves of Figure 17*b* is dealt with in the practical example in Figure 18.

In Chapter II 3.*b* we found an anchor pull of 15 t per running metre of bulkead. Let us select the dimensions of the anchor wall so as to resist that pull with a safety factor of 2. The anchor pull to be considered in this case will therefore be equal to

$$R_A = 2 \cdot 15 = 30 \text{ t}.$$

Let us assume further that the top of the anchor wall is 2 m beneath the free back-fill level. Generally that depth is left entirely to the estimation of the constructor.

Freely earth supported wall (Figure 18)

A horizontal ① is first drawn at a depth of 2 m which gives the points A_0' on the curves Ⓑ and Ⓜ.

On drawing a vertical ① from A_0' situated on curve Ⓑ one outlines on the string polygon the force vector (o – a) of the soil resistance that cannot affect the resistance of the anchor wall.

The line Ⓞ joining (a) to the pole of the string polygon is parallel to the tangent on Ⓜ at the level A_0'. This therefore enables the drawing of this tangent A_0' – Ⓞ to the line of moments Ⓜ.

On the string polygon of force vectors the intensity of the anchor pull, to be withstood by the wall with a safety factor of μ, (here equal to 2) is marked of; this gives the vector ($a - c_0$), in our case 30 t.

The position of (c_0) fixes the points C_0' on the curves Ⓑ and Ⓜ on drawing in turn the vertical ② and the horizontal ② which give directly the level C_0' of the toe of the anchor wall freely supported by the ground.

The line Ⓕ joining c_0 to the pole of the string polygon is parallel to the tangent on Ⓜ at the level C_0'.

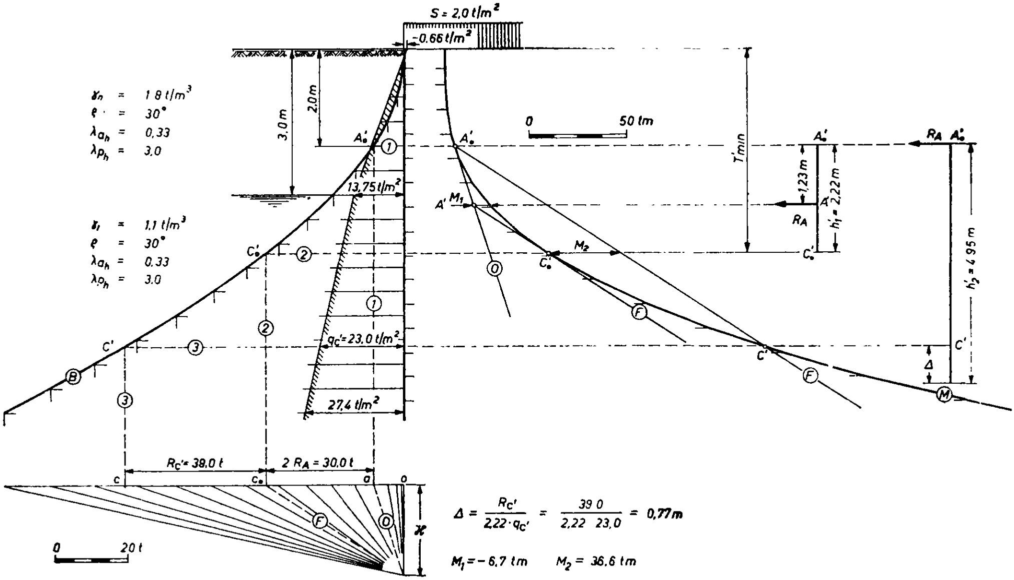
The intersection of the two tangents Ⓞ and Ⓕ gives the origin of the anchor pull and the value of the maximum bending moment (M_1) stressing the anchor wall under conditions of free earth support.

Taking the foregoing into account, the anchor wall should have a height of 2,22 m and the tie-rods should thus be placed at 1,23 m from the top of the wall which is at the centre of gravity of the soil resistance.

Fixed Anchor Wall (Figure 18)

In practice it can happen that the tie-rods cannot be placed at the centre of gravity of the soil resistance (if, for example, the fixing of the wale and the tie-rods is obstructed by water that cannot be drained off).

In that case the support of the anchor wall is no longer a



S = 2,0 t/m²
0,66 t/m²
γn = 1,8 t/m³
ρ = 30°
λah = 0,33
λph = 3,0
γ1 = 1,1 t/m³
ρ = 30°
λah = 0,33
λph = 3,0
2,0 m
3,0 m
13,75 t/m²
qc' = 23,0 t/m²
27,4 t/m²
Rc' = 39,0 t
2 RA = 30,0 t
0 50 tm
0 20 t
T'min
1,23 m
h'1 = 2,22 m
h'2 = 4,95 m
M1
M2
RA
Δ = Rc' / (2,22 · qc') = 39,0 / (2,22 · 23,0) = 0,77 m
M1 = − 6,7 tm
M2 = 36,6 tm

FIG. 18

free one as it is necessary that the eccentricity of the anchor pull should be counterbalanced by the reactions of the ground on the fixed toe of the wall.

To make the fundamental differences perfectly clear between the freely supported and the fixed anchor wall, we assume the pull $\mu \cdot R_A$, applied at the top of the wall, thus at the level A_0' chosen arbitrarily.

To compute the depth of the fixed earth support we assume, following Dr. Blum's hypothesis (see 1-7*b*) that soil resistance increases linearly up to the fixed support C of the wall where the counter-resistance is supposed to be reduced to a single force. The theoretical depth of the fixed earth support is that which cancels the moment caused by the anchor pull and the soil resistance on the front side of the anchor wall.

In Figure 18 the method of fixing this depth is stated. The tangent Ⓕ on curve Ⓜ at level C_0' is shifted parallel to itself up to its intersection with the tangent Ⓞ at the level of origin A_0' of the anchor pull.

The theoretical fixed toe is represented by the point C′ on curve Ⓜ.

The counter-resistance R_c' at the toe of the anchor wall is obtained on the string polygon by drawing the horizontal ③, then the vertical ③, by the vector $(c_0 - c)$ which is 39,0 t, difference between the total soil resistance $(a - c)$ on the wall and the anchor pull $(a - c_0)$.

The extension to be added to the anchor wall to allow the full development of the counter-resistance at C′ is given by the formula

$$\boxed{\Delta = 0{,}45 \cdot \frac{R_c'}{q_c'}}$$

where q_c' is the intensity of the soil resistance at the level C′, easily readable from the soil resistance diagram, in this case being 23,0 t/m².

The total height of the fixed wall $h'_2 = 4{,}95$ m in the case where the tie-rods originate at the level A_0'.

The maximum bending moment of the fixed anchor wall, always found at the level C_0' (penetration point of the free earth support) is in this case equal to $M_2 = 36{,}6$ tm.

Figure 19 illustrates the case of the best utilization of a fixed wall.

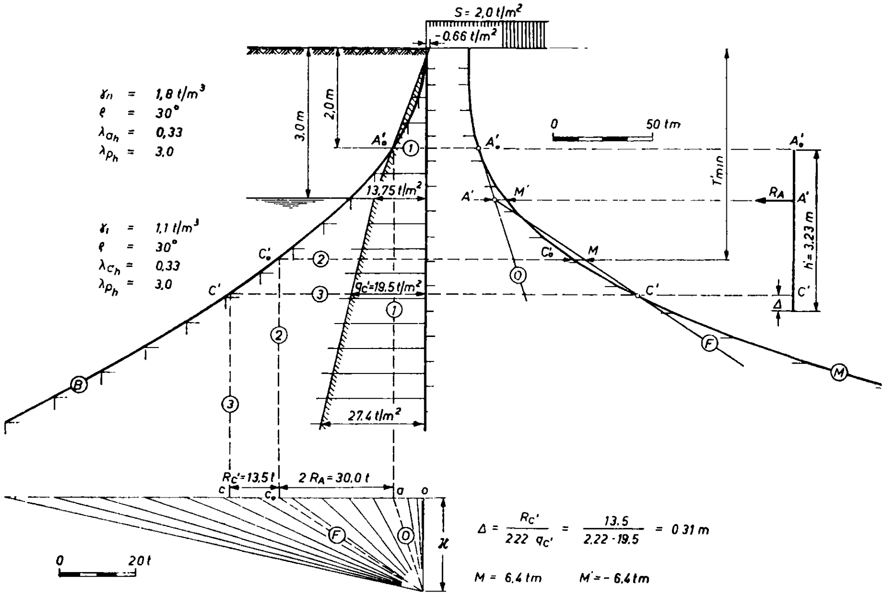


FIG. 19

The line Ⓕ should be shifted so, that the moments (M) and (M′) are equal in absolute value and minima at the same time.

The intersection of Ⓞ and Ⓕ indicates the origin A′ of the tie-rods.

The counter-resitsance at the toe C′ is reduced to $R_c' = 13{,}5$ t and the height of the wall to 3,23 m.

This method thus allows the immediate determination of the height of the anchor wall in terms of the level of origin of the anchor pull and the depth of the wall below the back-fill surface without much trial and error but with predetermined reliability.

Note

In the preceding design we have assumed that the anchor wall was continuous. But it is possible that the anchor wall may consist of isolated plates separated one from the other.

These separate plates behave like a continuous wall if the space that separates them does not exceed a certain critical value (e).

L. Descans [14] has analysed this problem and his investigations led him to conclude that this critical value (e) of the interval between the separate plates is given approximately by the formula

$$\boxed{e = \frac{2}{3} \cdot \frac{\sin \rho}{1 + \sin \rho} \cdot T'_{\text{min}}}$$

if T'_{min} is the depth of the toe C_0' of the free earth support of the anchor wall below the back-fill surface (or the level of the maximum bending moment of the fixed anchor wall, level where the shearing stress cancels itself).

The point C_0' can be regarded as the point of rotation of the fixed anchor wall (see also III.3).

Under the conditions of the example in Figure 18 this critical interval would be

$$e = \frac{2}{3} \cdot \frac{0{,}5}{1 + 0{,}5} \cdot 4{,}22 = 0{,}94 \text{ m} .$$

Thus the continuous wall may be replaced by separate plates 90 cm apart. In many cases in practice this course can lead to appreciable economies.

3. STABILITY OF BULKHEAD AND ANCHOR WALL

After having determined the dimensions of the anchor wall or plates that should resist a given pull, it remains to define the reasonable positions to be taken by the anchorage system.

In Figures 20*a* and 20*b*, two extreme cases of locating anchor walls are shown.

In the case of 20*a*, the anchor wall is situated too near to the bulkhead and it will be carried away by the general slip of the back-fill which will occur approximately on a inclined plane of $(\frac{1}{4}\pi + \frac{1}{2}\rho)$ to the horizontal, which passes through the toe C_0 of the principal wall.

That anchorage is therefore ineffective.

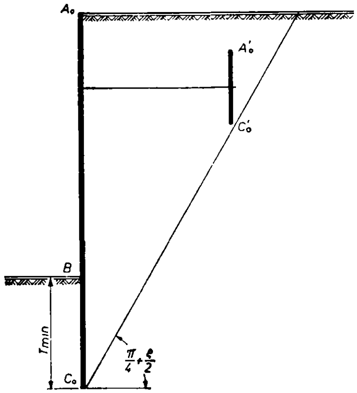


FIG. 20*a*

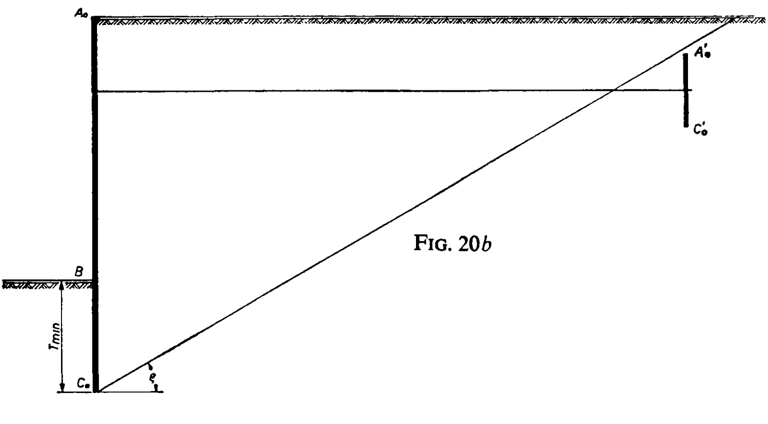


FIG. 20*b*

In the case of 20*b*, the anchor wall is buried on the far side of the natural slope plane inclined approximately under the angle of internal friction ρ to the horizontal and passing through the toe of the principal wall. These long anchorages ensure the stability of the bulkhead, but they are not at all economical in practice, owing to the excessive length of the tie-rods.

It is necessary therefore to choose a reasonable length for tie-rods in order to be both certain and economical.

DEPTH OF TOE C_0 OF WALL

A word remains to be said on the depth of the toe C_0 of the bulkhead. In the case of bulkheads with free earth support, the depth of the toe C_0 corresponds obviously with the minimum penetration (see II.3). That level is defined by the cancellation of the shearing stress.

In the case of fixed bulkheads, the level of the minimum penetration can be regarded as the level below which the counter-resistance of the soil begins to make its appearance. The lateral pressure of the back-fill can only manifest itself up to this level. We can therefore agree that the origin of the sliding surfaces is found at the depth T_{min}, as well for the fixed as for the freely supported bulkheads. Dr. Lackner [16] suggests the choice of the point C_0 at the level of the maximum bending moment on the buried part of the bulkhead, thus at the level where the shearing stress cancels itself. This brings us back to consideration of the level C_0 at the minimum depth of penetration T_{min}.

The same considerations apply to the anchor wall where the sliding surfaces have to pass through the extremity C_0' of the freely supported wall at the depth T'_{min} from the surface, even for fixed anchor walls.

A first check consists in determining the length of the tie-rods that will allow the full development of the soil-resistance on the anchor wall. This condition can be made sure by following the scheme in Figure 21.

The ground above the horizontal plane above the top A_0' of the anchor wall can be regarded as a simple ballast load, the weight of earth to the left of N presses the lower mass of earth towards the bulkhead, whilst the weight of earth to the right of N obstructs the rising of that earth mass that the anchor wall tends to rise.

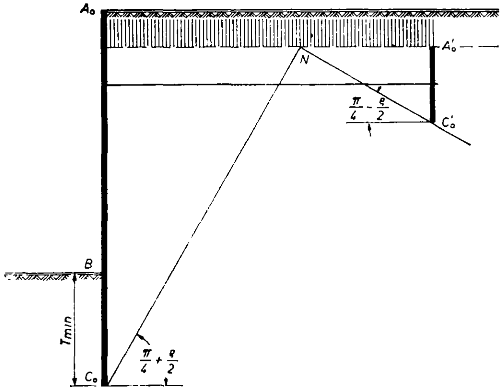


FIG. 21

The point N is in this case the intersection of the two sliding surfaces of the active and the passive wedges of pressure, inclined, always with close approximation, at $(\frac{1}{4}\pi+\frac{1}{2}\rho)$ and $(\frac{1}{4}\pi-\frac{1}{2}\rho)$ respectively, to the horizontal and issuing from the toes C_0 and C_0' of the two walls.

In this case it can be agreed that the full development of the soil resistance of the anchor wall is assured, which was tacitly supposed when computing the height of the anchor wall.

This first check gives in fact the lengths of the tie-rods strictly necessary to ensure the resistance of the soil in front of the anchor wall.

A second check is then necessarily applied. The object is to verify if the presence of the anchorage system in the back-fill of the bulkhead is not the cause of an additional pressure on this latter.

Dr. Kranz [17] has shown that the most dangerous sliding surface between bulkhead and anchor wall is that one joining the toes C_0 and C_0' of the two walls (Figure 22*a*).

This sliding surface C_0-C_0' being assumed plane, we propose to compute this last condition imposed on the anchor wall with the Culman diagram.

Using Culman's method we will verify if the mass A_0C_0X' (Fig. 22*b*) sliding on the plane surface C_0X' (initiated by the toe C_0' of the anchor wall) exerts on the principal wall A_0C_0 a pressure which, added to the anchor pull, does not exceed the total pressure on A_0C_0 in normal service.

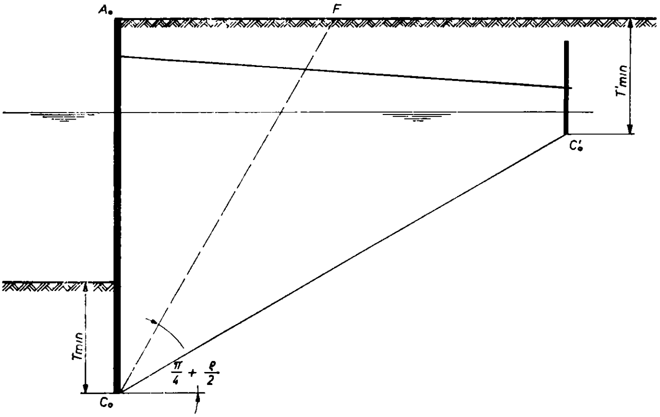


FIG. 22*a*

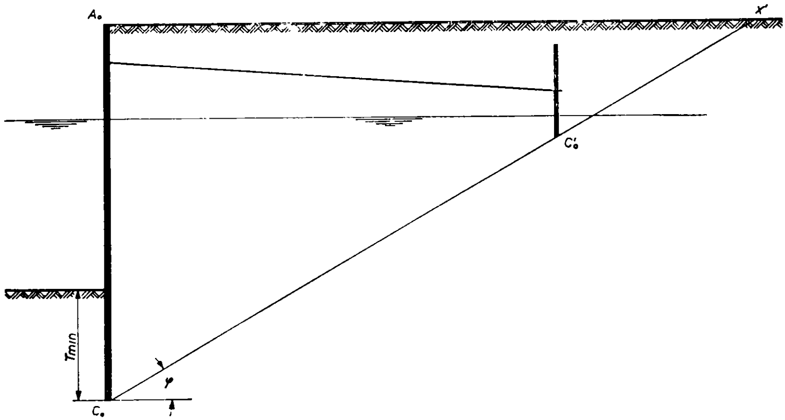


FIG. 22*b*

NOTE ON THE CULMAN DIAGRAM (Figures 23*a* and 23*b*)

a) Draw through the base C of the screen AC the line TN (so-called natural slope) making the angle of internal friction ρ with the horizontal, and the line LO (so-called line of orientation) making

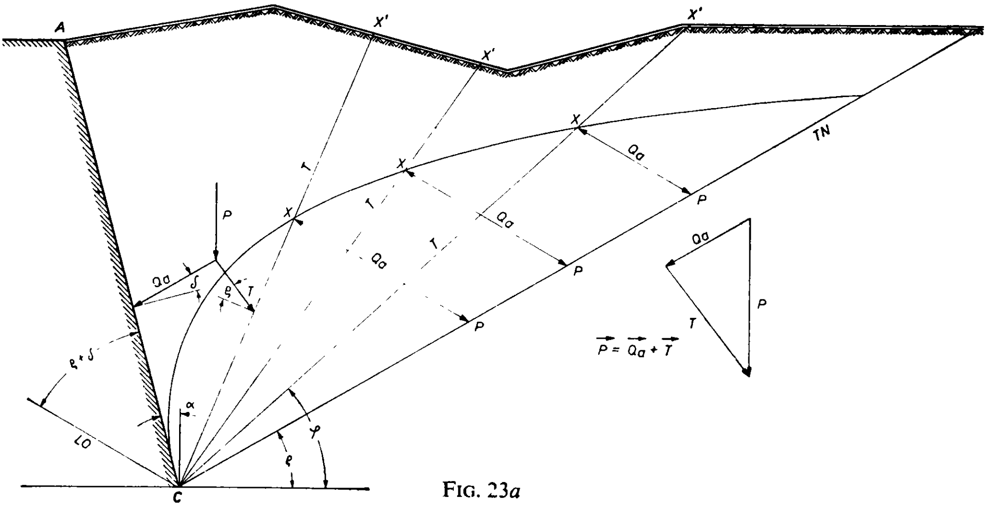


FIG. 23*a*

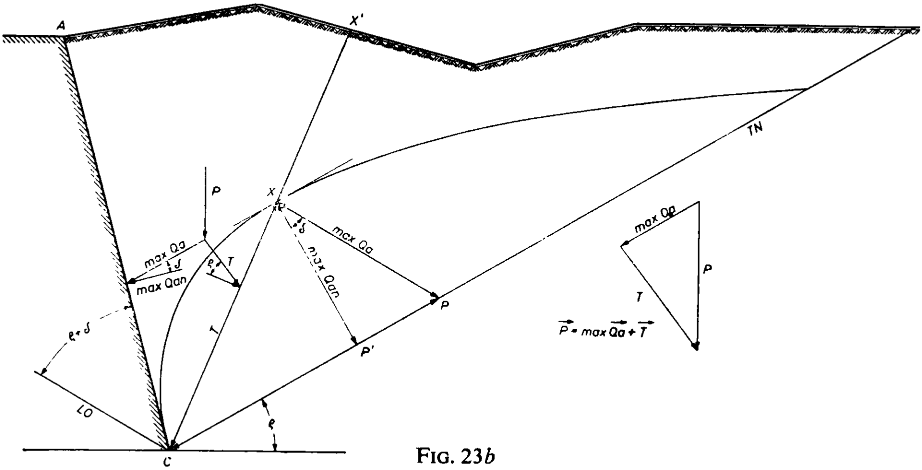


FIG. 23*b*

the angle $(\rho + \delta)$ with the direction CA of the screen, inclined at (α) to the vertical (δ being the angle of friction of earth on wall).

b) For each sliding surface CX′, chosen arbitrarily,

— determine the weight P of the prism ACX′;

— mark off the value P from C along the line TN;

— draw through the end P of this vector, a parallel to the line of orientation LO and note X, its point of intersection with CX′.

The value of the active pressure Q_a on the screen, exercised by the arbitrarily chosen prism of earth, is then given by the vector PX.

c) The same procedure is repeated for an adequate number of sliding surfaces and, through all the points X thus obtained, a curve is drawn which is representative of the successive values of the earth pressures on the screen AC.

d) The point of contact of the tangent to that curve (Fig. 23*b*) parallel to the line TN, gives the value of the maximum earth pressure max Q_a (vector PX) and the direction of the corresponding plane sliding surface CX′. The component of the pressure, normal to the screen, is given by the vector P′X, perpendicular to the line TN.

APPLICATION OF THE CULMAN DIAGRAM TO A VERTICAL BULKHEAD
(Figures 24*a* and 24*b*)

Assuming that the water table in the back-fill levels the free water in front of the bulkhead, the weight of the prisme A_0C_0X' is found as a simple function of the angle φ that the sliding surface C_0X' makes with the horizontal.

Taking account of the surcharge, the weight of the prism, A_0C_0X', is

$$P = \gamma_n \cdot a \cdot \frac{l_1 + l_2}{2} + \gamma_1 (T_{\min} + h) \frac{l_2}{2} + S \cdot l_1$$

an expression in which we have:

$$l_1 = \frac{1}{\operatorname{tg} \varphi} (T_{\min} + h + a)$$

$$l_2 = \frac{1}{\operatorname{tg} \varphi} (T_{\min} + h)$$

P is thus given by the simple relation:

$$\boxed{P = \frac{K}{\operatorname{tg} \varphi}}$$

where K is a constant, function simply of $T_{\min}$, h, a and S which are known.

On Figure 24*b* the Culman curve has been drawn.

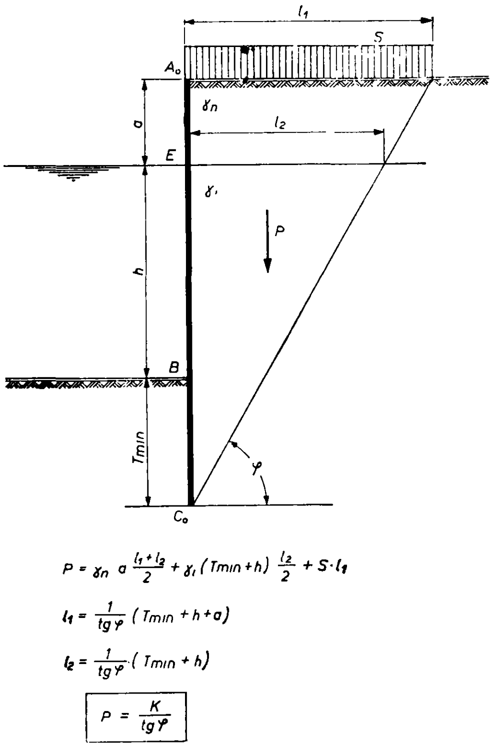


FIG. 24*a*

The toe C_0' of the anchor wall should be outside the prism of earth which, on sliding along the surface C_0C_0' would exert on the bulkhead C_0A_0 a total pressure Q_{ah} exceeding the total pressure max Q_{ah} minus the anchor pull.

From the vector max Q_{ah} (P′X) the anchor pull R_A (increased by the chosen factor of safety μ) must be deduced and at that distance a parallel to the line TN is drawn. That parallel to TN cuts the Culman curve at a point L. The vector LL′ is the horizontal pressure Q_{ah} exerted on the wall A_0C_0 by the prism A_0C_0X', sliding along C_0X'.

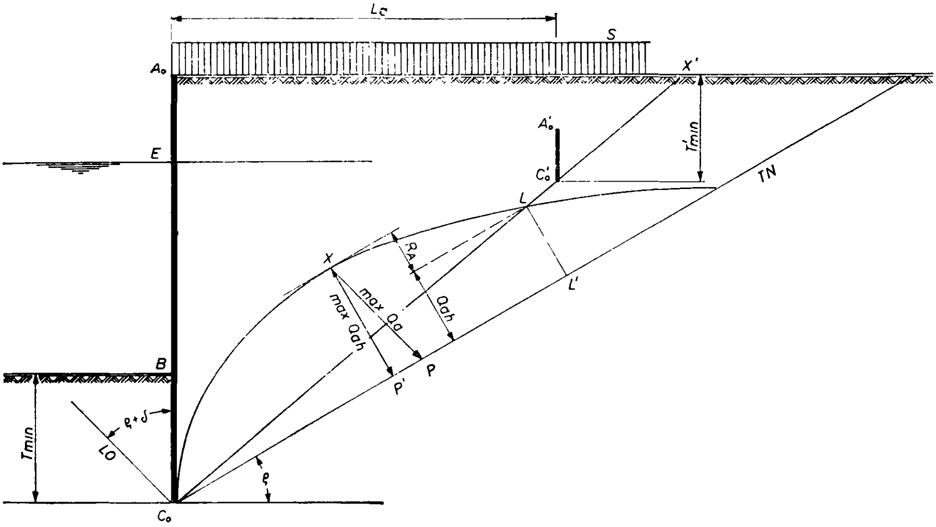


FIG. 24*b*

The toe C_0' of the anchor wall should therefore be situated outside or on this sliding surface C_0X' in order to have

$$Q_{ah} + R_a \leqslant \max Q_{ah}\,.$$

This determines the length L_a of the tie-rods.

In Figure 24*b* it is seen that, if the toe C_0' of the anchor wall is found behind the plane of sliding surface inclined at ρ to the horizontal, it will exert no pressure on the bulkhead, since Q_{ah} is cancelled.

It may be said then that maximum security is attained with "long anchorages" situated behind this so-called natural slope surface. The factor of safety in this case is

$$\mu = \frac{\max Q_{ah}}{R_A}$$

It remains constant for all "long anchorages".

PRACTICAL EXAMPLE

We propose to determine for the anchored and fixed bulkhead of Chapter II.3 the length of the anchorage system.

In the first place this involves determination of the depth of the origin C_0 of the sliding surfaces corresponding to the minimum depth of penetration T_{min}.

The determination of the minimum penetration of the bulkhead is facilitated by the graph Figure 11 when the reaction R_D of the "equivalent beam" of Dr. Blum, discussed in chapter II-3.*b.a* and in Figure 13, is known.

With this value of R_D the dimensionless constant

$$r = \frac{6 \cdot R_D}{\beta \cdot H^2} \text{ is calculated}$$

where R_D is the reaction of support D of the "equivalent beam" expressed in t/m;

H the span (AD) of the "equivalent beam" in m;

$\beta = \gamma_1 (\lambda_{ph} - \lambda_{ah})$ the residual coefficient of the soil-resistance in t/m³.

In Figure 13 we find $R_D = 14{,}93$ t/m for $H = 8{,}92$ m and $\beta = 4{,}95$ t/m³. These values give the constant:

$$r = \frac{6 \cdot 14{,}93}{4{,}95 \cdot 8{,}92^2} = 0{,}228 .$$

In the graph (Fig. 11), for this value of r we find $\vartheta = 0{,}26$.

Thus we obtain the minimum depth of penetration t_{min} below level D

$$t_{min} = \vartheta \cdot H = 0{,}26 \cdot 8{,}92 = 2{,}32 \text{ m}.$$

The level of C_0 below level B of the dredge line

$$T_{min} = t_{min} + z = 2{,}32 + 0{,}92 = 3{,}24 \text{ m}.$$

z represents the depth of the point D of zero pressure below B.

To calculate the weights of the prisms of earth in terms of angle φ one determines, in turn:

$$l_1 = \frac{1}{\operatorname{tg}\varphi}(T_{min} + h + a) = \frac{1}{\operatorname{tg}\varphi}\cdot(3{,}24 + 7 + 3) = \frac{13{,}24}{\operatorname{tg}\varphi}\text{ m};$$

$$l_2 = \frac{1}{\operatorname{tg}\varphi}(T_{min} + h) = \frac{1}{\operatorname{tg}\varphi}\cdot(3{,}24 + 7) = \frac{10{,}24}{\operatorname{tg}\varphi}\text{ m};$$

$$\frac{l_1 + l_2}{2} = \frac{11{,}74}{\operatorname{tg}\varphi} \qquad \frac{l_2}{2} = \frac{5{,}12}{\operatorname{tg}\varphi}\text{ m};$$

$$P = \gamma_n \cdot a\frac{l_1 + l_2}{2} + \gamma_1(T_{min} + h)\frac{l_2}{2} + S\cdot l_1$$

$$= 1{,}8\cdot 3\,\frac{11{,}74}{\operatorname{tg}\varphi} + 1{,}1\cdot 10{,}24\,\frac{5{,}12}{\operatorname{tg}\varphi} + 2\cdot\frac{13{,}24}{\operatorname{tg}\varphi}$$

$$= \frac{63{,}4}{\operatorname{tg}\varphi} + \frac{57{,}5}{\operatorname{tg}\varphi} + \frac{26{,}5}{\operatorname{tg}\varphi}$$

$$\boxed{P = \frac{147{,}4}{\operatorname{tg}\varphi}}$$

With these values of P, determined for a serie of sliding surfaces at variable inclination φ, the Culman curve is drawn. The horizontal component of the maximum active pressure on A_0C_0 is equal to

$$\max Q_{ah} = 43 \text{ t}.$$

Allowing the anchor pull $R_A = 14{,}88$ t or $R_A = 15$ t with a safety factor of $\mu = 1.5$, we shall have:

$$\mu \cdot R_A = 1{,}5 \cdot 15 = 22{,}5 \text{ t},$$

which value we deduct from max Q_{ah}.

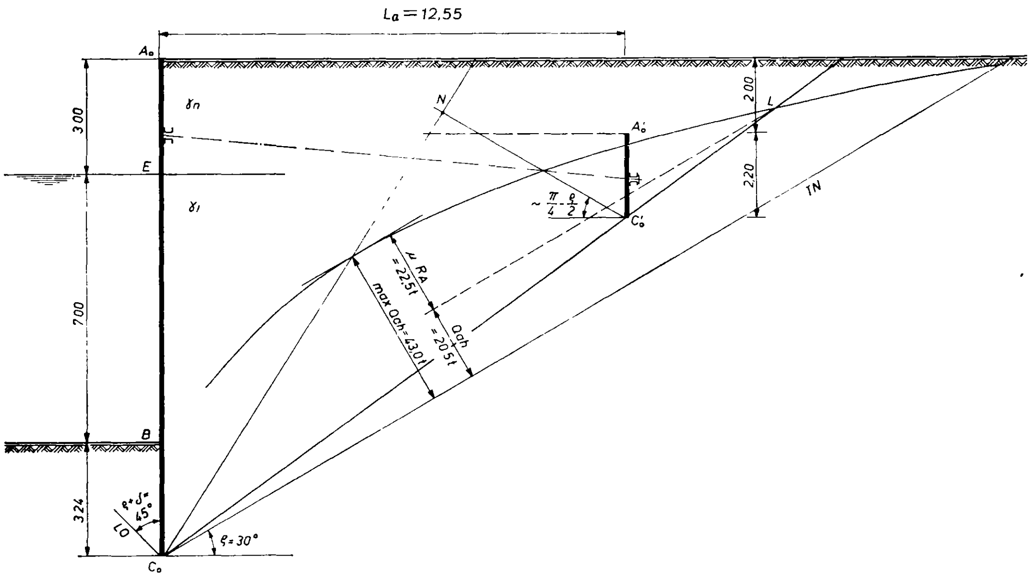


FIG. 25

A parallel to TN, drawn at the distance (max $Q_{ah} - \mu \cdot R_A$) from TN, defines on the Culman curve the point L which is the second point on the sliding surface C_0L which satisfies the imposed condition to exert on the bulkhead A_0C_0 a horizontal pressure $Q_{ah} = 20{,}5$ t which, added to $\mu \cdot R_A = 22{,}5$ t, equals the maximum lateral pressure max $Q_{ah} = 43$ t.

A horizontal at the depth $T'_{min} = 4{,}20$ m determines the site of the anchor wall $A_0'C_0'$.

For a safety factor of 1.5, the length of the tie-rods should therefore be equal to 12,55 m.

The maximum safety factor that can be attained with the so-called "long anchorages" amounts to

$$\mu = \frac{\max Q_{ah}}{R_A} = \frac{43}{15} = 2,86$$

FINAL REMARKS

It remains to observe that earth-wall angle of friction δ adopted in this method should correspond to the angle adopted in the design covering the bulkhead. The coefficients $\lambda_{ah} = 0,3$ and $\lambda_{ph} = 4,8$ used in the example sub II-3.*b* correspond to a value of $\delta = \pm \frac{1}{2} \rho$ as can be verified in the graph of Figure 3.

If therefore we neglect the earth-wall friction in the bulkhead design covering the bulkhead. The coefficients $\lambda_{ah} = 0,3$ and $\lambda_{ph} = 4,8$ lengths, for in neglecting this friction, the active pressure on the bulkhead increases and causes unavoidably a higher anchor pull than that found when the friction is taken into account.

In Figure 25, it is seen that the sliding surface of the passive wedge of earth pressure starting from C_0' of the anchor wall and inclined at $(\frac{1}{4} \pi - \frac{1}{2} \rho) = 30°$ to the horizontal (we have neglected the earth-wall friction) cuts the sliding surface of the active wedge of pressure at a level N situated above the top A_0' of the anchor wall. The first condition previously expressed is therefore satisfied: the full development of the soil resistance on the anchor wall is assured.

This method can very well serve to verify the stability of the bulkhead and anchor wall in all cases of exceptional loading (see II-4.*a*) and in fixing the safety factor. One has only to determine for each case the toe C_0 of the bulkhead, origin of the sliding surfaces, by making use of the graph of Figure 11. The reaction R_D is always that of the "equivalent beam" A_0D, assumed to be supported at A and at D.

4. SELECTION OF ACCESSORIES

The choice of the accessories for the anchorage system is made with simple assumptions currently used for structural steel work.

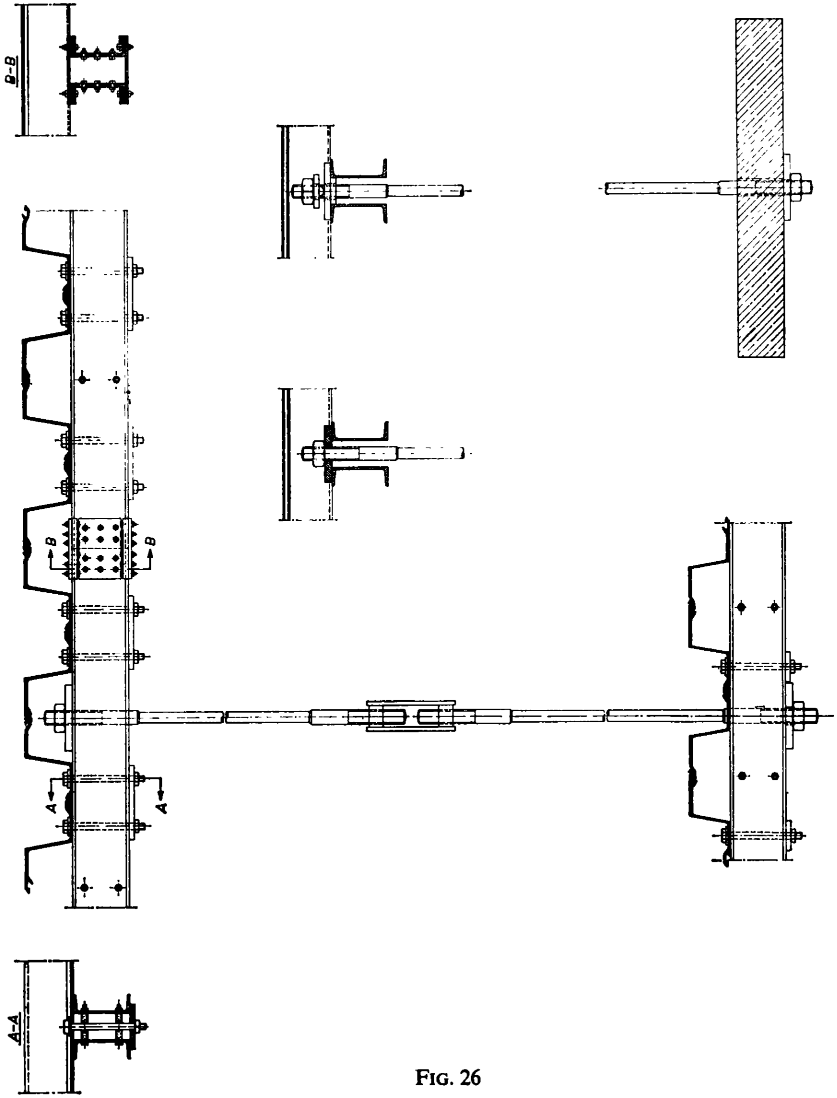


FIG. 26

The dimensions of those fittings can vary widely in terms of the chosen quality of steel, the conditions and the possibilities of supply of the material. We confine ourselves therefore to enumerate several general rules concerning the design of various pieces. Figure 26 shows a range of such anchorage accessories.

4.a *WALE OR WALING*

The wale serves to transmit the anchor pull R_A to the tie-rods which carry that pull to the anchor wall. It consists generally of two spaced channel sections set back to back.

The wale can be computed as a continuous beam with several supports (origins of the tie-rods) and bearing a linear load R_A.

The bending stress on this beam can be estimated by the method of the three moments, for example, or can be found in appropriate tables.

One must not forget however that the "supports" of the wale (the tie-rods) are more or less elastic. Because of that the application of the three moments method is only an approximation. An "exact" computation would have to take into account the elasticity of the tie-rods and the rigidity of the wale. Such a computation is exceedingly complicated and lengthy.

Further, when fixed to the bulkhead, the wale serves to straighten its axis, giving it its desired alignment. This straightening operation causes some additional stress on the wale, which it is difficult, if not impossible, to value. That stress superposes itself on that computed from normal loading and thus alters the results of the computations.

That is why, in practice, it is satisfactory to assess a certain stress on the wale and finally to choose a strong wale proportional to the section of the sheet piles.

One may assess the maximum bending moment of the wale as:

$$\boxed{M_{max} = \frac{1}{10} \cdot R_A \cdot d^2}$$

if R_A is the linear anchor pull in t/m and d the distance c/c in m between tie-rods.

The section modulus of the wale should be above

$$W_x = \frac{M_{max}}{\sigma}$$

if σ is the permissible stress on the steel, which should not exceed 80 % of its normal allowable stress.

A heavy and relatively low stressed wale is preferable to a lighter wale in a higher steel quality.

There are many engineering handbooks and suppliers catalogues that provide a choice of channels for the wale suitable in every case.

The space between the channels may be ensured either by distance pieces, in the form of tubes fitted by bolts, or by upset parts provided for on the fixing plates of the wale.

4.b *FITTING WALE TO WALL*

The wale is fixed to the sheet piling by means of fixing plates and bolts. Each bolt transmits a pull proportional to the width l of a single sheet pile and equal to

$$R_b = R_A \cdot l \cdot \mu$$

R_b being pull in t per bolt;

R_A anchor pull in t/m;

l the width of a single sheet pile, varying from 0,42 to 0,50 m for the BZ sections;

μ a safety factor introduced to cover additional stresses on the bolts due to straightening of the wall. This factor μ can be chosen between 1.2 and 1.5.

In the case of U-shaped sheet piles, the width l is that of two single sheet piles, for in that case the wale is fixed only to every second single element.

It is recommended that for the bolts the allowable stress should be below 1 000 kg/cm^2 (normal quality).

The fixing plates can be computed as beams supported at two points (longitudinal webs of the wale) and bearing a single load R_b in the centre.

The width and thickness of the fixing plates are deduced from the stress on this beam.

The fixing plates shall be computed for a permissible stress not exceeding the allowed bending stress for the employed steel.

4.c *TIE-RODS*

These consist generally of steel bars threaded at both ends. They are made in two or three pieces (depending on their length) which are joined one to the other by single couplings or by standard opposed thread turnbuckles. Their attachment to the wale is effected by means of nuts and bearing plates.

The pull on a tie-rod can be assessed as

$$\boxed{R_T = \frac{R_A \cdot d}{\cos\alpha} \cdot \mu}$$

if R_T is the pull in t per rod;
R_A the anchor pull in t/m;
d distance c/c between rods in m;
α inclinaison of tie-rod in relation to the horizontal

and μ a safety factor taking into account a certain prestressing of the tie-rods due to erection or an abnormal increase of the anchor pull. That factor should be taken as equal, at least, to 1.1 and various authors allow 1.2 as the average value.

The inclined tie-rods need special beveled washers to be inserted between the nuts and the bearing plates. These special washers can be avoided by the use of articulated or swivelling plates.

The articulated plates also allow a slight inclination of the horizontal tie-rods, which eliminates the large bending moments at the origins of the threads which may arise from the vertical yield of the tie-rods due to an eventual settlement of the back-fill. Certain cases of failure of bulkheads are attributable to the breaking of the threads under the influence of these unforeseen bending moments.

The permissible stresses in the tie-rods may be fixed

— at 75 % of the allowable stress in the area at the root of the thread;

— at 100 % of the allowable stress in the full section of the bar.

To obtain a better use of the steel of the tie-rods and, at the same time, to reduce their weight, reinforced ends can be provided for the tie-rods of which the diameter exceeds 2½″. They can be prepared by upsetting or by butt welding on threaded bars of larger diameter. A suitable section for the upset ends would be chosen in order that the area at the root of the thread might be 100/75 = 1.33 times the section of the full bar.

The bearing plates of the tie-rods may be computed as beams supported at two points but, instead of allowing one single load R_T, that load may be distributed over the height of the nut.

4.d *JOINTING WALE*

The bolted joints, known as fish plates or splices, assure continuity of the wale.

One may splice the two channels of the wale independently one of the other and stagger the joints.

But, from the practical point of view, it is preferable to splice the two members at one and the same time and to place the joints at a recess in the double piling elements where the attachment of the wale to the sheet piles does not interfere with them.

The transmission of some bending moment should be allowed by the splicing, conception of which varies to some extent and makes it difficult to propose a standardization.

APPENDIX

ASSUMPTIONS AND CALCULATIONS CONCERNING VARIOUS DIAGRAMS

1. DIAGRAM OF FIGURE 10

PENETRATION AND MAXIMUM BENDING MOMENT OF THE CANTILEVER WALL (Figure 27)

The cantilever wall is regarded as a beam A_0C fixed at C where the counter-resistance R_c acts as a single force in accord with the hypothesis of Dr. Blum.

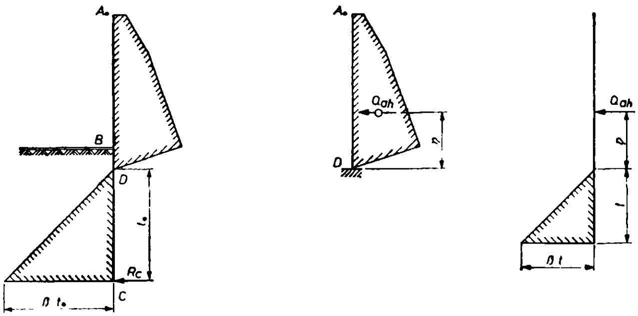


FIG. 27

The depth of the fixed earth support of the cantilever is that which cancels out the moments resulting from the active and the passive pressures.

The part of the cantilever charged with active pressures can in its turn be regarded as the beam A_0D fixed at D. The diagram of active pressures can be replaced by the force vector Q_{ah} of the lateral pressures, acting at the distance η from the point D.

Q_{ah} and η are easily obtained by decomposing the diagram of active pressure.

The equation can be written in this form:

$$M_c = Q_{ah}(t_0 + \eta) - \beta \cdot t_0 \cdot \frac{t_0}{2} \cdot \frac{t_0}{3} = 0$$

or

$$\frac{6\,Q_{ah}}{\beta} \cdot \eta \left(1 + \frac{t_0}{\eta}\right) - \eta^3 \cdot \left(\frac{t_0}{\eta}\right)^3 = 0\,;$$

in substituting

$$\frac{t_0}{\eta} = \vartheta$$

we obtain finally the simple relation

$$\boxed{\frac{6 \cdot Q_{ah}}{\beta \cdot \eta^2} = \frac{\vartheta^3}{1+\vartheta}}$$

On substituting

$$r = \frac{6 \cdot Q_{ah}}{\beta \cdot \eta^2}$$

the curve relating r to ϑ can be drawn

$$\boxed{r = \frac{\vartheta^3}{1+\vartheta}}$$

In Figure 27, the bending moment, stressing the cantilever below the level D of zero pressures, is obtained from the relation

$$M_t = Q_{ah}(\eta + t) - \beta \cdot t \cdot \frac{t}{2} \cdot \frac{t}{3}.$$

This moment becomes maximum when the shearing effort is cancelled. That effort is given by the derivative of M_t in relation to t

$$\frac{d(M_t)}{dt} = Q_{ah} - \frac{\beta \cdot t^2}{2} = 0,$$

from which we obtain

$$t^2 = \frac{2 \cdot Q_{ah}}{\beta}.$$

By substituting that value of t in the equation of moment, we obtain

$$M_{max} = Q_{ah} \cdot \eta \left(1 + \frac{2}{3} \frac{t}{\eta}\right)$$

$$= Q_{ah} \cdot \eta \left(1 + \frac{2}{3}\sqrt{\frac{2\,Q_{ah}}{\beta \cdot \eta^2}}\right).$$

However, from equation (1) above

$$\frac{2\,Q_{ah}}{\beta \cdot \eta^2} = \frac{1}{3} \frac{\vartheta^3}{1+\vartheta}.$$

By replacement of this value below the radical, we obtain finally:

$$M_{max} = Q_{ah} \cdot \eta \left(1 + \sqrt{\frac{4 \cdot \vartheta^3}{27\,(1+\vartheta)}}\right)$$

or again

$$\boxed{M_{max} = M_D \cdot \mu}$$

in substituting

$$\boxed{\mu = 1 + \sqrt{\frac{4}{27} \cdot \frac{\vartheta^3}{1+\vartheta}}}$$

which is easy to convert into the form of a curve.

2. DIAGRAM OF FIGURE 11

MINIMUM PENETRATION OF THE ANCHORED BULKHEAD (Figure 28)

The condition controlling minimum depth of penetration of the anchored bulkhead is that the wall should be simply supported in the earth. The triangular diagram of soil resistance below the point D of zero pressure is replaced by its force vector Q_{ph} applied to the lower third of the diagram.

An equation of the moment about A gives:

$$M_A \equiv Q_{ah}\,(H - \eta) - Q_{ph}\,(H + 2/3 \cdot t_{min}) = 0.$$

However, on regarding the part A_0D of the bulkhead as a beam supported at A and D, the relation may be written

$$Q_{ah}(H-\eta) = R_D \cdot H$$

if R_D is the reaction of support D of that beam and

$$Q_{ph} = \beta \cdot t_{min} \cdot \frac{t_{min}}{2}.$$

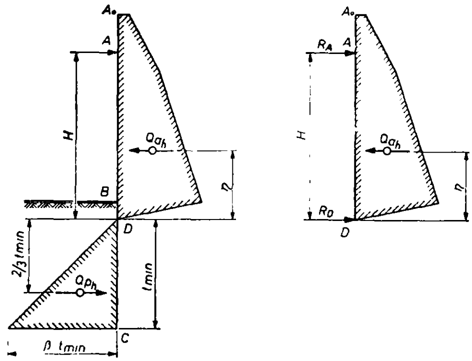


FIG. 28

On replacing these values in the initial equation we obtain

$$R_D \cdot H - \beta \cdot t_{min} \cdot \frac{t_{min}}{2}(H + \tfrac{2}{3} \cdot t_{min}) = 0$$

$$R_D \cdot H = \beta \cdot \frac{t^2_{min}}{2} \cdot H + \beta \cdot \frac{t^3_{min}}{3}$$

A slight change transforms that equation to:

$$\frac{6R_D}{\beta} = 3 \cdot t^2_{min} + 2 \cdot \frac{t^3_{min}}{H}$$

or again
$$\frac{6R_D}{\beta \cdot H^2} = 3 \cdot \left(\frac{t_{min}}{H}\right)^2 + 2\left(\frac{t_{min}}{H}\right)^3.$$

On adopting that

$$r = \frac{6R_D}{\beta \cdot H^2} \quad \text{and} \quad \frac{t_{min}}{H} = \vartheta$$

the final relation is obtained as

$$\boxed{r = 3\,\vartheta^2 + 2\,\vartheta^3 = \vartheta^2(3 + 2\,\vartheta)}$$

enabling the curve of Figure 11 to be drawn.

INDEX
OF NOTATIONS EMPLOYED

A_0		Free level of back-fill and top of bulkhead
A_0'		Top of anchor wall
A		Origin of anchor pull on the bulkhead
A′		Origin of anchor pull on the anchor wall
a	(m)	Freeboard of bulkhead above free water level
α	(°)	— Slope of wall — Slope of tie-rods
B		Dredge line level in front of the bulkhead
β	(°)	Slope of back-fill retained by the bulkhead
β	(t/m^3)	$\gamma(\lambda_{ph} - \lambda_{ah})$ residual coefficient of horizontal soil resistance
C_0		Toe of minimum driving depth of the bulkhead
C_0'		Toe of anchor wall supported freely in the soil
C		Theoretical toe of fixed earth support of the bulkhead
C′		Theoretical toe of anchor wall with fixed earth support
c	(t/m^2)	Cohesion of soil
D		Level where pressure diagram of bulkhead is cancelled
d	(m)	Distance c/c of tie-rods
δ	(°)	Angel of earth friction on wall
Δ	(m)	Extension of theoretical penetration of fixed earth support allowing the full development of counterresistance of the soil at foot of wall

ΔC	(m)	Displacement of toe C of bulkhead having insufficient penetration
Δf_A	(tm^3)	"Error" of the elastic line of the bulkhead at level A
ΔM_C	(tm)	Correction of moment to apply at level C following Hedde's correction
E_0		Water table in the back-fill
E		Free water level in front of the bulkhead
E	(kg/cm^2)	Modulus of elasticity of steel $= 2{,}1 \cdot 10^6$
η	(m)	Distance of the origin of the force vector Q_{ah} of active pressures from level D
F	(cm)	Actual yield of the elastic line
f_{max}	(tm^3)	Maximum yield of the elastic line
f_A	(tm^3)	Maximum yield of cantilever wall
Γ	(t/m^3)	Actual specific gravity of solid particles composing soil
γ_s	(t/m^3)	Unit weight of soil in dry state
γ_n	(t/m^3)	Unit weight of soil in natural damp state
γ_i	(t/m^3)	Immerged unit weight of soil
γ_0	(t/m^3)	Unit weight of water for practical purposes equal to 1,0
γ_C	(°)	Angular deformation of bulkhead at its toe C
H	(m)	Span AD of the "equivalent" beam
$\mathcal{H}$	(cm)	Polar distance in the string polygons of force vectors
h	(m)	Depth of considered level below free back-fill level
h	(m)	Depth of free water in front of the bulkhead
h'	(m)	Total height of the anchor wall
I	(cm^4/m)	Moment of inertia of the sheet piles
K	(t/m)	Constant of vertical bulkhead for the plotting of the Culman curve
L	(m)	Total length of sheet piles
L_a	(m)	Distance between bulkhead and anchor walls
l	(m)	Width of a single sheet pile

l_f	(m)	Bukling length of a bulkhead
l_1, l_2	(m)	Various lengths indicated in the text
λ_0	(1)	Coefficient of earth pressure at rest
λ_a	(1)	Coefficient of active earth pressure
λ_{ah}	(1)	Coefficient of horizontal active earth pressure
λ_p	(1)	Coefficient of passive earth pressure or soil resistance
λ_{ph}	(1)	Coefficient of horizontal passive earth pressure or soil resistance
λ_e	(1)	Coefficient of hydrostatic pressure
λ_f	(1)	Slenderness ratio of bulkhead
M_D	(tm)	Cantilever moment of the pressures acting above D
M_{max}	(tm)	Maximum bending moment of bulkhead
μ	(1)	Multiplicator giving as from M_D the maximum bending moment of cantilever walls
μ	(1)	General factor of safety
P	(t/m)	Vertical load stressing bulkhead per unit of width
p	(cm)	Perimeter of sheet piles per running metre
φ	(°)	Slope of sliding surface
p_h; $p_{t'}$	(t/m^2)	Vertical stress on horizontal facet at depths h and t' respectively
p_1; p_B	(t/m^2)	Vertical stress on horizontal facet at levels ① and Ⓑ respectively
Q_a	(t)	Total lateral pressure on bulkhead or force vector of active pressures on unit of width
Q_{ah}	(t)	Horizontal component of Q_a
$\max Q_{ah}$	(t)	Horizontal component of maximum total pressure over whole height A_0C_0 of freely supported bulkhead
$\max Q_{an}$	(t)	Normal component of maximum total pressure on inclined screen AC
Q_p	(t)	Total soil resistance or force vector of passive earth pressures on unit of width of bulkhead

Q_{ph}	(t)	Horizontal component of Q_p
q_{ah}; q_{ph}	(t/m^2)	Horizontal pressures, active and passive respectively, on an inclined or vertical facet
q_1; q_B	(t/m^2)	Horizontal pressures on inclined or vertical facet at level Ⓘ and Ⓑ respectively
R_A	(t)	Anchor pull per unit of width
R_{Ah}; R_{Av}	(t)	Horizontal and vertical components of R_A
R_c	(t)	Total counter-resistance at toe of fixed bulkhead
R_D	(t)	Reaction at support D of the "equivalent beam" of Dr. Blum
R_T	(t)	Pull per tie-rod
R_b	(t)	Pull per fixing bolt
r	(cm)	Radius of gyration of sheet piles per running metre
r	(1)	Characteristic constant of bulkhead stressed by active pressures
ρ	(°)	Angle of internal friction of soil
S	(t/m^2)	Surcharge of back-fill
s	(cm^2)	Section of steel of sheet piles per running metre
σ	(kg/cm^2)	Working stress of steel
T	(m)	Total penetration of fixed bulkhead
T_{min}	(m)	Total penetration of the free earth support of the bulkhead
T'_{min}	(m)	Depth, below back-fill level, of the toe C_0' of the freely supported anchor wall
t	(m)	Depth of considered level below D
t'	(m)	Depth of considered level below B
t_{min}	(m)	Minimum penetration below D for free earth support
t_0'	(m)	Approximate penetration below D of fixed earth support
t_0	(m)	Theoretical penetration below D of fixed earth support
tg δ	(1)	Coefficient of earth friction on a bulkhead

ϑ	(1)	t_0/η ; ratio of penetration of cantilever wall below D to the distance of the force vector of the active pressures Q_{ah} above level D
ϑ	(1)	t_{min}/H ; ratio of minimum depth of penetration below D of anchored bulkheads and the span of the equivalent beam
v	(%)	Volume of voids in comparison with unit volume of soil
W	(cm^3/m)	Section modulus of sheet piles per running metre
W_x	(cm^3)	Section modulus of wale
w	(%)	Natural moisture content of soils expressed as % of γ_s
W	(m)	Hydraulic head of unbalanced water pressure
z	(m)	Depth of point D below dredge line B

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