R. H. MANN.

The Association
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FOUNDATIONS.

By J. McHARDY YOUNG,
B.Sc., M.I.Struct.E., A.M.I.C.E.

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ERRATUM

Will readers please note the following errata in the A.E.S.D. technical pamphlet *Foundations* by J. McHardy Young, B.Sc., M.I.Struct.E., A.M.I.C.E.

"passive pressure $0{\cdot}5 \times wh \dfrac{1 + \sin \theta}{1 - \sin \theta}$"

should read :—

"passive pressure $1{\cdot}5 \times wh \dfrac{1 + \sin \theta}{1 - \sin \theta}$"

and "0·5" in the footnote on the same page should read "1·5."

On page 6, line 12, the words "fine sand" and "silt" should be transposed.

FOUNDATIONS.

J. McHardy Young,
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INTRODUCTION.

The object in designing and constructing foundations is to spread the load from the superstructure upon the subsoil in the most efficient and economical manner. It is a matter of regret that in the past "rule of thumb" methods have been used in many cases with unfortunate results. In some cases undue settlements have occurred and, in other cases, foundations have been made unnecessarily heavy and costly.

The problems to be solved are (1) required depth of foundation ; (2) safe bearing pressure ; (3) probable amount of settlement ; (4) most economical type of foundation. The first three involve site and laboratory investigation but the fourth problem can only be solved by considering the economics of design. In fact, it can be said that items (1) to (3) can be solved by or with the help of an expert in soil mechanics, but the last item is solely the function of the designing engineer. It is therefore logical to treat the subject of foundations under two headings, (1) Site Investigation and Soil Mechanics, and (2) Design.

Soil Mechanics.

During the past few decades much attention has been given to the subject called "soil mechanics." This embraces all problems such as foundations, earth pressures, etc., and while it is not strictly correct to say that soil mechanics is a new science, the more scientific approach to such problems has helped to rationalize the design of many engineering works.

Before proceeding to deal with site exploration it is as well to classify the various types as below.

Rocks.		Soils or Earths.	
Granites Dolerites Basalt	Igneous	Gravels Sands Silts Clays	Detrital Sediments

Rocks.		Soils or Earths.	
Gneiss, Schists, Slates	Metamorphic	Peat	Organic
Sandstones, Shales, Limestones	Sedimentary	Top Soil, Laterite	Residual

The soils grouped under the heading of "Detrital Sediments" form the most important section but sedimentary rocks may also be considered.

Purposes of Site Investigation.

1. To find the geological sequence or stratography.
2. To find the ground water level.
3. To take samples for identification.
4. To take samples for tests.

In taking borings the following points should be noted :—

1. The borings should be deep enough to include all stressed zones affected by construction.
2. Sufficient number of borings should be taken (the cost is rarely more than 1% to 2% of the total cost of any work).
3. Water bearing seams and water level should be found and any tidal variations noted.
4. Boulders should not be mistaken for bedrock.
5. Any buried channels should be plotted.

Geophysics includes the exploration of the subsoil. In addition to borings, two other methods may be used (1) *resistivity* or *potential* method by which the drop in potential between two electrodes is measured when a current is passed through the ground. The nature of the ground is found by plotting resistivity as ordinate against electrode spacing as abscissa. A rising curve shows the presence of rock while a falling curve indicates earth or clay; (2) *seismic* method in which artificial earthquake shocks are produced by explosives and the result is recorded on a photographic film. The method is based upon the fact that sand, clay and similar materials transmit sound waves at 1,000 to 6,000 ft. per sec. while rock and other crystalline materials transmit sound at 16,000 to 20,000 ft. per sec. This method is very useful when rock is found within a few feet of the surface.

Borings are cheaper than trial pits although the latter may be better up to a depth of 10 ft. In clays borings up to 30 ft. deep may be made with hand auger. Borings in soils up to 70 ft. deep may be done by a hand winch. For heavier work power winches may be used (note that when dealing with sand, lining tubes should be used).

Ground water is generally caused by water entering the top soil during rains or thaws and filling up the pore spaces in the soils. It rises to a level known as the *water table* which is usually a subdued replica of the ground surface. The zone below the water table is called the *zone of saturation* and the zone above the water table is called the *zone of aeration*. There may be local zones of saturation lying above the water table. There is also a certain amount of moisture above the water table due to capillary action which depends upon the number of pores in the soil and is inversely proportional to their diameter. It may be necessary to lower the level of the water table temporarily or permanently by driving sheet piles and pumping or by sinking perforated pipes into the soil and pumping. When ground water is found at or near foundation level, tests should be carried out to find its nature (acid, basic or neutral). The relative acidity or alkalinity is indicated by the ρH value which may be defined as the logarithm of the reciprocal of the grammes of ionized hydrogen per litre of solution or suspension (ρH for distilled water = 7·0). The lower the ρH value the greater the acidity, the higher the ρH value the greater the alkalinity. As the acidity rises so the corrosion of metals increases. Acids and acid salts attack concrete. Soft or distilled water may also attack concrete as will sulphates, sulphites, thio-sulphates and sulphides. Wherever the presence of deleterious matter in the ground water is detected, special precautions should be taken. Generally dense concrete with well graded fine aggregate is best for resisting attack and (with a few exceptions) high alumina cement has a greater resistance to attack by chemicals than ordinary portland cement.

Field Identification (for other than "wash" borings).

Boulders over 3 inches.
Gravel—3 inches to $\frac{1}{8}$ inch or No. 7 B.S. Sieve.
Sands No. 7 to No. 200 B.S. Sieve.

Sands possess *no* plasticity and *no* cohesion. They may be found well mixed in coarse, medium or fine grades.

Coarse	7 - 25	B.S. Sieves
Medium	25 - 72	
Fine	72 - 200	

Silts mostly pass No. 200 B.S. Sieve and the particles are mostly invisible. They possess plasticity and cohesion but very little permeability.

Particle Sizes.

Sand.	Coarse	2·00 mm. - 0·6 mm.
	Medium	0·6 mm. - 0·2 mm.
	Fine	0·2 mm. - 0·06 m.m
Silt.	Coarse	0·06 mm. - 0·02 mm.
	Medium	0·02 mm. - 0·006 mm.
	Fine	0·006 mm. - 0·002 mm.
Clay.	Less than 0·002 mm.	

Clays possess plasticity and cohesion.

In order to distinguish between fine sands and silts the following points should be noted :—

	Silt.	*Fine Sand.*
1.	Particles invisible.	Most particles visible.
2.	Some plasticity (a thread can be rolled).	No plasticity.
3.	Rough texture.	Gritty.
4.	Dries into lumps with some cohesion but can be easily powdered.	Dry lumps (almost no cohesion) can be powdered but not easily).

A similar comparison of clay and silt gives :—

	Clay.	*Silt.*
1.	Smooth greasy touch.	Rough texture.
2.	Sticks to fingers.	Dries off rapidly.
3.	No dilatancy.	Definite dilatancy.
4.	Dry lumps cannot be powdered.	Dry lumps can be powdered.

Dilatancy Test.—Take a small pat and mould it into a ball, then water comes out ; if the ball is pressed it absorbs water (this applies also to fine sands).

Various mixtures, *e.g.*, silty clay or silty sand, may occur, also intermediate types such as sandy clay. Marls are clays with lime.

Structures of Types of Subsoil.

Sands. Loose or dense (compact). Fine sands tend to be loose. Coarse sands tend to be dense.

Clays. Soft, firm or stiff.

Soft clays are easily moulded in the fingers. They may have been consolidated only by their own overburden pressure.

Firm clays cannot be moulded in the fingers without pressure.

Stiff clays (*e.g.*, London) cannot be moulded at all, being over-consolidated.

Clays in general are not elastic and their shear strength varies considerably.

Undisturbed Sampling.

Sands can be removed from a trial pit in a tin. Boring is almost impossible except by freezing or by injection of some emulsion. Clay borings can be done by tubes. At least one sample should be taken for each stratum with a maximum of one every 5 ft.

Physical Properties of Soils.

Since the physical properties of soils affect their behaviour in foundations, these will now be considered. The behaviour of any soil will vary according to its porosity and water content.

$$\text{Porosity } (n) = \frac{\text{volume of voids}}{\text{total volume}}$$

$$\text{Voids ratio } (e) = \frac{\text{volume of voids}}{\text{volume of solids}}$$

$$\therefore\ n = \frac{e}{1+e} \text{ and } e = \frac{n}{1-n}$$

For any *unit* volume then

$$\text{Volume of solids} = 1 - n = \frac{1}{1+e}$$

$$\text{Weight of solids} = SW\,(1-n) = \frac{SW}{1+e}$$

where S = specific gravity of grains.
and W = weight of water per unit volume.

Now if the voids are saturated

$$\begin{aligned}\text{Weight of water} &= \text{porosity} \times \text{density of water.}\\ &= n \times W\\ &= \frac{e}{1+e} \times \text{density of water}\end{aligned}$$

$$\text{Dry density} = SW\,(1-n) = \frac{SW}{1+e}$$

$$\begin{aligned}\text{Saturated or bulk density} &= \text{weight of water and dry density.}\\ &= W\,[n + S\,(1-n)]\\ &= W \times \frac{S+e}{1+e}\end{aligned}$$

$$\begin{aligned}\text{Submerged density} &= \text{wt. per unit volume under ground water.}\\ &= \text{dry density} - \text{weight of water.}\\ &= SW\,(1-n) - W\,(1-n)\end{aligned}$$

$$= W \quad (1-n) \; (S-1)$$

$$= \frac{S-1}{1+e} \times W$$

Example.—Take a sand with S = 2·4 and let n = 25%.

Then dry density = 2·4 × 62·3 × ·75 = 112 lbs./ft.^{3}
bulk density = 62·3 (·25 + 2·4 × ·75)
= 62·3 × 2·05 = 127·8 lbs/ft.^{3}
submerged density = 62·3 × ·75 × 1·4 = 65·5 lbs/ft.^{3}

$$\text{Water content (w)} = \frac{\text{weight of water}}{\text{weight of solids}}$$

$$= \frac{e}{S} \text{ (when soil is saturated).}$$

Water content can be found by weighing a small sample, drying for 24 hours and weighing again.

Example.—A clay has a water content of 35% and S = 2·8.

Assuming the clay to be saturated e = wS = 0·98.

∴ dry density = 87·5
bulk density = 119·62
Submerged density = 119·62 −62·3 = 57·32 } lbs/ft.^{3}

Partial Saturation.

If degree of saturation = s.

$$\text{Then weight of water} = s \times \frac{e}{1+e} \times W$$

$$\text{and water content} = se/S = w$$

$$\text{also bulk density} = \frac{S+se}{1+e} = W$$

$$= \frac{S\,(1+W)}{1+e} \times W$$

For instance if bulk density = 110 lbs/ft.^{3}

w = 20%. S = 2·6, find s. e = wS = 0·52

$$\text{since bulk density} = 110 = \frac{2{\cdot}6 + {\cdot}52s}{1 + {\cdot}52} \times 62{\cdot}3$$

$$s = \frac{\dfrac{1{\cdot}52\,(110)}{62{\cdot}3} - 2{\cdot}6}{0{\cdot}52}$$

$$= \frac{0 \cdot 09}{0 \cdot 52} = 17 \cdot 3\%$$

Liquid Limit is the *upper* end of the plastic range and can be defined as the moisture content (per cent) at which 25 light blows on the dish containing the specimen will only just close the groove in the sample over a length of ½″ (see Fig. 1).

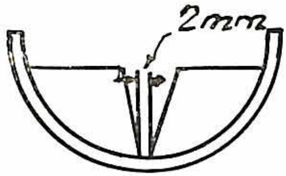

Fig. 1.

Plastic Limit is the *lower* end of the plastic range and can be taken as the moisture content (per cent) at which threads ⅛ inch thick can be rolled from the sample without breaking.

Typical values are given below.

	London Clay	*Silty Clay*	*Silt*
Liquid Limit	75	68	36
Plastic Limit	25	24	20

Plasticity Index = liquid limit − plastic limit (per cent.).

Shrinkage Limit is usually defined as the moisture content (per cent.) at which evaporation fails to produce any further decrease in *volume* (although the *weight* may decrease). Since the volume of any sample at its shrinkage limit is the same as that in its completely dry state, the voids ratio can be found from specific gravity S and dry density P_d.

$$\text{S.L.} = \frac{1}{\text{dry density}} - \frac{1}{s} = \frac{e}{s}$$

$$e = \frac{\text{volume of voids}}{\text{volume of solids}}$$

The shrinkage limit is also the limit between the semi-solid and solid states.

Shrinkage ratio = dry density (which may vary according to whether the specimen is disturbed or undisturbed).

$$\text{Dry Density} = \frac{\text{Weight of dry soil sample}}{\text{Volume of dry soil sample}}$$

Examination of Soil Samples.

The laboratory examination is most important and provides evidence as to the physical characteristics and properties. The sample should first be tested for liquid limit and plastic limit and corresponding values plotted against depth of boring. These give a record of the soil profile, indicating the more important strata and which samples should be subjected to mechanical testing.

The two most important mechanical tests are the consolidation test and the shear test. The *consolidation* test is usually carried out by means of an oedometer which was first introduced by Prof. K. v. Terzaghi and is shown diagrammatically in Fig. 2. The specimen is placed between two porous stones in a brass ring and a certain known pressure is applied. Observations are taken from time to time of the thickness of the specimen until no further consolidation takes place (in about 24 hours). The pressure is changed and the process repeated until the maximum pressure is reached, and a series of curves is plotted showing the relation of pressure and thickness (or voids ratio). Another series of curves is plotted showing the relation between thickness and time and degree of consolidation (Figs. 3*a* and 3*b*). From these curves it is possible to estimate the compressibility and consolidation coefficient.

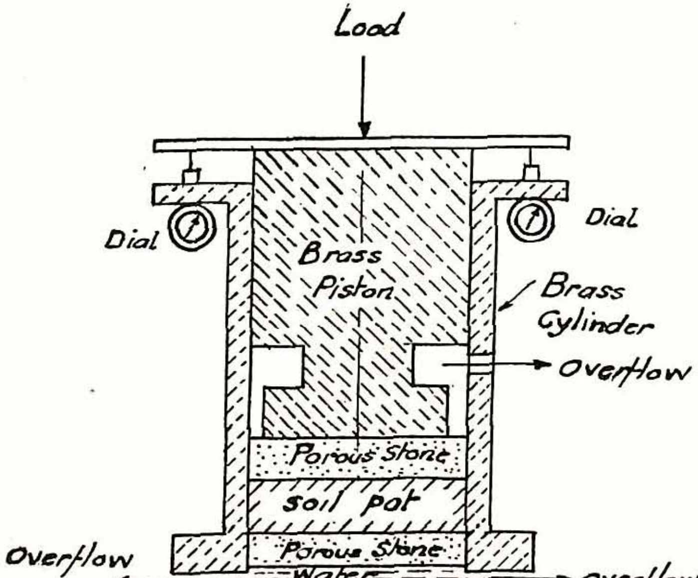

Fig. 2.

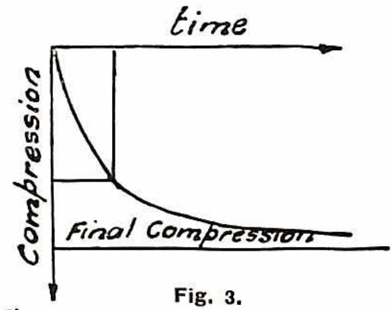

Fig. 3.

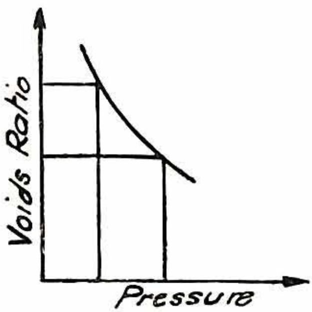

Fig. 3 (a).

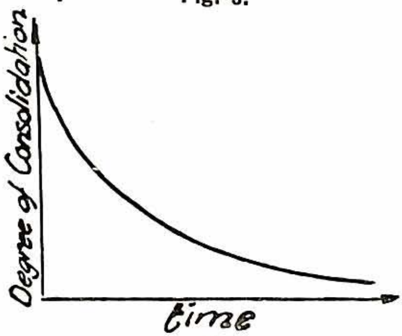

Fig. 3 (b).

Theory of Consolidation.

Let e = voids ratio = $\dfrac{\text{volume of voids}}{\text{volume of solids}}$

Let original thickness $= l_1$ }
Let original voids ratio $= e_1$ } for pressure ρ_1
Final pressure $= \rho_1 + \sigma$
Final thickness $= l_2$ }
Final voids ratio $= e_2$ } for pressure $\rho_1 + \sigma$

$$\frac{\text{Original volume of voids}}{\text{Original volume of solids}} = \frac{e_1}{e_1 + 1}$$

$$\frac{\text{Decrease in volume of voids}}{\text{Original volume of specimen}} = \frac{e_1 - e_2}{e_1 + 1}.$$

But, since decrease in volume of voids = decrease in volume or specimen

$$\frac{\text{Decrease in volume of specimen}}{\text{Original volume of specimen}} = \frac{e_1 - e_2}{e_1 + 1}$$

and since area is constant.

$$\frac{\text{Decrease in thickness}}{\text{Original thickness}} = \frac{e_1 - e_2}{e_1 + 1} = \frac{l_1 - l_2}{l_1}$$

$$\therefore \quad l_1 - l_2 = \frac{e_1 - e_2}{e_1 + 1} \times l_1$$

$$\therefore \quad \text{compressibility} = \frac{l_1 - l_2}{\sigma \times l_1} = \frac{e_1 - e_2}{\sigma\,(e_1 + 1)}$$

If the slope of the curve $p - e$ over the length corresponding to σ is a

$$a = \frac{e_1 - e_2}{\sigma}$$

if σ is small $a = -de/dp$

$$\text{Compressibility} = \frac{a}{1 + e_1}$$

which is a function of e and hence of p. The value of $\frac{a}{1+e}$ can be found from the $p - e$ curve for any value of p and e and the final compression calculated for each increment of pressure.

The *rate* of consolidation is more difficult to estimate since it involves the degree of consolidation μ and the factor ct/d^2.

where c = coefficient of consolidation (this is constant for any given specimen).

t = time after the application of the load.

d = drainage path, *i.e.*, maximum distance which water has to travel to reach a free draining surface.

For oedometer test $d = \dfrac{\text{thickness of specimen}}{2}$

If $s\infty$ = final compression of specimen.

st = compression of specimen after time t.

$$\mu = \frac{st}{s\infty} \text{ (See Fig. 3 (b)).}$$

The *shear* test is usually carried out in a shear box (shown diagrammatically in Fig. 4). The test gives the relation between shear strength and normal pressure. In carrying out the test the specimen is contained between two layers of porous stone, each of which has a series of parallel ridges at right angles to the direction of shear strain. The ridges permit surplus water to be drained off and also grip the specimen. At least two tests should be carried out (with different pressures) in order to find the cohesion and the angle of internal friction. The shear strength is most

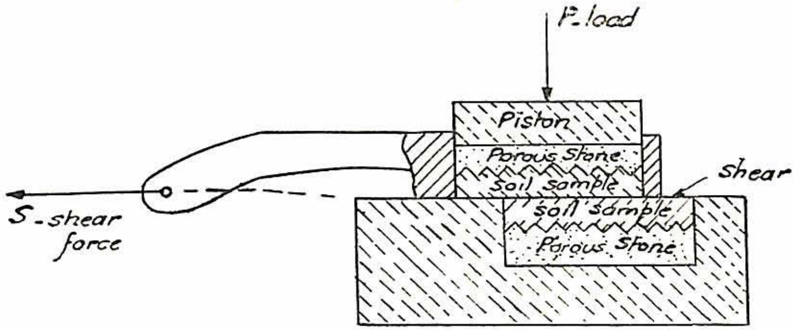

Fig. 4.

important in the case of clays. Field tests on small samples can be carried out for cohesive soils such as clays, silts, etc., by using specimens taken from borings and a portable compression apparatus (as described in Engineering, Jan. 19, 1940, p. 57). For cohesive soils the compressive strength is twice the shear strength and therefore the apparatus gives the amount of the shear strength. Shear strength should be plotted against depth. Specimens should be placed in airtight containers after the field tests, and sent to the laboratory to be tested for density and index properties. This method can be used for borings up to 25 ft. deep, for deeper borings large undisturbed samples should be obtained by normal boring apparatus.

Laws of Soil Mechanics.

(1) **Coulomb's law** giving the relation between shear stress and pressure.

$$S = c + p \tan \phi.$$

where c = cohesion.

p = pressure.

ϕ = angle of internal friction.

(2) **Boussinesq's Law** of transmission of stress in a homogeneous isotropic soil

$$f_z = \frac{3Q}{2\pi p r^2} \cos^3\alpha$$

where f_z = vertical component of stress produced by a load Q applied to the surface of an indefinite space.

r = distance of a point in a horizontal plane measured from the point of application of the load to the upper surface.

α = vertical angle of the straight line joining the point of application of the load on the upper surface to the point on the plane where f is to be found.

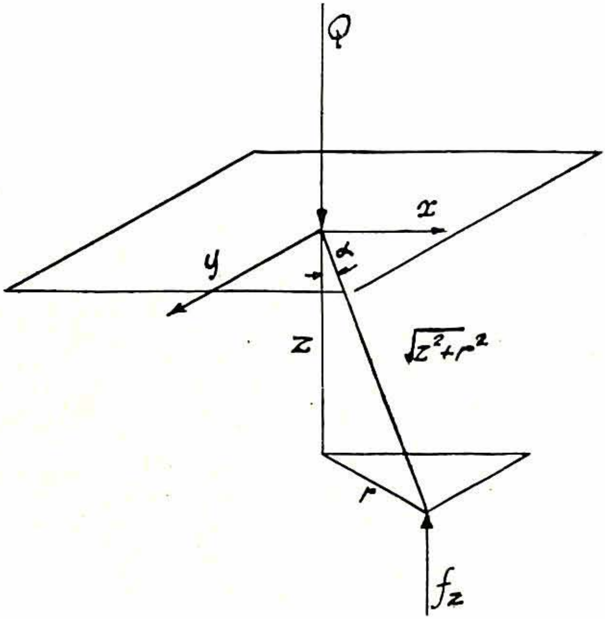

Fig. 5.

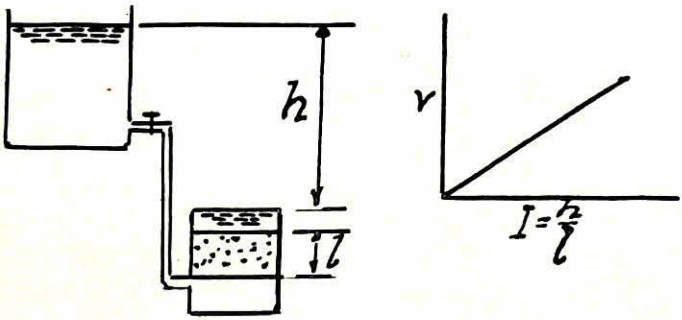

Fig. 6.

(3) **Darcy's Law of Filtration** (see Fig. 6).

$$\text{Velocity} = \frac{Q}{At} = K\frac{h}{l}$$

where Q = discharge.
A = area of sample.
t = time.
K = coefficient of permeability.
$\frac{h}{l}$ = hydraulic gradient = I

K can be found by plotting V for various values of h/l and plotting a graph. Then $K = D \div h/l$

Boussinesq's Law assumes soil to be homogeneous and isotropic. While this is not strictly true, the relationship has been widely accepted. It can be expressed in the form

$$f_z = \frac{3Q}{2\pi z^2}\cos^5\alpha = \frac{3Q}{2\pi} \times \frac{z^3}{(r^2+z^2)^{5/2}}$$

and shear

$$S_z = \frac{3Q}{2\pi z^2}\cos^4\alpha\sin\alpha = \frac{3Q}{2\pi} \times \frac{z^2r}{(r^2+z^2)^{5/2}}$$

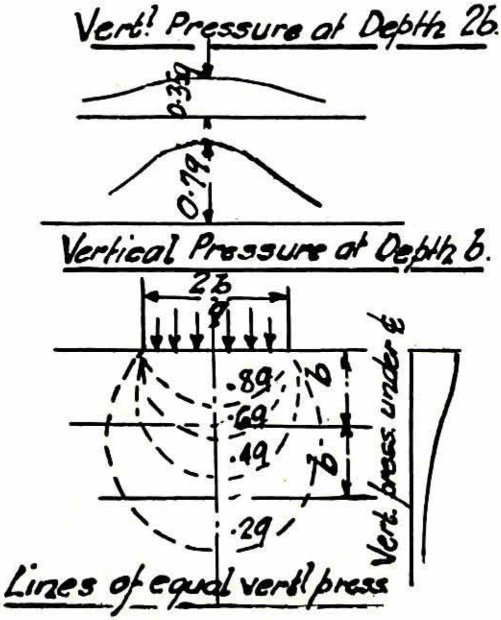

Fig. 7.

By plotting values of f_z the well-known pressure bulb is obtained (see Fig. 7). This shows the pressure under a strip footing due to a uniform load. For point loads the solution of various problems can be simplified by the use of charts prepared by Prof. N. M. Newmark.* From the pressure bulb, the vertical pressure at various depths can be found. If this is less than the safe bearing value of the subsoil at that depth, the design is satisfactory. If it exceeds the safe value, the design must be modified. When dealing with a number of adjacent footings, the pressure bulbs may overlap, hence some zones of the subsoil may be subject to additional stress and this condition should be checked. By referring to Fig. 7 it can be seen that at a depth of $1\frac{1}{2}$ times the width of the footing the pressure = 0·16 × pressure under the footing. This indicates the necessity for carrying the borings to a sufficient depth. The diagram also shows the pressure variation under the centre line of the foundation and the variation of pressure on a horizontal plane at a depth of half the width of the footing.

When dealing with foundations on clay subsoils the shear strength should be checked, since the supporting strength of clays is a function of the shear stress.

The intensity of shear stress at depth z is given by

$$s_z = \frac{q}{\pi} \sin 2a$$

$$= \frac{q}{\pi} \times \frac{2\,z/b}{1 + (z/b)^2}$$

where q = pressure under footing.
$2b$ = width of footing.

Fig. 8 shows a typical diagram showing shear stress in a clay subsoil.

If q is assumed to be unity, s_z can be expressed in terms of q thus for various depth

$z/2b$	s_z/q
1	0·255
2	0·150
3	0·103
4	0·079

At a depth of twice the width of the footing, the shear is 0·15 × pressure under the footing. Hence the borings should be at least as deep as the width of the building or structure (*not* the width of the *individual footings* if the spacing is less than 5 times their width). The depth of borings is most important where the strata vary, *e.g.*, a footing may rest on a stiff clay but this may overlay softer clay

* "Influence Charts for Computation of Stresses in Elastic Foundations" —Unversity of Illinois Eng. Expt. Station 40, No. 12.

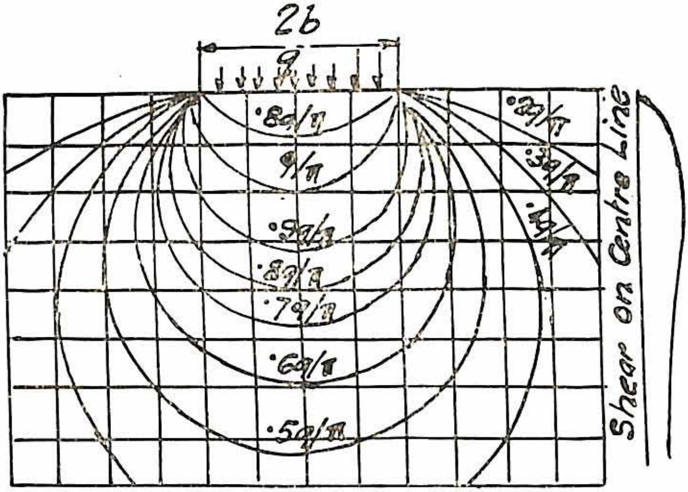

Fig. 8.

or similar material. Many notable failures have occurred through borings not being carried to a sufficient depth to reveal the fact that an overstressed zone existed below the stratum on which the foundation rests. A good rule is to carry the borings to a depth $= 1\frac{1}{2} \times$ width of foundation.

Bearing Capacity of Soils.

(1) **Sands.**

Rankine gives $\rho \times D \left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2$

ϕ = angle of friction.
ρ = density.
Terzaghi gives
$qu = B\rho \tan^4 (45^\circ + \phi/2) \, [\,1 + D/B + K\,(D/B)^2\,]$
K = constant usually less than 0·25.

If the last term is omitted

$qu = B\rho \tan^4 (45^\circ + \phi/2) \, [1 + D/B]$
which is the usual form.

Since $\tan^4 (45^\circ + \phi/2) = \left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2$

the formula can be written thus

$$qu = \rho \left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2 (B + D)$$

$$= \text{Rankine value} + B\rho \left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2$$

Ritter gives $qu = [\rho D + \rho B/2 \tan(45° + \phi/2)] (\tan^4 (45° + \phi/2) - 1) + \rho D$

ρD = overburden pressure.
and $\rho B/2 \tan(45° + \phi/2)$ = correction for width of base.

The B.S. Code of Practice suggests that qu should be found by a load test on the subsoil. Comparing the different expression take B = 6 ft ; D = 5 ft. ; ρ = 110 lbs/ft.^{3} ; and ϕ = 30° and 35°. Values of qu are given in tons per sq. ft.

ϕ	*Rankine*	*Terzaghi*	*Ritter*
30°	2·22	4·87	4·25
35°	3·35	7·4	7·00

The values given by Terzaghi and Ritter agree fairly closely. Since they involve the base width B, the value of qu will rise as B increases and the settlement will be accordingly greater. Such foundations should be designed with the settlement as the governing condition. For sands settlements occur quickly and reach their maxima soon after the load is applied. The maximum values are usually comparatively small and can be found from the expression

$$\triangle = q^2 \times \frac{2B}{N}$$

where N is a function of the centre of gravity of the stressed zone, *i.e.* (B + D). D = depth of foundation below surface).

N can be found by carrying out load tests on small areas and plotting the settlements. If a curve is plotted showing N as ordinate against (D + B) as abscissa, the value of N corresponding to the particular value of (B + D) can be found and $\triangle$ calculated.

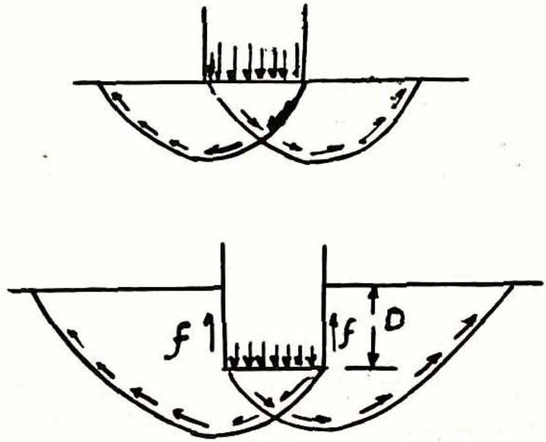

Fig. 9.

(2) **Clays and Silts.**

In the case of clay subsoils, the bearing value depends principally upon the shear strength.

Fig. 9 shows a foundation resting upon the surface of a clay layer. The base will fail as shown, the value of qu being approximately qs (s = average shear strength in a depth equal to the width of the base).

If in Fig. 9 (*a*) the base is at depth D below the surface then if

W = total load.
A = area of base.
f = skin friction.
p = perimeter area in contact with the clay.

Nett load on base = $W^1 = W - pf$

and pressure $= \dfrac{W - pf}{A} = 6s + \rho D$ (approx.).

Example.—A bridge pier is 10 ft. dia. and rests upon clay whose average shear strength is 3 cwts. per sq. ft. and whose skin friction is $2\frac{1}{2}$ cwts per sq. ft. Density of clay 1 cwt/ft.^{3}.

When D = 0. $qu = 3 \times 6 = 18$ cwts/ft.^{2} = 0·9 ton/ft.^{2}

At a depth of 20 ft.

$Pf = \pi \times 10 \times 20 \times 2{\cdot}5 = 500\ \pi$ cwts.
$= 25\pi$ tons = 78·5 tons.

$$\frac{W - Pf}{A} = 18 + 20 = 38 \text{ cwts.} = 1{\cdot}9 \text{ tons/ft.}^2$$

$A = 25\pi = 78{\cdot}5$ ft.^{2}

$\therefore$ W − Pf = 78·5 × 1·9 = 149·5 tons
Add Pf = 78·5 ,,
228·0 ,,

$$qu = \frac{228}{78{\cdot}5} = 2{\cdot}9 \text{ tons/ft.}^2$$

Increase in qu = 2·9 − 0·9 = 2·00 tons/ft.

Many authorities have given various values of qu for clay and space does not permit these to be dealt with at length.

Bell gives

$$qu = \rho D \tan^4 (45° + \phi/2) + 2c \tan^3 (45° + \phi/2) + 2c \tan (45° + \phi/2)$$

$$= \rho D \left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)^2 + 2c\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)^{3/2} + 2c\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)^{1/2}$$

The first term is of course the Rankine value. For cohesive soils, Terzaghi gives the following value :—

$$qu = \frac{B\rho}{2}\left(\frac{1-\tan^4\beta}{\tan^5\beta}\right) + \frac{2c}{\tan\beta\sin^2\beta}$$

$$= \frac{B\rho}{2}\left(\frac{4\sin\phi}{(1-\sin\phi)^2\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^{1/2}}\right) + \frac{2c}{\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^{1/2}\sin^2(45^\circ-\phi/2)}$$

If $\phi = 20^\circ$ $c = 750$ lbs/ft.^{2} $\rho = 112$ lbs/ft.^{3}
Bell's value = 3·96 T/ft.^{2}
Terzaghi value = 3·47 ,,

In the case of Bell's formula when $\phi = 0$, $s = c$
and $qu = \rho D + 4c = \rho D + 4s$
ρD = overburden pressure.

Other values are
Fellenius $qu = 5{\cdot}55s$.
Krey $qu = 6{\cdot}05s$
Prandtl-Jurgenson $qu = 5{\cdot}14s$.

The B.S. Code of Practice gives $3c + \rho D$ for strip footings and $c\left(3 + \frac{2}{b}\right) + \rho D$ for piers and columns ($c = 2s$).

For *rigid* circular areas Prandtl's method as developed by Hencky and Jurgenson gives $qu = 5{\cdot}64s$.

The ultimate capacity is about $5s$ for shallow footings. For deeper footings qu should be increased by but more investigation is necessary. The shear strength should be tested at depths at least equal to $1\frac{1}{2}\times$ width of base and a conservative factor of safety adopted.

Settlement of Foundations on Clays.

The theory of consolidation can be applied to this problem. Take the case of a column base 5 ft. square carrying a load of 100 tons resting on sand. The underside of the base is 20 ft. above the top of a layer of clay which is 20 ft. thick.

$$qu = \frac{100}{25} = 4 \text{ tons/ft.}^2$$

If angle of spread through sand is 1·2 then q at top of clay

$$= \frac{100}{25 \times 25} = 0{\cdot}16 \text{ tons/ft.}^2$$

If depth of sand is 25 ft. original pressure is

$$\frac{25 \times 100}{2240} = 1{\cdot}1 \text{ tons/ft.}^2$$

$$\text{final pressure} = 1{\cdot}26 \text{ tons/ft.}^2$$

$$\text{At centre of clay original pressure} = 1{\cdot}1 + \frac{10 \times 112}{2240} = 1{\cdot}6 \text{ tons/ft.}^2$$

$$\text{Final pressure} = 1{\cdot}6 + 0{\cdot}16 = 1{\cdot}76 \text{ tons/ft.}^2$$

If voids ratios from consolidation tests on clay are

$$p_1 = 1{\cdot}6 \qquad e_1 = 0{\cdot}85$$
$$p_2 = 1{\cdot}76 \qquad e_2 = 0{\cdot}80$$

$$\triangle = \frac{e_1 - e_2}{1 + e_1} \times \text{depth}$$

$$= \frac{0{\cdot}05}{1{\cdot}85} \times 240 = 6{\cdot}5 \text{ inches}$$

The rate of consolidation for clay varies inversely as the square of the thickness. For laboratory tests $d = \frac{1}{2} \times$ thickness of sample. In this case $d = 20$ ft. as the clay is assumed to be overlying impervious strata. For laboratory tests half the settlement takes place in ten minutes.

$$\therefore \frac{\triangle}{2} \text{ takes place in } \frac{(20 \times 12)^2}{(\frac{1}{2})^2} \text{ minutes.}$$

$$= 1600 \text{ days.}$$

i.e., about 3 inches in $4\frac{1}{2}$ years.

Factors of Safety.

Two factors should be considered (1) against failure, and (2) against large or differential settlements. For (1) the F. of S. should be between 1·5 and 2·0 (a F. of S. greater than 2 would lead to expensive foundations). 1·5 may be used for dense sand, etc., but 2·0 should be used for clays. 2·0 can be used for light sheds, etc., but in the case of steel and R.C. frame buildings a F. of S. of 3 should be adopted.

Unequal settlements lead to secondary stresses, distortion and cracking of plaster, etc. In order to reduce this a high F. of S. should be used when designing the foundation. This again involves the question of the type of footing, *i.e.*, independent, strip or raft.

Distribution of Pressure.

For most cases except circular tanks and other special structures, foundations should be designed for uniform pressure under the base. The distribution depends to some extent on the rigidity of the base. For perfectly flexible bases, the distribution is parabolic; for rigid bases the maximum pressure may be at the edges. For "rigid" foundations the "spread" of pressure through the subsoil is at 1 horizontal to 2 vertical, which gives a close approximation to the elastic theory.

The principal types of foundations are :—

(1) Footings to brick or masonry walls or piers.
(2) Grillage foundations to stanchions.
(3) Concrete foundations to stanchions.
(4) "Pier" foundations to stanchions.
(5) "Bridge" and "cantilever" foundations.
(6) Combined and raft foundations.
(7) Piled foundations.
(8) Bridge piers.
(9) Machinery foundations.

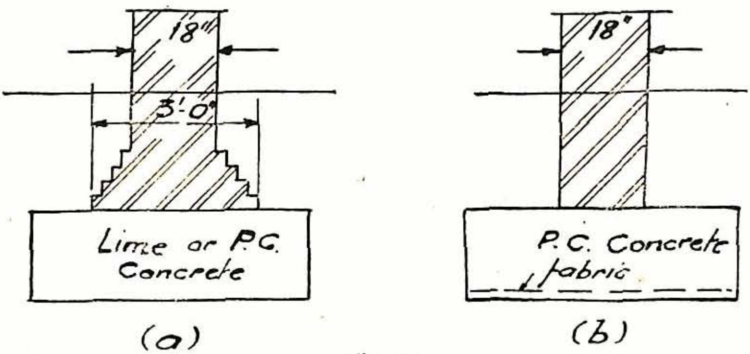

Fig. 10.

Type (1). Before the introduction of steel-framed construction and reinforced concrete, practically all superstructures were carried on brick or masonry walls and piers. Foundations were constructed by stepping-out the brickwork at the base to double its thickness, the lowest course resting on a block of lime or P.C. concrete resting on the subsoil (see Fig. 10 (*a*)). Since the steel or R.C. frame was generally adopted, the walls were greatly reduced in thickness, as their only function now is to keep the weather out. Present-day practice is to keep the wall the same thickness throughout and the foundation consists of a P.C. concrete block reinforced by a layer of light fabric (see Fig. 10 (*b*)). The thickness of concrete and amount of reinforcement are determined by the shear and B.M. at the wall face, as in the case of Type (3)—Concrete Foundations.

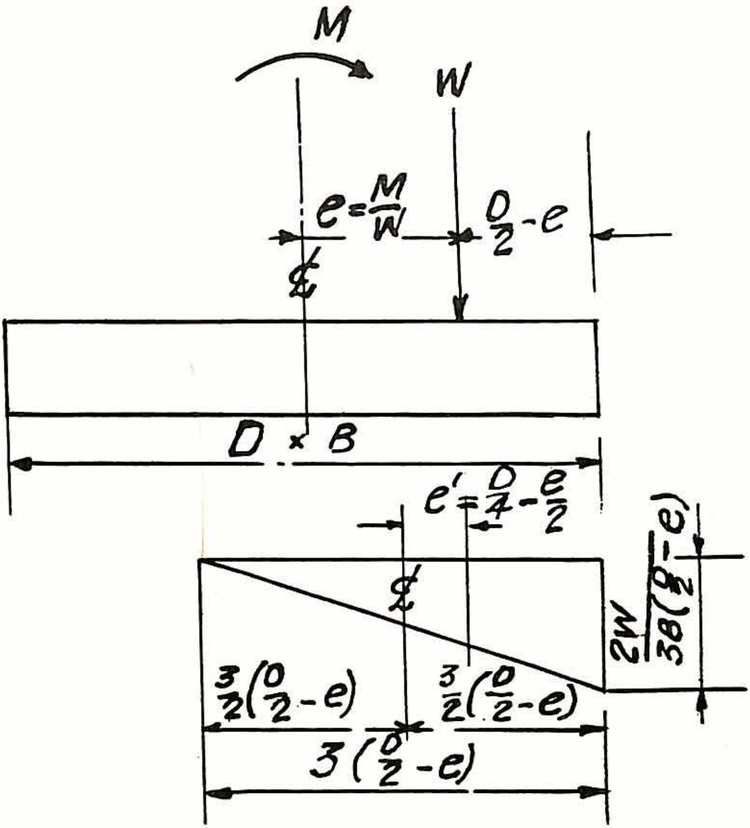

Fig. 11.

Type (2). Grillage foundations are commonly used for stanchions. Before discussing this type of foundation, it should be remembered that many stanchions are subject to overturning moment (due to wind or live loads) as well as to direct load. Where a foundation carries direct load only, the size of the bearing on the subsoil is determined by the bearing value. Where there is bending in addition to direct load, this must be taken into account. Consider a foundation block B ft. wide × D ft. long, carrying a direct load W tons and a moment of M ft/tons.

$$\text{Direct pressure } p_d = \frac{W}{B \times D} \text{ tons per square foot.}$$

Section modulus $= BD^2/6$ ft.^{3} (about centre of gravity).
Bending pressure $= \pm M/BD^2/6 = \pm 6M/BD^2$ tons per sq. foot.
Maximum pressure $= W/BD + 6M/BD^2$ tons per sq. foot.
Minimum pressure $= W/BD - 6M/BD^2$ tons per sq. foot.

The maximum pressure should not exceed the bearing value of the subsoil. The minimum pressure should be positive, *i.e.*, direct pressure should be greater than the bending pressure, otherwise there will be "uplift" on the foundation block. For zero pressure $W/BD = 6M/BD^2$, *i.e.*, $D = 6M/W$.

This rule tends to make the bearing block rather large and it is more economical to adopt the "no-tension" method. The rule that $D = 6M/W$ means that the equivalent direct load must lie within the middle third. The more economical method is to calculate the eccentricity $e = M/W$ in ft. Then, if we assume that e gives the position of the line of the middle third and the effective length of the block is $3\ (D/2 - e)$. See Fig. 11.

$$\text{The direct pressure} = \frac{W}{3\ B\ (D/2 - e)} \text{ tons per sq. foot.}$$

The bending pressure can be calculated from the "effective" eccentricity $e^1 = D/4 - e/2$ and is given by

$$W\ (D/4 - e/2) \div \frac{B\ \{3/2\ D - 3\ e\}^2}{6}$$

$$= 6\ W\ (D/4 - e/2) \div 9\ B\ (D/2 - e)^2$$
$$= \tfrac{2}{3}\ W\ (D/4 - e/2) \div B\ (D/2 - e)^2$$

$$\text{Max. pressure} = \frac{W}{3\ B\ (D/2 - e)} + \frac{W\ (D/2 - e)}{3\ B\ (D/2 - e)^2}$$

$$= \frac{W}{3B\,(D/2 - e)} + \frac{W}{3B\,(D/2 - e)} = 2 \times \frac{W}{3B\,(D/2 - e)}$$

$$\text{Minimum pressure} = \frac{W}{3B\ (D/2 - e)} - \frac{W}{3B\ (D/2 - e)} = 0$$

The maximum pressure $\left(2 \times \frac{W}{3B\ (D/2 - e)}\right)$ must not exceed the safe bearing value for the subsoil.

Wherever a column or stanchion is subject to bending, it becomes necessary to provide foundation or holding-down bolts. If W is the direct load (tons) and M the moment (ft. tons), then the force per bolt or set of bolts is $M/s - W/2$, where s is the spacing of the bolts (ft.) measured in the plane of bending. The allowable stress in H.D. bolts of mild steel is usually taken at $7\frac{1}{2}$ tons per sq. inch, measured on the nett section, *i.e.*, area at the bottom of the thread. H.D. bolts should have square heads and necks and washer plates, with square holes to prevent rotation during tightening. They should be set to template and in wooden boxes tapered from top to bottom to allow for adjustment during erection. The holes in the baseplate should have $\frac{1}{4}''$ to $\frac{1}{2}''$ tolerance for the same reason. The usual practice is to allow 1″ space between the underside of the baseplate and the top of the concrete. This space is filled with neat cement grout after final positioning and levelling of the stanchion. For large base plates, it is necessary to provide two or more grout holes in the base 2″ to 3″ dia. to ensure proper filling of the grout space.

Grillage foundations consist of one or two tiers of steel joists encased in concrete and usually rest on blocks of mass concrete. The allowable pressures on mass concrete can be taken as below :—

1 : 12 mix., 5 tons/ft.^{2}; 1 : 10 mix, 10 tons/ft.^{2}; 1 : 8 mix., 15 tons/ft.^{2}; 1 : 6 mix., 20 tons/ft.^{2} The corresponding values for reinforced concrete are :—1 : 2 : 4 mix., 30 tons/ft.^{2} 1 : $1\frac{1}{2}$: 3 mix., 35 tons/ft.^{2}; 1 : 1 : 2 mix., 40 tons/ft.^{2}.

The steel joists should have a cover of 4″ at the sides and ends, and be spaced so as to allow not less than 2″ between flange edges, so that spaces between joists can be filled with concrete. The joists should be connected together by angles or tube ferrules. The most convenient sections are joists with a 5″ to 6″ flange and comparatively shallow web, *i.e.*, really a column section. By reason of the extra lateral stability given by the concrete casing and filling, working stresses 50% higher than usual are allowed in grillage joists (see L.C.C. Bye-laws). The effective area of a grillage can be taken as either (1) the area of the steel length × overall breadth, or (2) the area of the steel + concrete cover. The more usual practice is to take area (1). In designing grillages the following maximum stresses must be found :—(*a*) bending, (*b*) shear, (*c*) web buckling. Web buckling is really failure of the web as a short column and must not be confused with shear.

In order to illustrate grillage design, take as a practical example a stanchion carrying a load of 400 tons, the bearing value of the subsoil being 2 tons per square foot and base plate 3 ft. square (approx.). Use 6 : 1 concrete and steel stresses in accordance with B.S. 449 : 1948, including amendment of July, 1949. Safe pressure on concrete 20 tons per square foot.

Area of grillages required = 400/20 = 20 sq. ft.
∴ Length of beams required = 20/3 = 6′8″.
Use 5 beams. Then load per beam = 80 tons.

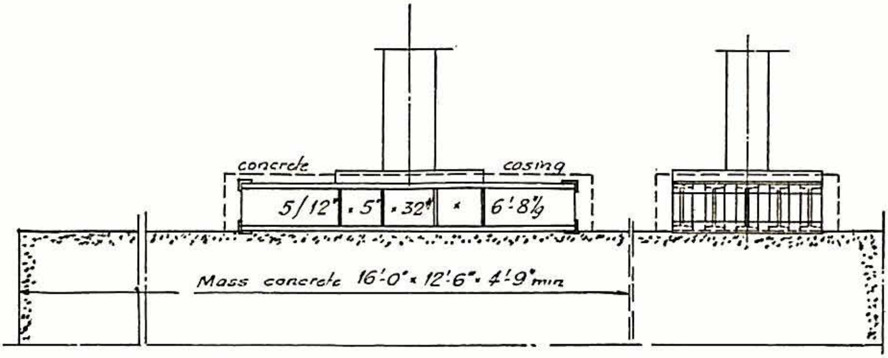

Fig. 12.

$$\text{Max. shear} = \frac{80 \times 1{\cdot}83}{6{\cdot}67} = 22 \text{ tons.}$$

Try $12'' \times 5'' \times 32$ lbs. Shear value (ex-tables) $= 27{\cdot}3$ tons
$+33\frac{1}{3}\% = 9{\cdot}1$,,
$36{\cdot}4$,,

Max. B.M. $= 40 \times (20-9) = 440$ in. tons.

$$\text{Section modulus required} = \frac{440}{10 \times 1\frac{1}{3}} = 33 \text{ in.}^3$$

$12'' \times 5'' \times 32$ lbs. has section modulus of $36{\cdot}84$ in.^{3}, therefore this section is suitable. Check web buckling.

Load $= 80$ tons. Safe value $= (13{\cdot}58 + 36 \times 2{\cdot}26)\ 1\frac{1}{3} > 80$.
Bearing value at 12 tons/in.^{2} Safe bearing value
$= (8{\cdot}53 + 36 \times 4{\cdot}20)1\frac{1}{3} > 80$ tons.

$$\text{Space between beams} = \frac{36 - (5 \times 5)}{4} = 2\frac{3}{4}''$$

(Width of base can be increased to 3′ 1″ so that space between beam flanges is 3″ to satisfy provisions of Clause 27b of B.S. 449). No stiffeners are required.

The area of concrete required $= 400/2 = 200$ sq. ft.

Assuming 45° dispersion through concrete, if D is depth of concrete,

$$(6{\cdot}67 + 2\,D)\ (3 + 2\,D) = 200$$
$$20 + 19{\cdot}34\,D + 4\,D^2 = 200$$
$$4\,D^2 + 19{\cdot}340 - 180 = 0$$

$$D = \frac{-19{\cdot}34}{8} + \frac{\sqrt{19{\cdot}34^2 + 2880}}{8} = \frac{57 - 19{\cdot}34}{8} = \frac{37{\cdot}66}{8} = 4{\cdot}71'$$

say, 4′-9″

∴ Area of concrete $= 12'\text{-}6'' \times 16'\text{-}0'' = 200$ ft.^{2} (See Fig. 12).

Now, if the stanchion in above example is subject to a B.M. of 100 ft. tons, as well as to direct load, the design has to be modified. First, find force in H.D. bolts. For base 3′ square, take bolt spacing at 2′-6″ and use four bolts. Then force per pair of bolts = 100/2·5−400/2 = 40−200 = −160 tons. Use nominal section say 1″ diameter. Increase length of grillage beams to 8′-0″ to allow for B.M.

$$\text{Direct pressure} = \frac{400}{8 \times 3} = 16{\cdot}67 \text{ tons per sq. ft.}$$

$$\textit{Modulus of base} = \frac{3 \times 8^2}{6} = 32 \text{ ft.}^3$$

Bending pressure = 100/32 = ± 3·125 tons per sq. ft.
Maximum pressure = 19·795 ,, ,,
Minimum pressure = 13·545 ,, ,,

(See Fig. 13). Maximum shear per beam :—

$$2{\cdot}5 \times \frac{17{\cdot}845 + 19{\cdot}795}{2} \times \frac{3}{5} = 28{\cdot}23 \text{ tons.}$$

Maximum B.M. per beam :—

		ft. tons
$16{\cdot}67 \times 3/5 \times 4^2/2$		= 80
$3{\cdot}125 \times 3/5 \times 4^2/3$		= 10
		90
− $\{200 \times 3/4 \times 1/5$	= 30	
$\;\;22{\cdot}22 \times 3/5 \times 1{\cdot}5^2/3$	= 10	
	—	40
		50

Modulus required = 50/10 × $\frac{3}{4}$ × 12 = 45 in.³

Use 12″ × 6″ × 44 lbs. (Z = 52·79 shear value = 31·2 × 4/3 = 41·6 tons) and web stiffeners under baseplate.

If D is same as before, *i.e.*, 4′-9″, then area of concrete = 12′-6″ × 17′-6″.

$$\text{Direct pressure} = \frac{400}{12{\cdot}5 \times 17{\cdot}5} = 1{\cdot}83 \text{ tons per sq. ft.}$$

$$\text{Modulus of base} = \frac{12{\cdot}5 \times 17{\cdot}5^2}{6} = 640 \text{ ft.}^3$$

Bending pressure = 100/640 = ±0·16 tons per sq. ft.
Maximum pressure = 1·99 ,, ,,
Minimum pressure = 1·67 ,, ,,

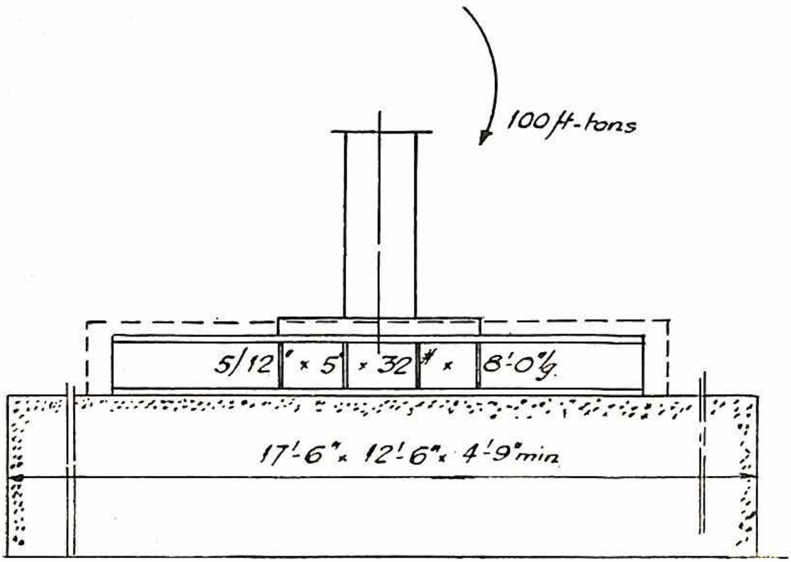

Fig. 13.

Two-Tier Grillage.

For heavy loads it may be necessary to adopt a grillage consisting of two or more tiers of beams. The two-tier grillage is designed in a similar manner to those already dealt with. The width of the upper tier is determined by the size of the column base and the width of the lower tier is the same as the length of the upper tier. Take, for example, a stanchion carrying a load of 600 tons. Assume base plate 3′-6″ square. Using concrete as before and a subsoil pressure of 4 tons per square foot :—

Area of grillage = 600/20 = 30 sq. ft.
Area of concrete = 600/4 = 150 sq. ft.
Take grillage 6′-6″ square. (See Fig. 14).

Lower Tier.

Load per ft. run = 600/6·5 tons.
Maximum shear = 600/6·5 × 1·5 = 139 tons.
Maximum B.M. = 300 (6·5/4—3·5/4 = 225 ft. tons.

$$\text{Section modulus required} = \frac{225 \times 12}{10 \times 10} \times \frac{3}{4} = 20{\cdot}25 \text{ in.}^3$$

10″ × $4\frac{1}{2}$ × 25 lbs. has section modulus = 24·47 in.^{3}
Shear per beam = 139/10 = 13·9 tons.
Shear value of 10″ × $4\frac{1}{2}$″ beam = 19·5 ,,
+$33\frac{1}{3}$% = 6·5 ,,
26·0 ,,
Check web buckling. Load = 60 tons.

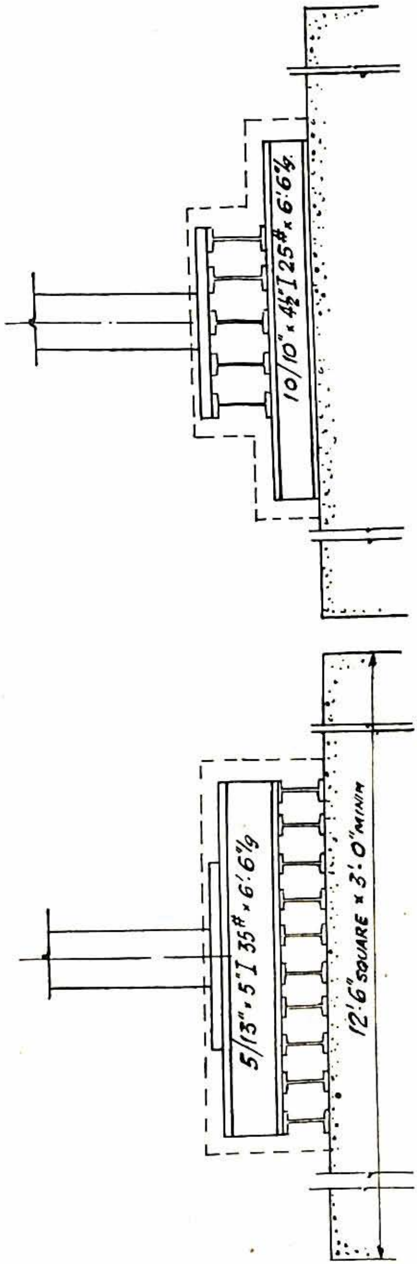

Fig. 14.

Safe load $= (9{\cdot}85 + 42 \times 1{\cdot}97)\ 4/3 > 60.$
Safe beam value $= (6{\cdot}72 + 42 \times 3{\cdot}6)\ 1\frac{1}{3} > 60.$
$\therefore$ No stiffeners are required.

Upper Tier. Use 5 beams.

Max. shear per beam $= 120/6{\cdot}5 \times 1{\cdot}5 = 180/6{\cdot}5 = 27{\cdot}6$ tons.
Max. B.M. per beam $= 60\ (6{\cdot}5/4 - 3{\cdot}5/4) = 45$ ft. tons.

$$\text{Section modulus required} = \frac{45 \times 12 \times 3}{10 \times 4} = 40{\cdot}5 \text{ in.}^3$$

13″ × 5″ × 35 lbs. has section modulus $= 43{\cdot}62$ in.³

Shear value $= 29{\cdot}57$ tons.
$+33\frac{1}{3}\% = 3{\cdot}86$,,

$39{\cdot}43$,,

$$\text{Space between joists} = \frac{42 - 5 \times 5}{4} = 4\tfrac{1}{4} \text{ ins.}$$

Web buckling: Load per beam = 120 tons.
Safe value $= (14{\cdot}23 + 42 \times 2{\cdot}19)\ 1\frac{1}{3} > 120$ tons.
Safe beam value $= (8{\cdot}93 + 42 \times 4{\cdot}20)\ 1\frac{1}{3} > 120$ tons.
$\therefore$ No stiffeners are required.

(Values for shearing, web buckling and direct bearing are taken from B.C.S.A. Safe Load Tables, pp. 52, 53, 1950).

Concrete area required = 150 sq. ft., say 12′-6″ square.
Assuming 45° spread, minimum depth of concrete

$$D = \frac{12'\text{-}6'' - 6'\text{-}6''}{2} = 3'\text{-}0''$$

Overturning moment can be treated for two-tier grillages as for single tier grillage (see previous example).

Type (3). Reinforced Concrete Foundations.

This type of foundation is commonly used, and in buildings it may replace the grillage foundation; in other words, the base plate may rest directly on a foundation of reinforced concrete. The R.C. may rest directly on the subsoil where the load is not excessive or where the bearing stratum is not too deep. In cases where the load is large or where the bearing stratum lies deeper, it may be an advantage to place an R.C. footing on a mass concrete block resting on the subsoil. (Both these types are suitable for R.C. columns).

Dealing with the former case, where the R.C. rests directly on the subsoil, the size of the base and effective height of the C.R. are determined by the bending stresses and the punching shear respectively. "Punching" shear is shear due to the tendency to

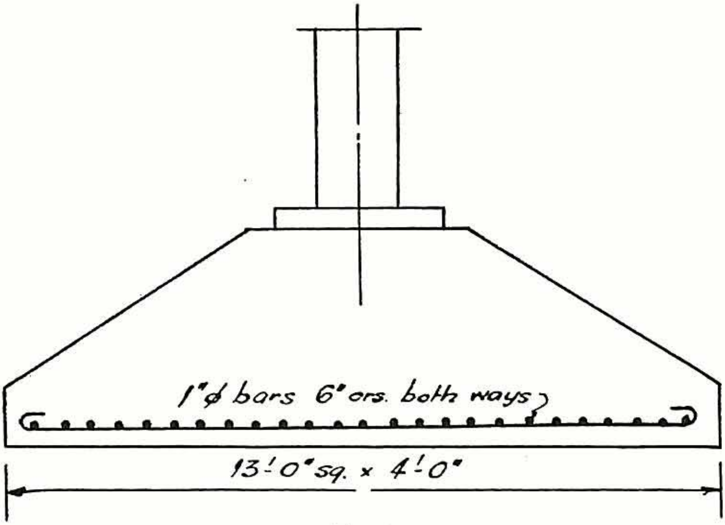

Fig. 15.

punch out a small portion of the base. Punching shear is generally taken as twice the value for ordinary shear for the grade of concrete used. For example, take a stanchion carrying a load of 400 tons with a base plate 3 ft. square and assume the bearing value of subsoil at $2\frac{1}{2}$ tons per square foot. Use 1 : 2 : 4 concrete and take working stresses as compression, $c = 750$ lbs/in.^{2}; shear 75 lbs/in.^{2}; punching shear 150 lbs/in.^{2} and tension in steel 18,000 lbs/in^2 and $m = 15$. The minimum area of foundation required is $400/2{\cdot}5 =$ 160 sq. ft., say 13 ft. square. The force tending to punch out the base is given by $W\,(1 - d^2/D^2)$, where W is the total load; D is the side of concrete base, and d the width of column (for R.C. column) or width of base plate in this case. Applying this formula, we get $400\,(1 - 3^2/13^2) = 400\,(160/169)$ tons $= 400 \times 2240 \times 160/169$ lbs. The perimeter of the base plate is $4 \times 36 = 144$ ins. and allowable stress is 150 lbs/in.^{2} Therefore, effective depth

$$= \frac{400 \times 2240 \times 160}{144 \times 150 \times 169} = 39{\cdot}4, \text{ say } 40\text{in.}$$

The effective depth is measured to the centre of the reinforcing steel. In this case we can make overall depth 4′-0″ (see Fig. 15). The "effective width" is 36″ + 2 (48″—say 4″) = 124″ = 10′-4″.

The depth must be checked for shear; taking a strip 12″ wide, then shear force $= 2{\cdot}5 \times 2240 \times 5 = 28{,}000$ lbs. The effective area resisting this vertical shear is then $\cdot 872 \times 44 \times 12 = 460$ in.^{2}.

$\therefore$ Unit shear $= 28{,}000/460 = 60{\cdot}5$ lbs/in.^{2}, which is in order.

Considering now the bending of the base, we get :—

B.M. per ft. width $= 5600 \times 5^2/2 \times 12 = 840{,}000$ in. lbs.
M.R. of concrete $= bcn/2 \times (d - n/3)$

$$\frac{12 \times 750 \times \cdot 385}{2} \times \left(44 - \frac{\cdot 385 \times 44}{3}\right)$$

$$= \underset{\sim}{Q}\ bd^2 \quad \text{where } \underset{\sim}{Q} = 125{\cdot}7$$
$$= 125{\cdot}7 \times 12 \times 44^2$$
$$= 2{,}950{,}000 \text{ in. lbs.,}$$

which is amply strong enough. The reinforcement required per foot width

$$= \frac{840{,}000}{18{,}000 \times \cdot 872 \times 44} = 1{\cdot}2 \text{ in.}^2$$

This can be increased in the ratio total width/effective width, which gives

$$1{\cdot}2 \times \frac{156}{124} = 1{\cdot}5 \text{ in.}^2$$

Referring to Table 3 (Appendix C), we find that 1″ bars at 6″ centres give area $= 1{\cdot}57$ in.^{2}, so we use 1″ bars at 6″ centres, both ways hooked at ends. (To develop full strength of concrete, we would require $\cdot 008 \times 12 \times 44 = 4{\cdot}21$ in.^{2}). Tables giving properties and various data for R.C. design, areas of bars are given in the tables in Appendix C, at the end of this pamphlet. (For fuller explanation of the use of these tables and design of R.C. structures, see pamphlet "Reinforced Concrete" in this series, by the same author).

Where concrete is in contact with the subsoil, reinforcing bars should have a cover of not less than 2″ to 3″, and it is usual to "blind" the subsoil with a layer of lean concrete before constructing the concrete base. The pyramidal shape, shown in Fig. 13, effects an economy in concrete, although it may require more shuttering, but the latter may be re-used where there are a number of similar bases. Where the column is subject to bending, the size of base must be increased and the design modified as shown in previous examples. Note that where the column is R.C., the main bars of the column should be turned round into the base of the concrete unless the depth of foundation is sufficient to develop the strength of the bars in bond. For practical reasons, it is better to break these bars above the top of the concrete and to provide the same number of bars in the base lapping with the main bars.

Where the load is larger and the subsoil level some distance below the ground level, it may be economical to construct an R.C. footing similar to the previous type, resting on a block of mass concrete. This applies to foundations of moderate depths, as the cost of timbering, sheet piling and pumping may cause a substantial rise in construction costs and it may be cheaper to use piling. Every problem in foundations must be investigated in the light of site conditions, relative costs of materials, labour, etc. As the design in this instance follows closely on that of the example immediately previous, it can be dealt with briefly. Take a stanchion 3′-6″ square at base, carrying 600 tons, mass concrete 1 : 6 mix, on subsoil capable of carrying $2\frac{1}{2}$ tons per sq. ft.

Then area of mass concrete = 600/2·5 = 240 sq. ft., say 16′ square.
Area of R.C. footing = 600/20 = 30 sq. ft., say 6′-6″ square.
Punching shear force = $600\ (1 - 3{\cdot}5^2/6{\cdot}5^2)$ = 453 tons.
Perimeter of base = 4 × 42 = 168 ins.

$$\therefore \text{ Effective depth required } = \frac{453 \times 2240}{168 \times 150} = 40 \text{ ins.}$$

say 3′-6″ deep overall.

$$\text{Shear stress } = \frac{20 \times 1{\cdot}5 \times 2240}{12 \times {\cdot}872 \times 40} = 160 \text{ lbs/in.}^2$$

As this is more than allowable, shear reinforcement is required.

$$\text{B.M. per ft. width } = \frac{20 \times 2240 \times 1{\cdot}5^2 \times 12}{2} = 605{,}000 \text{ in. lbs.}$$

$$\text{M.R. of concrete } = 125{\cdot}7 \times 12 \times 40^2 = 2{,}400{,}000 \text{ ,,}$$

$$\text{Reinforcement p. ft. } = \frac{605{,}000}{18{,}000 \times 40 \times {\cdot}872} = {\cdot}965 \text{ in.}^2$$

$\frac{3}{4}$″ bars at 5″ centres give an area of 1·06 in.^{2}, so we use these in both directions. The footing can be made either pyramidal or rectangular to save shuttering. As the footing is 6′-6″ square, and the mass concrete 16′-0″ square, then assuming 45° dispersion of load through the concrete, the minimum depth of the latter is $\frac{1}{2}$ (16′-0″ – 6′-6″) = 4′-9″, say 5′-0″. (See Fig. 16).

Type (4). Pier Foundations.

In cases where a column is subject to an overturning moment relatively large in relation to the direct load, and where the subsoil lies fairly near ground level, it may be an advantage to adopt this type of foundation.

It can be used where "made" ground of moderate depth overlies the bearing stratum. It consists essentially of a mass concrete pier with nominal reinforcement, the pier being rectangular in plan and increased in size at the bottom. Where the B.M. is large it may be the deciding factor in determining the size of the foundation. In order to effect economy in concrete, excavation and timbering, the resistance of the earth at each end of the pier may be taken into account. In any structure buried in the ground and subject to overturning, the tendency is to rotate about a certain point.

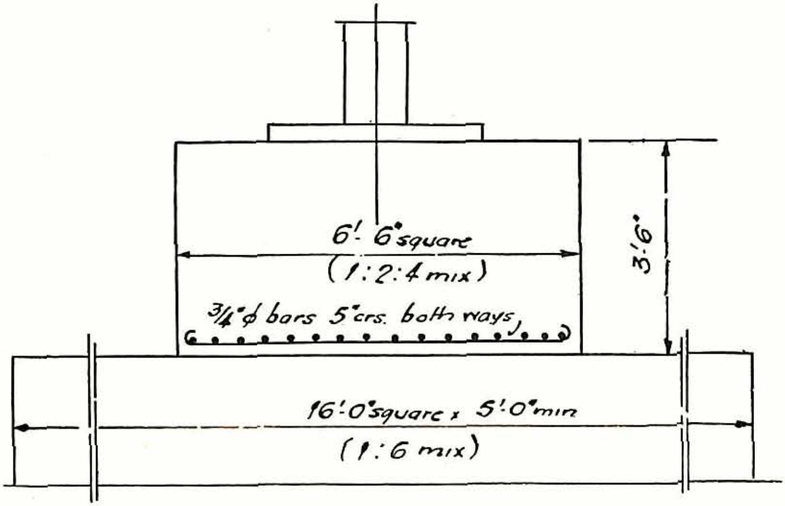

Fig. 16.

This tendency is resisted by the earth pressure, active on one side and passive on the other. These forces form a couple which acts against the applied moment. The moment acting at the underside of the pier is then the difference between the applied and the resisting moments. This "residual" moment should be used in calculating the maximum pressure on the subsoil. To illustrate the method, we shall take an example from the author's own experience. In designing a large workshop building, the following design conditions had to be met :—centres of main crane and roof stanchions 65′ × 30′. Live load—from No. 2, 50 ton cranes and wind loading. The foundation stratum was Thames ballast, good for 4 tons per square foot, lying at a depth of about 8-10 ft. below rail level. It was therefore decided to adopt a pier type of foundation.

The maximum direct load was 148 tons, and the B.M. due to wind and live load 36·5 ft. tons (see Fig. 17). The active and passive resistance can be calculated, knowing w the weight of earth per cubic foot and ϕ the angle of repose, by any of the recognised formulae.

In this case, the values were $w = 100$ lbs/ft.³. $\phi = 30°$.

The minimum depth for a pressure of 4 tons per sq. ft.

$$= \frac{4 \times 2240}{W} \times \frac{1}{\left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2} = \frac{8960}{100} \times \frac{1}{9} = 10 \text{ ft.}$$

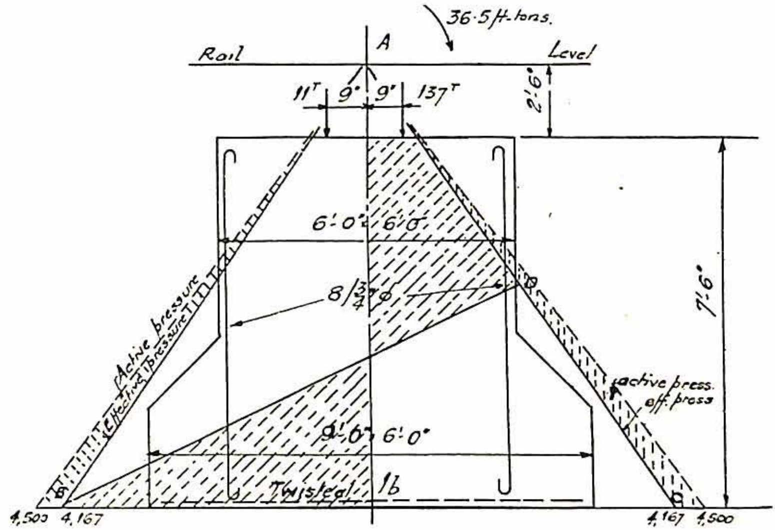

Fig. 17.

$$\text{Minimum depth} = \frac{W}{A \times w} \left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2 *$$

Referring to Fig. 15, the lines of pressure AB, AC represent the difference between the active pressure $wh\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$ and the passive pressure $1{\cdot}5 \times wh\left(\frac{1+\sin\phi}{1-\sin\phi}\right)$.† By trial and error method, a line BD is drawn so that the shaded areas are equal and represent pressures resisting overturning.

Then total pressure $= 4167 \times 6 \times 3/2 = 37{,}500$ lbs. $= 16{\cdot}8$ tons.
Lever arm $= 4$ ft. Moment $= 16{\cdot}8 \times 4 = 67{\cdot}2$ ft. tons.
∴ Nett moment $= 36{\cdot}5 + (126 \times {\cdot}75) - 67{\cdot}2 = 63{\cdot}8$ ft. tons.

$$\text{Weight of foundation} \begin{cases} 9 \times 6 \times 2 = 108 \\ 6 \times 6 \times 4 = 144 \\ 45 \times 1{\cdot}5 = 67{\cdot}5 \end{cases}$$

319·5, say 320 cub. ft.

* Where $\frac{W}{A}$ = pressure on subsoil (lbs/ft.^{2}).
W = weight of earth (lbs/ft.^{3}).
See Rankine Theory of Earth Pressure.

† 1·5 is adopted as a factor of safety as the passive resistance may not be fully developed until movement takes place.

320 cub. ft. = 20 tons. $\therefore$ Total load = 168 tons.

Direct pressure $= \dfrac{168}{9 \times 6} = 3{\cdot}12$ tons/ft.^{2}

Section modulus of base $= \dfrac{6 \times 9^2}{6} = 81$ ft.^{3}

Pressure due to overturning = 63·8/81 = +0·79 tons/ft.^{2}
Maximum pressure = 3·91 ,,
Minimum pressure = 2·33 ,,
Maximum moment on pier = 36·5 + (126 × ·75)
= 131 ft. tons.

Area of reinforcement required $= \dfrac{131 \times 12 \times 2240}{{\cdot}87 \times 70 \times 18{,}000}$

= 3·18 in.^{2}

Use 8 $\frac{3}{4}''$ diam. bars. Area = 3·52 in.^{2}

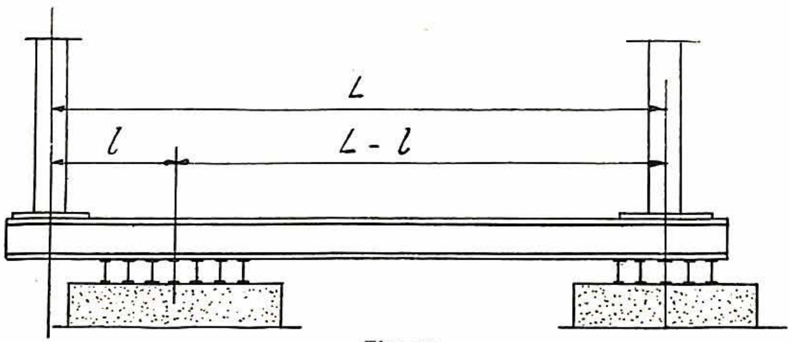

Fig. 18.

Type (5). "Bridge" and Cantilever Foundations.

"Bridge" foundations are used in buildings where it is impossible to place a foundation directly under a stanchion. This may be due to the presence of a subway, duct or similar obstruction. This case also occurs when using existing foundation blocks which do not coincide with the stanchions. The "bridge" may be composed of joists cased in concrete or an R.C. beam. Whichever method is used the B.M. $= Wab/l$ (or $Wl/4$), where W = column load, l = centres of foundation blocks and a, b the distances from column to blocks. The value in brackets is when W is at mid-span. If the bridge is composed of joists, then the working stresses can be taken in accordance with the rules laid down in B.S.S. 449, *i.e.*, 9 to 11 + t tons/in.^{2} (t = concrete cover over top in inches). In dealing with this type of foundation, it is necessary to ensure that the deflection is not excessive. Where the load is at mid-span the deflection is given by $\delta = Wl^3/48\,EI$. In estimating I, the moment of inertia, we must use the equivalent M.I. in concrete units, *i.e.*, for the R.C. bridge we must replace the area of the steel by $(m-1)$ times its area and in the case of the composite section we must find the M.I. in a similar manner, allowing for the concrete in compression. The value of E_c for concrete varies a good deal, and it is as well to be conservative in assessing its value. The usual value for 1 : 2 : 4 concrete is 2,000,000 lbs/in.^{2} and c = 750 lbs/in.^{2}.

"Cantilever" foundations are often used where a column or row of columns lies too near to adjoining property to have a proper foundation. In this case it is usual to place the column at the end of a cantilever beam composed of steel joist in concrete or an R.C. beam (see Fig. 18). The actual foundation block is placed as near the boundary as possible, and the load on it is given by WL/l. The other end of the cantilever beam carries another column which acts as kentledge and resists any tendency to uplift. The maximum B.M. on the beam is $= w\ (L-l)$, and the maximum deflection

$$\delta = \frac{W\ (L-l)^3}{3\,E\,I}.$$

The same remarks as to working stresses and inertias apply as in the case of "bridge" foundations. In this case the deflection is really the determining factor in fixing the size of the beam, as deflection must be restricted to a reasonable figure to avoid damaging plaster, etc.

Type (6). Combined and Raft Foundations.

Combined foundations can be used with advantage for carrying a number of point loads, *e.g.*, stanchions in cases where the spacing of the loads and the bearing value of the subsoil produce a condition where separate foundations would be almost touching each other. The combined foundation takes the form of a narrow continuous strip of concrete which behaves as an inverted R.C. beam with double reinforcement. So far as possible the centre of gravity of the foundation should coincide with the C.G. of the loads in order to reduce the pressure on the subsoil and the maximum B.M. and shear must be developed at each point along the beam. Take the loads and spacings as shown in Fig. 19 and the allowable pressure on subsoil as $1\frac{1}{2}$ tons per square foot.

Taking moments about L.H. load, we get

$$120 \times 20 = 2400. \qquad 100 \times 60 = 6000.$$
$$80 \times 40 = 3200. \qquad 60 \times 80 = 4800.$$

Total moment = 16,400 ft. tons. Total load = 460 tons.
Distance of C.G. from L.H. load = 16,400/460 = 35·6 ft.
Distance of C.G. of loads from central load C = 4·4 ft.
Take foundation as 90 ft. long and 4′-6″ wide.

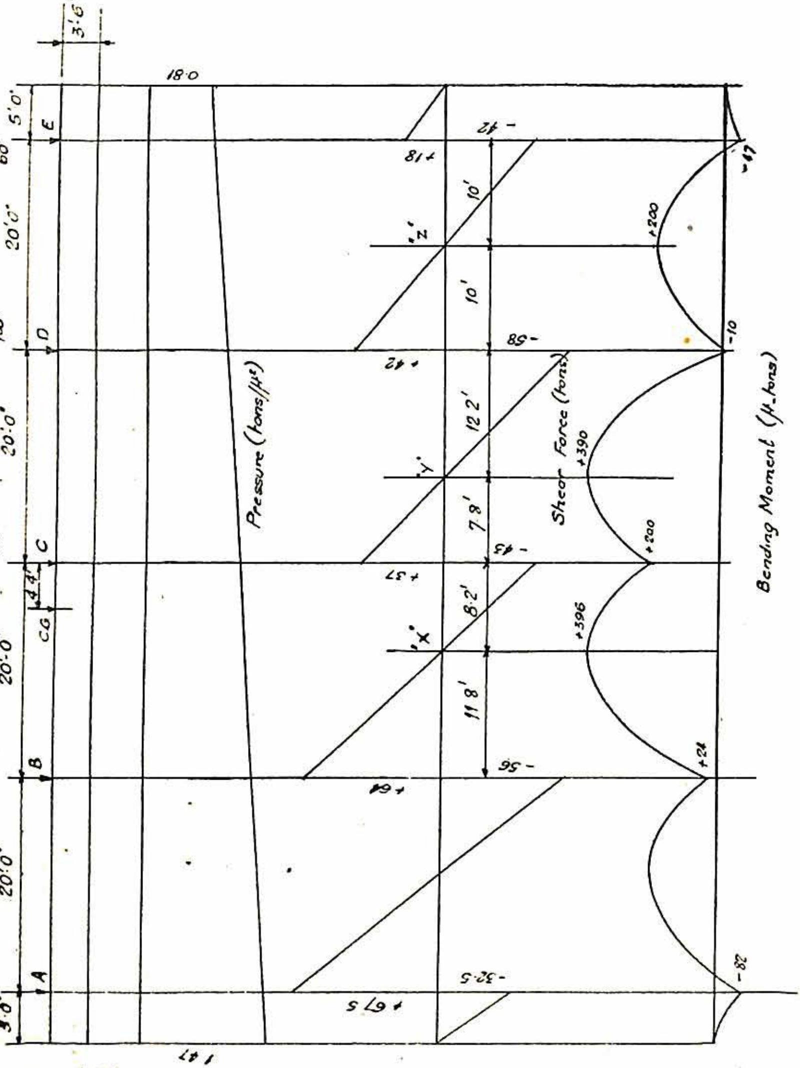
Pressure (tons/ft²)
Shear Force (tons)
Bending Moment (ft.-tons)
A
B
C
D
E
CG
"X"
"Y"
"Z"

Fig. 19.

Then direct pressure $= \dfrac{460}{90 \times 4{\cdot}5} = 1{\cdot}14$ tons/ft.^{2}

Modulus of foundation $= \dfrac{4{\cdot}5 \times 90^2}{6} = 6075$ ft.^{3}

Bending pressure $= \dfrac{460 \times 4{\cdot}4}{6075} = 0{\cdot}33$ tons/ft.^{2}

Maximum pressure $= 1{\cdot}47$ „
Minimum pressure $= 0{\cdot}81$ „

Shear Forces.

Negative shear at A $= \dfrac{1{\cdot}47 + 1{\cdot}433}{2} \times 4{\cdot}5 \times$
$= 32{\cdot}5$ tons.

Negative shear at B $= \dfrac{1{\cdot}47 + 1{\cdot}287}{2} \times 4{\cdot}5 \times 25 - 100$
$= 56$ tons.

Negative shear at C $= \dfrac{1{\cdot}47 + 1{\cdot}14}{2} \times 4{\cdot}5 \times 45 - 220$
$= 43$ tons.

Negative shear at D $= \dfrac{1{\cdot}47 + {\cdot}994}{2} \times 4{\cdot}5 \times 65 - 300$
$= 58$ tons.

Negative shear at E $= \dfrac{1{\cdot}47 + {\cdot}847}{2} \times 4{\cdot}5 \times 85 - 400$
$= 42$ tons.

Bending Moments. In ft.-tons.

	Negative.	Positive.	Nett.
At A $= \dfrac{1{\cdot}433 \times 4{\cdot}5 \times 5^2}{2}$	$= 80{\cdot}5$		
$+ \dfrac{0{\cdot}37 \times 4{\cdot}5 \times 5^2}{3}$	$= 1{\cdot}4$		
	81·9	Nil.	−81·9

At B	$= \frac{1 \cdot 287 \times 4 \cdot 5 \times 25^2}{2}$	$= 1805$	$100 \times 20 = 2000$	
	$+ \frac{0 \cdot 183 \times 4 \cdot 5 \times 25^2}{3}$	$= 171$		
		1976		+24
At C	$= \frac{1 \cdot 14 \times 4 \cdot 5 \times 45^2}{2}$	$= 5200$	$120 \times 20 = 2400$	
	$+ \frac{\cdot 33 \times 4 \cdot 5 \times 45^2}{3}$	$= 1000$	$100 \times 40 = 4000$	
		6200	6400	+200
At D	$= \frac{\cdot 994 \times 4 \cdot 5 \times 65^2}{2}$	$= 9400$	$80 \times 20 = 1600$ $120 \times 40 = 4800$	
	$+ \frac{\cdot 476 \times 4 \cdot 5 \times 65^2}{3}$	$= 3010$	$100 \times 60 = 6000$	
		12,410	12,400	−10

$$\text{B.M. at E} = - \cdot 81 \times 4 \cdot 5 \times 5^2/2 = -46 \cdot 7$$

$$- \frac{\cdot 037 \times 4 \cdot 5^2 \times 5^2}{6} = -0 \cdot 7$$

$$-47 \cdot 4$$

B.M.'s at Points of Zero Shear (in ft. tons).

		Negative.	Positive.	
At X	$\frac{1 \cdot 2 \times 4 \cdot 5 \times 36 \cdot 8^2}{2}$	$= 3650$	$120 \times 11 \cdot 8 = 1416$	
	$\frac{\cdot 27 \times 4 \cdot 5 \times 36 \cdot 8^2}{3}$	$= 550$	$100 \times 31 \cdot 8 = 3180$	
		4200	4596	+396 nett.
At Y	$\frac{1 \cdot 085 \times 4 \cdot 5 \times 52 \cdot 8^2}{2}$	$= 6750$	$80 \times 7 \cdot 8 = 624$ $120 \times 27 \cdot 8 = 3336$	
	$\frac{\cdot 385 \times 4 \cdot 5 \times 52 \cdot 8^2}{3}$	$= 1600$	$100 \times 47 \cdot 8 = 4780$	
		8350	8740	+390 nett.

At Z $\frac{\cdot 92 \times 4\cdot 5 \times 75^2}{2} = 11{,}600$ $\quad 100 \times 10 = 1000$

$80 \times 30 = 2400$

$\frac{\cdot 55 \times 4\cdot 5 \times 75^2}{3} = 4600$ $\quad 120 \times 50 = 6000$

$100 \times 70 = 7000$

16,200 per ft width. $\quad$ 16,400 $\quad$ +200 nett.

Max. shear = 67·5 tons = 15 tons = 33,600 lbs.
Max. B.M. = 396 ft. tons = 88 ft. tons = $88 \times 12 \times 2240$
= 2,400,000 *in.* lbs.

Using 1 : 2 : 4 concrete $\quad Q = 125\cdot 7$

$125\cdot 7 \times 12d^2 = 2{,}400{,}000$

$d = 40$ in.

Reinforcement per ft. $= 12 \times 40 \times \cdot 088$

$= 3\cdot 84$ in.^{2}

Use $1\frac{1}{8}''$ diam. bars at 3″ centres. For convenience, use same steel top and bottom. Make beam 42″ overall and blind subsoil with layer of lean concrete.

$$\text{Shear stress} = \frac{33{,}600}{12 \times \cdot 872 \times 40} = 80 \text{ lbs/in.}^2,$$ which is slightly more than permissible. Provide nominal shear steel, say $\frac{1}{4}''$ diam. stirrups at $12 \times 1\frac{1}{8} = 13\frac{1}{2}''$ centres. It should be borne in mind that any local settlement may produce redistribution of bending moments.

Raft Foundations.

On subsoils of low-bearing value, *e.g.*, made-up ground, it may be economical to provide a reinforced concrete raft to spread the load from a number of point loads as evenly as possible. The raft is like an inverted floor system consisting of a flat slab with stiffening ribs and main beams. The upward pressure from the subsoil must be carried by the raft in such a way that the reactions are the column loads. To a certain extent the design is approximate and empirical, but it must be remembered that the assumption on which the design is based, *i.e.*, that the subsoil is uniform, may not be correct, and therefore any error in design may be relatively small. So long as the raft is sufficiently stiff, any inequality can be distributed and uneven settlement minimised. Take the case shown in Fig. 20, and value of subsoil at half-ton per sq. foot.

Total load on subsoil	=	1120 lbs/ft.^{2}
Weight of slabs, ribs, etc.	=	220
∴ Net pressure	=	900

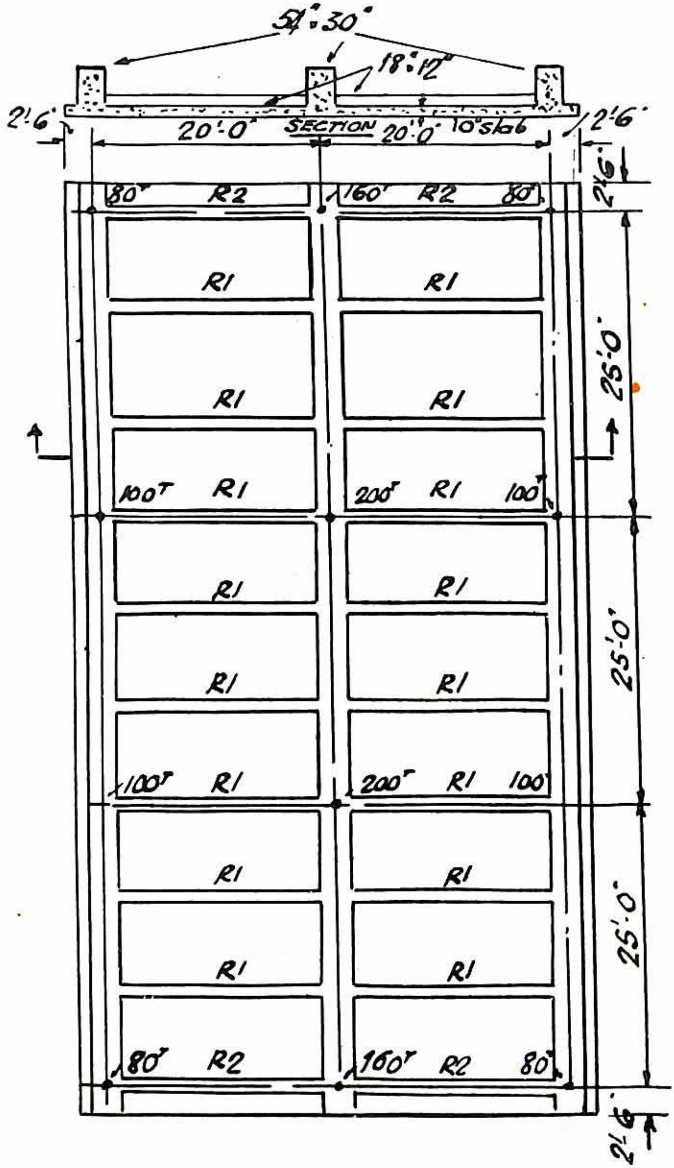

Fig. 20.

Total load on raft $= 1440$ tons

$$\therefore \text{ Area required} = \frac{1440 \times 2240}{900} = 3600 \text{ sq. ft.}$$

Let l = projection of cantilever portion, then

$$(40+2l)\ (75+2l) = 3600$$
$$3000+230\,l+4\,l^2 = 3600$$
$$4\,l^2 +230\,l - 600 = 0$$
$$l = 2{\cdot}5 = 2'\text{-}6''$$

Check area : $45 \times 80 = 3600$

Design of Slab.

Ribs at 8′-4″ centres, continuous slabs.

$$\text{B.M.} = \text{WL}/12$$
$$= \frac{900 \times 8{\cdot}33^2 \times 12}{12}$$
$$= 62{,}500 \text{ in lbs. per ft. width.}$$

Cantilever portion :—

$$\text{B.M.} = \text{W}_1\text{L}/2$$
$$= \frac{900 \times 2{\cdot}5^2 \times 12}{2} = 33{,}800 \text{ in. lbs. per ft. width.}$$

Max. B.M. in slab $= 62{,}500$ in lbs.
$= 125{\cdot}7 \times 12d^2$
$\therefore \quad d = 6{\cdot}5''$

For practical reasons we shall make the slab 10″ thick ; this is to provide a good flange for the main beams.

Reinforcement in slab per ft. $= {\cdot}008 \times 12 \times 8{\cdot}5$
$= {\cdot}896$ in.^{2}

$\frac{3}{4}''$ bars at 6″ centres give area $= {\cdot}884$ in.^{2}

Design of Ribs.

In this case we must make the assumption that the reactions from the ribs total the same amount as the column loads on the centre longitudinal beam.

Area carried by $R_1 = 40 \times 8{\cdot}33$ ft.^{2}
,, ,, $R_2 = 40 \times 6{\cdot}67$ ft.^{2}
$\therefore \quad R_2/R_1 = 6{\cdot}67/8{\cdot}33 = {\cdot}8$
$\therefore \quad R_2 = {\cdot}8\ R_1$
But $8\ R_1 + 2\ R_2 = 720$ tons
$\therefore \quad 9{\cdot}6\ R_1 = 720$,,
$\therefore \quad R_1 = 75$,, } centre
$R_2 = 60$,, } reactions
R_1 Centre reaction $= 75$ T

Upward pressure $= \dfrac{900 \times 8{\cdot}33 \times 40}{2240} = 136$ tons

Reaction of R_1 on outer longl. beams $= \dfrac{136 - 75}{2} = 30{\cdot}5$ tons

Max. B.M. on $R_1 = + \dfrac{75 \times 40}{4} = +750$ ft. tons

$- \dfrac{136 \times 40}{8} = -680$,,

+ 70 nett.

The rib acts as a T-beam in conjunction with the slab. Effective width of flange = centres of ribs

$= 8'\text{-}4'' = 100''$

B.M. $=$ 70 ft. tons $= 1{,}880{,}000$ in. lbs.

$= 125{\cdot}7 \times 100 \times d^2$

$\therefore\ d = 12{\cdot}25''$

Make overall depth of rib 18″

$$A_t = \frac{1{,}880{,}000}{18{,}000 \times {\cdot}872 \times 15} = 8{\cdot}00 \text{ in.}^2$$

Use 8—1⅛″ diam. bars. Area $= 7{\cdot}95$ in.^{2}

R_2 Centre reaction = 60T

Upward pressure $= \dfrac{900 \times 6{\cdot}67 \times 40}{2240} = 107$ tons

Reaction on outer beams $= \dfrac{107 - 60}{2} = 23{\cdot}5$ tons

Maximum moments $+ \dfrac{60 \times 40}{4} = +600$ ft. tons

$- \dfrac{107 \times 40}{8} = -535$,,

Nett + 65 ,,

Make as R_1

Max. shear on ribs = 30·5 tons = 68,500 lbs.

Shear stress $= \dfrac{68{,}500}{12 \times 15 \times {\cdot}872} = 438$ lbs/in.^{2}

$\therefore$ Shear steel is necessary.

Value of 4—1⅛″ diam. bars bent up at 45°

$= 3{\cdot}97 \times 18{,}000 \times 0{\cdot}707 = 50{,}000$ lbs.

Bend bars up at ends and provide ½″ stirrups at 12″ centres throughout.

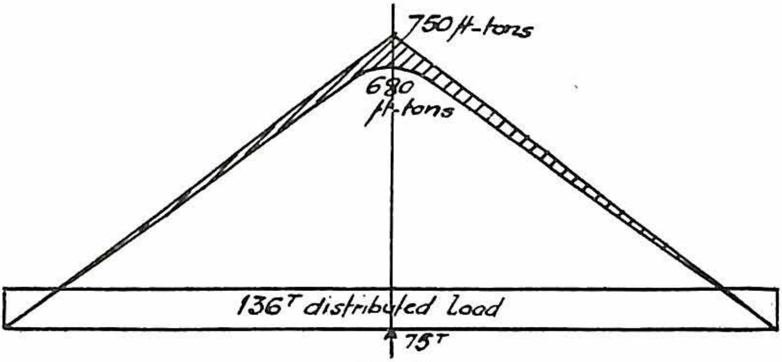

Fig. 21.

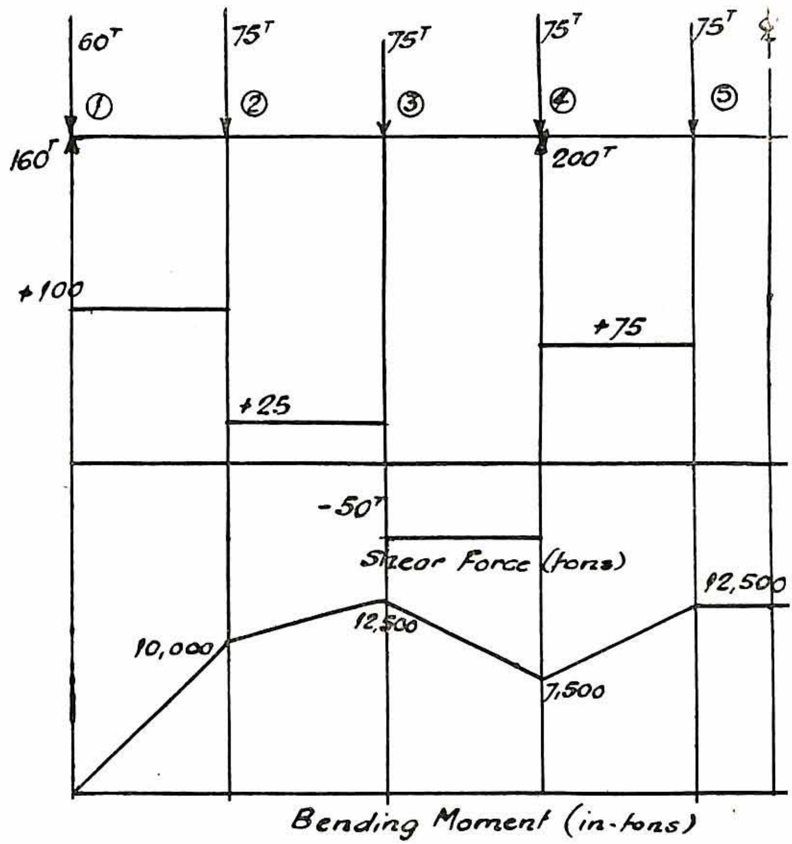

Fig. 21a.

Design of Centre Longitudinal Beam.

(See Fig. 21 (*a*).

Shear at (2)	=	160 − 60	=	100 tons
,, ,, (3)	=	150 − 100	=	50 ,,
,, ,, (4)	=	360 − 285	=	75 ,,
M_2	=	100 × 100	=	10,000 in. tons
M_3	=	100 × 200	=	20,000 ,,
	−	75 × 100	=	7,500 ,,
				12,500 ,,
M_4	=	100 × 300	=	30,000 ,,
		−75 (100 + 200)	=	22,500
				7,500 ,,

Maximum B.M. = 12,500 in. tons = 28,000,000 in. lbs.

Take effective breadth as 100″ and effective depth as 50″,

$n_1 = \cdot385 \times 50 = 19\cdot25''$

Then average stress in flange = 750/19·25 (19·25 − 10/2)
= 555 lbs/in.²

Compressive force in flange = 100 × 10 × 555
= 555,000 lbs.

Lever arm = 50 − 10/2 = 45″

Moment of resistance of concrete = 555,000 × 45
= 25,000,000

Moment to be taken up by compression steel
28,000,000 − 25,000,000 = 3,000,000 in. lbs.

Stress in compression steel = 555 × 14
= 7770 lbs/in.²

Area of compression steel $= \dfrac{3{,}000{,}000}{45 \times 7770} = 8\cdot55$ in.²

Use 12—1″ diam. bars. Area = 9·4 in.²

$A_t = \dfrac{28{,}000{,}000}{45 \times 18{,}000} = 35$ in.²

Use 12—2″ diam. bars. Area = 37·49 in.²
Make rib 54″ × 30″ overall.
Max. shear = 100 T = 224,000 lbs.

Shear stress $= \dfrac{224{,}000}{30 \times \cdot872 \times 50} = 170$ lbs/in.²

Allowable shear stress $= 75 \times \dfrac{75}{170} = 32\cdot5$ lbs/in.²

$$\text{Shear taken by concrete} = \frac{32 \cdot 5}{170} \times 224{,}000$$

$= 43{,}000$ lbs.

Shear to be taken by steel $= 181{,}000$ lbs.

Shear taken by 6—2″ diam. bars, bent up at 45°

$= 6 \times 3{\cdot}14 \times 18{,}000 \times 0{\cdot}707 = 240{,}000$lbs.

Shear taken by 12—½″ diam. stirrups at 12″ pitch

$$= \frac{18{,}000 \times 12 \times \cdot 196 \times \cdot 872 \times 50}{12}$$

$= 154{,}000$ lbs.

Provide stirrups throughout and bend up bars as necessary.

The outer longitudinal beams can be designed in a similar manner, bearing in mind the upward pressure on cantilever portion. It will probably be that the outer beams can be made the same as the centre beam to simplify detail and construction. Space does not permit of more detailed treatment of this subject.

Type (7). Piled foundations are used where the bearing subsoil lies at some depth below the surface and where the cost of timbering and pumping renders the adoption of any form of concrete foundation out of the question. Piling can be of various kinds :—(*a*) Timber, (*b*) Concrete (pre-cast or in situ), (*c*) Screw, (*d*) Sheet (this will be dealt with under Type 8), (*e*) Sand, and others.

Dealing with (*a*), it can be said that timber piling has been in use for some thousands of years, *e.g.*, the lake dwellings found on the Continent and elsewhere. Generally timber piles are of square section—12″ × 12″, 14″ × 14″, etc. Timber piles have been commonly used in the past for bridge foundations, piers, wharves, jetties and other structures. They are still used largely for temporary works during the course of construction. Timber piles in certain waters are subject to attack by marine creatures, such as teredo navalis and limnoria. This may be prevented by charring or other treatment. The present tendency is to substitute R.C. piles, except for temporary works. It is when a timber pile is subject to alternate wetting and drying, *e.g.*, in tidal waters, that decay takes place most rapidly.

It is interesting to note that the piles removed from under the piers of the old Waterloo Bridge were in perfect condition after over 100 years' immersion, in fact, some of the wood (elm) has been used for panelling, near London. Piles can be driven either by gravity hammers (*i.e.*, falling weight) or by steam hammers (single or double acting). They must be protected by helmets at the top during driving and the type of shoe depends entirely on the nature of the strata to be penetrated. The bearing capacity of a pile is given by formulae which are based on the average penetration during the last 10-20 blows, the weight of the hammer and its fall. Formulae are given in Appendix A. The bearing value of a pile depends on the resistance at the toe and also on the skin friction existing between the surface of the pile and the strata passed through.

(*b*) Concrete piles are of two types—pre-cast and in situ. Both types have their advantages, but on the whole the pre-cast type is preferable. This consists of a square section generally reinforced with at least one steel rod at each corner. The stresses during driving are complex and the spiral or secondary steel should be closely spaced near each end. Fork spacers should also be provided to keep the main reinforcement in place and lifting holes are necessary for handling purposes. These are generally cored holes at 1/7 length from each end to minimise bending stresses. The type of shoe depends on the nature of the strata. Where piles are to be driven in water or through water-logged strata, care should be taken to obtain as dense a concrete as possible, with ample cover to the reinforcement, and, where the water contains harmful salts, it may be necessary to use special cement. Piles are cast horizontally and should be allowed to mature as long as possible before driving. Piles are generally driven in pairs or in groups. Where a number of piles are driven close together, it is as well to reduce the load per pile as the driving of adjacent piles tends to disturb the strata and therefore reduce the skin friction. After driving, the concrete at the head of the piles is stripped away, leaving the steel exposed. The steel is then embedded in the pile capping or slab. The pile capping should be thick enough and have enough reinforcement to develop the shear and bending moment. In order to illustrate the principles involved, we shall take a case from the author's experience. The problem was to carry the stanchions of a large transit shed. Borings revealed that the site was over made ground to a depth of 10 feet. It was decided to carry each stanchion on a group of three piles and eventually over 200 piles were driven for this purpose. The direct load per stanchion was 20 tons and the overturning moment was 36·27 ft. tons.

Total load per 3 piles,	20 tons
Add weight of concrete,	4 ,,
	24 tons

$$\text{Max. load per pile} = 24/3 + 36{\cdot}27/4{\cdot}5$$
$$= 8 + 8{\cdot}06 = 16{\cdot}06 \text{ tons}$$

Pile 12″ × 12″ × 30′-0″. Weight of hammer = 2 tons

Height of drop = 4 ft. Weight of pile $= \dfrac{30 \times 144}{2240} = 2$ tons

Specified penetration—3″ in last 10 blows.

$$P/W = 2{\cdot}5/2 = 1{\cdot}25$$

Hiley Formula.

Effective drop = 48″. S = 3/10 = ·3. c = ·31. n = ·48.

$$R = \frac{\cdot 48 \times 2 \times 48}{\cdot 3 + \cdot 155} = 101 \text{ tons}$$

$$R_2 = 101 + (2 + 2\cdot 5)$$
$$= 105\cdot 5 \text{ tons}$$

$$L_w = \frac{105\cdot 5}{3} - 2\cdot 5 = 35\cdot 2 - 2\cdot 8 = 32\cdot 4 \text{ tons}$$

Dutch Formula.

$$W_1 = \frac{w\,H\,\eta}{C\,(1 + R)}$$
$$= 28 \text{ tons}$$

As both these values are greater than the maximum load, the design is safe.

As maximum load is less than either of these values, the design is safe.

Max. shear	= 16·06 tons	= 36,000 lbs.	
Max. B.M.	= 16·06 × (36—16)	= 321·2 in. tons	
	= 720,000 in. lbs.	= 125·7 bd^2	
Area required for shear	= 36,000/75	= 480 in.^{2}	
Actual area	= 33 × ·872 × 21	= 600 in^2.	O.K.
The moment of resistance	= 125·7 × 51 × 21^2	= 2,800,000 in. lbs.	
	(See Fig. 22).		O.K.

Space does not permit more detailed treatment, but reference can be made for more information to publications mentioned in bibliography.

In situ piles are of several types and fuller information can be obtained by application to the firms specialising in such work. One type is formed by driving a steel tube into the ground, afterwards lowering a "cage" of reinforcement and filling up with concrete as the tube is withdrawn. Another is formed by threading pre-cast concrete rings on a collapsible mandrel, afterwards proceeding as before. The most favourable cases for the use of in situ piles is where vibration is not permissible owing to danger to adjacent structures; where restricted headroom prevents the erection of a piling frame or where the depth of bearing stratum is unknown. It is as well to apply a test load to this type of pile, as the resistance to driving does not give an indication of the bearing value as in the case of the pre-cast pile. At the same time, the author has known cases where the use of pressure piles has been advantageous in view of site conditions. It is interesting to note that the pre-cast piles can be lengthened where necessary.

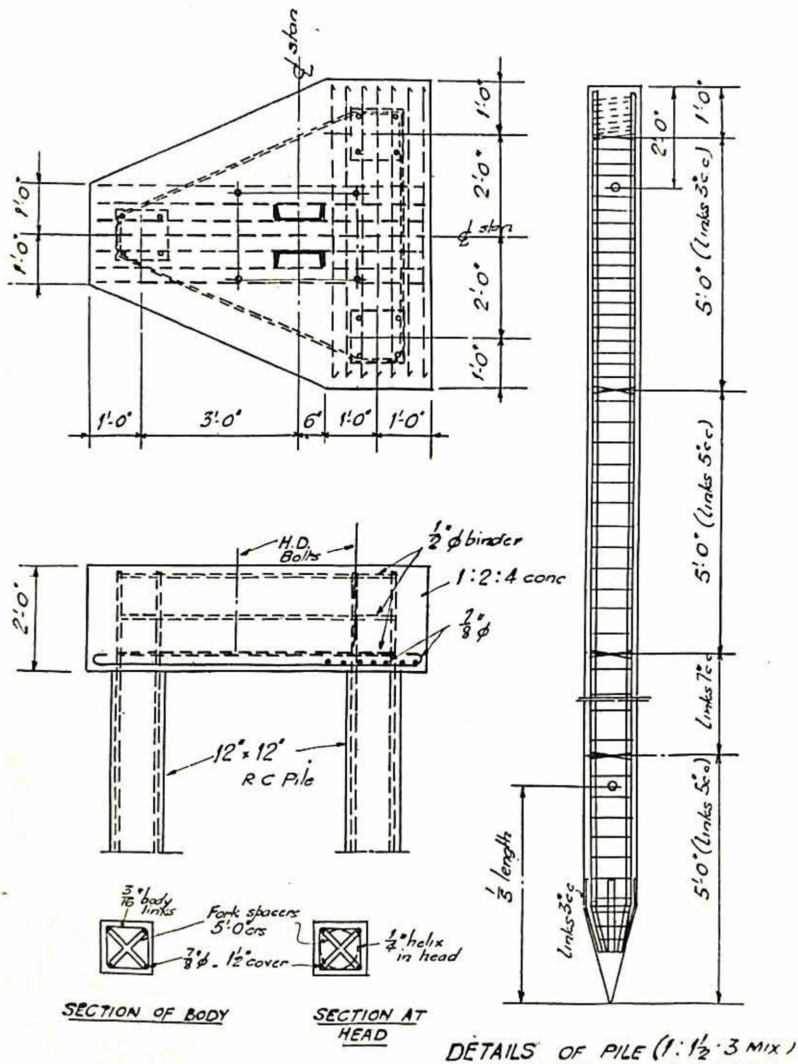
℄ stan
1'-0"
3'-0"
6"
1'-0"
1'-0"
2'-0"
H.D. Bolts
½" φ binder
1:2:4 conc
⅞" φ
12" × 12" R C Pile
3/16" body links
Fork spacers 5'-0" crs
⅞" φ - 1½" cover
¼" helix in head
SECTION OF BODY
SECTION AT HEAD
5'-0" (links 3" c.c.)
5'-0" (links 5" c.c.)
Links 7" c.c.
5'-0" (links 5" c.c.)
links 3" c.c.
⅓ length
DETAILS OF PILE (1 : 1½ : 3 MIX)

Fig. 22.

Screw piles are generally formed of steel tubes with a broad blade or screw at the bottom. They can be driven by a specially-designed screwing capstan and may be used for piers or jetties where the bearing value of the subsoil is low. They can be braced by steel members. Where immersed in water and not accessible for painting, they are liable to corrosion unless made from alloy steel.

Sand piles are not common in this country, but have been used in special cases in America.

Type (8)—Bridge Piers. Where these can be constructed in the dry they present no special difficulty and one of the methods previously described can be used according to the site conditions. In practice, however, many cases occur where the presence of water necessitates either of two methods, viz. :—(*a*) cofferdam, or (*b*) caisson. Taking method (*a*), cofferdams are usually constructed of steel sheet piling driven vertically and supported by timbering. Steel sheet piling is manufactured by various steelmakers and particulars can be obtained from their handbooks. It may be in the form of joists with a special device for connecting each pair of joists, or in the form of interlocking trough-shaped sections specially rolled for the purpose. Sheet piling should be practically watertight, but any small leaks may be stopped up with puddled clay. (Sheet piling is also used for permanent works, such as river and quay walls). The horizontal timbers supporting the sheet piling are called walings and these are connected by timber struts and wedges. Some notes on the strength of timber struts and walings are given in Appendix B. The piling is designed as a continuous beam, spanning between the walings and subjected to hydrostatic pressure. When the piles have been driven and timbering erected, the procedure is to pump out any water and excavate to the proper depth. The concrete in the foundation is then deposited in sections and the timbering removed in stages. When the work is completed to above water level, the piles are extracted and can be re-used.

Fig. No. 23 shows the spacing of walings so that the loads are the same and the same section of waling can be used throughout.

(*b*) Caissons can be of two types, either open or closed. In the case of the open caisson or cylinder, it is lowered either by own weight or by kentledge until it reaches the required depth. The surplus material is removed by open grabbing, the water is pumped out and the concrete deposited. The closed caisson, which is generally used for large bridge piers, consists essentially of a steel box built of layers or "strakes" and having a heavy cutting edge at the bottom. The plates forming the outside are called the outer skin plating. A working chamber is provided, which is roofed

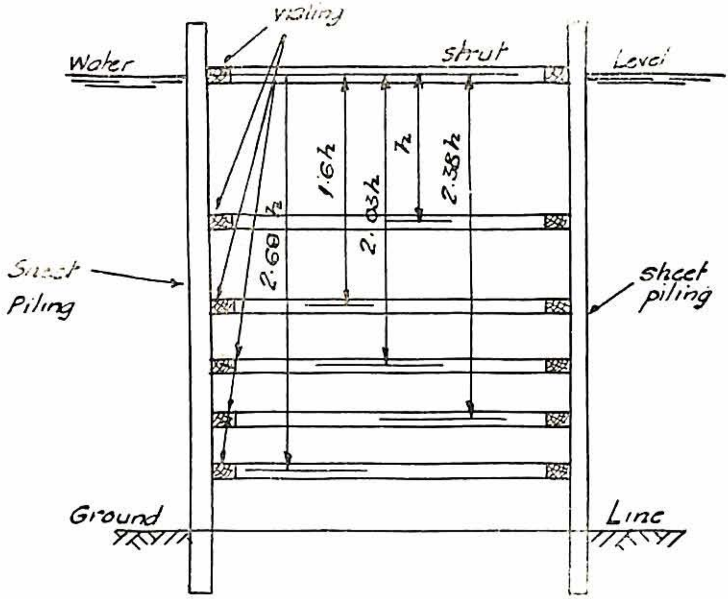

Fig. 23.

in by the inner skin plating. The access to the working chamber is gained by a steel tube provided with an air lock for the workmen and a similar tube for materials. Compressed air is passed down in order that the pressure in the working chamber is equal to the hydrostatic pressure.

It is essential to provide a decompression chamber for the workmen when entering or leaving the caisson, otherwise they are liable to "bends" or caisson disease. The time spent in the working chamber must be regulated according to the pressure. The procedure is to excavate the material in the working chamber and allow the caisson to sink under its own weight or that of kentledge until the required depth is reached. The working chamber is then filled with concrete and the work of completing the pier above the top of the caisson (if necessary by means of temporary strakes) is put in hand. Caissons are used where the depth of water makes piling impracticable. (See Fig. 24).

Type (9)—Machine Foundations. While it is rather difficult to generalise on this subject, it can be said that foundations must be large enough to spread the load on the subsoil and to resist any tendency to uplift. There should be a certain relation between the weight or inertia and the horse-power generated. Some in-

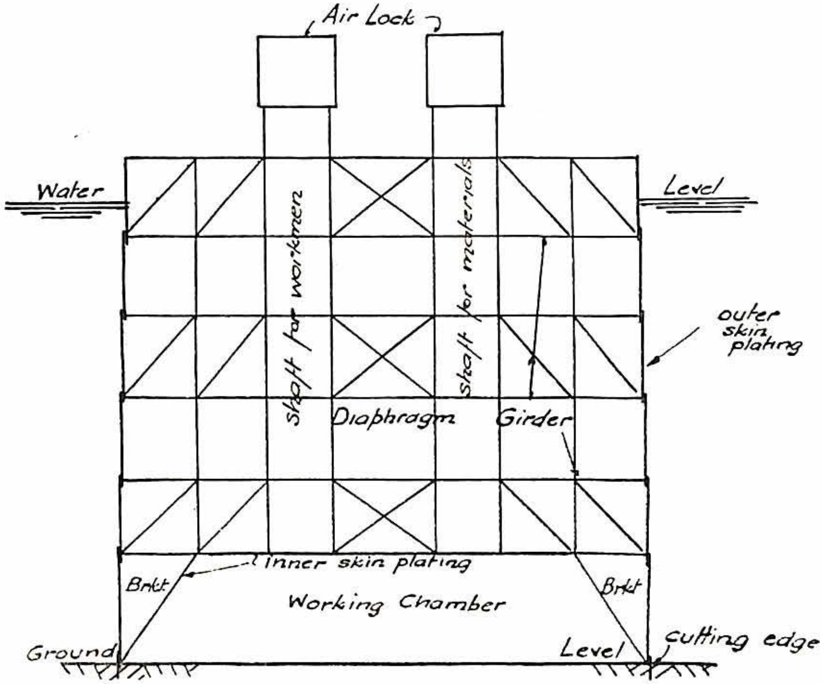

Fig. 24.

teresting data is given in Kent's *Mechanical Engineers' Handbook*, published by John Wiley & Co., New York. This gives a graph relating volume of foundation to the I.H.P. for steam engine. For turbines, the foundation should be rigid enough to limit the deflection to 0·02 inch. In the case of gas engines the depth of foundations should be 5 to 6 times the cylinder bore and the volume should vary with the B.H.P.

For oil engines the concrete should be 1 : 3 : 4 mix., and the weight of foundations not less than :—

2000	lbs. per B.H.P. for	single cylinder horizontal types,
1300	,, ,, ,,	multi-cylinder vertical types,
1750	,, ,, ,,	twin-cylinder vertical types.

and the depth not less than five times the cylinder bore.

In all cases the foundations should be provided with H.D. bolts, strong enough to resist uplift, and bedplates grouted up after final positioning and levelling. Care should be taken to isolate foundations on account of vibration, and they should be kept separate from the foundations and floors of the building. Insulation against

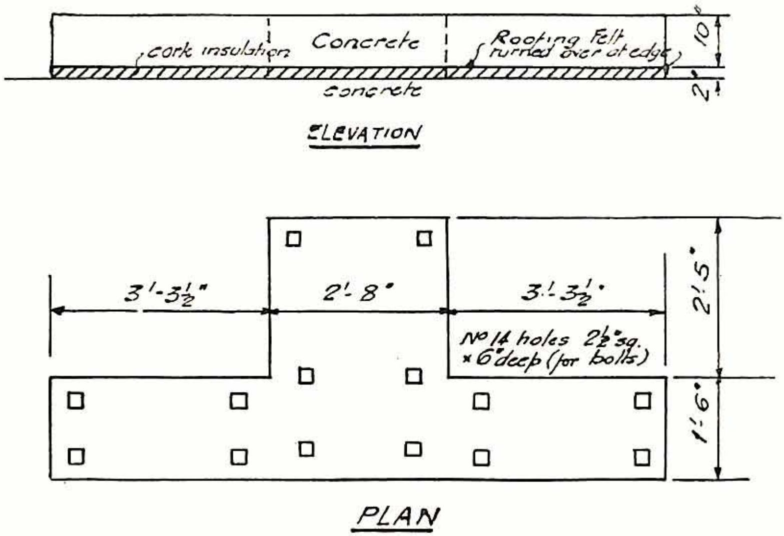

Fig. 25.

vibration can be done by means of cork, rubber, felt, sand or soft wood. Many makers of machinery prefer to design their own foundations to suit any particular case. Fig. 25 shows a typical foundation for ventilating plant.

Excavation and Timbering.

When a foundation has to be excavated, it is generally necessary to support the sides of the trench or pit with timbering. The amount of timbering depends on the depth, nature of the ground, and any adjacent dead and live loads. Usually the sides are covered with poling boards or timber sheeting, supported by longitudinal runners known as walings, which are connected by props or struts

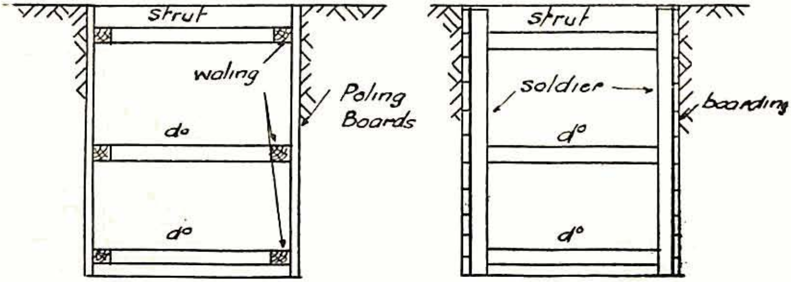

Fig. 26.

with the necessary wedges. Fig. 26 shows type details and some useful information will be found in Appendix B on the strength of timber struts and walings.

Conclusion.

It is obviously impossible in the space available to cover this subject in detail, and therefore a bibliography is attached, giving names of text-books, specifications, etc., for reference.

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APPENDIX A.

I. American Formula for Timber Piles.

(1) For gravity hammers. $P = \frac{2\,W\,H}{s + 1{\cdot}0}$

(2) ,, single-acting steam hammers. $P = \frac{2\,W\,H}{s + 0{\cdot}1}$

(3) ,, double-acting steam hammers. $p = \frac{2\,H\,(W + Ap)}{s + 0{\cdot}1}$

where P = safe bearing value in lbs.
W = weight of striking part in lbs.
H = height of fall in feet.
A = area of piston in square inches.
p = steam pressure in lbs/in.^{2}
s = average penetration in inches.
in last { 5-10 blows for gravity hammers.
10-20 ,, ,, steam ,,

This formula is applicable only if (1) the hammer has a free fall, (2) head is not "broomed" or crushed, (3) penetration is reasonably quick and uniform, (4) there is no sensible bounce after the blow. The safe load can be taken as half the test load which produces a settlement of not more than ¼″ after 48 hours.

II. Notes on Concrete Pile-Driving (reproduced by permission of Institution of Structural Engineers from "Specification for Concrete Pile-Driving—Model Clauses, with Explanatory Notes," 1936).

The Hiley pile-driving formula,

$$R = \frac{\eta\, wh}{s + c/2}$$

where R = ultimate resistance of the ground (tons) to further penetration by the pile (as caused by the hammer blow).
P = weight of pile which includes helmet and driving cap or anvil (tons).
W = weight of kinetic member or ram of hammer (tons).
η = efficiency of the blow which depends on the nature of the materials receiving impact and upon the ratio P/W.
H = the actual stroke of hammer or ram in inches.
h = height of free fall of ram in inches.

The value of h shall be

100% H for drop hammers released by a monkey trigger.
90% H for single-acting steam hammers.
80% H for drop hammer actuated by a wire rope from friction winch.

For double-acting steam hammers, $h = \dfrac{H\ (w + A \times M)}{W}$

where A = area of piston in square inches acted on by steam, and
M = mean effective steam pressure in the hammer cylinder.

Where single-acting hammers or drop hammers work in leader guides, inclined at an angle θ from the vertical, a further allowance must be made for the frictional resistance of the guides and for the reduced component of gravity acting along the direction of the guides and h_1 (which replaces h) $= h \times (\cos\theta - \mu \sin\theta)$.

s = "Set" per blow, being the permanent penetration of pile per blow in inches.

c = Temporary elastic compression of the pile and cap, and of the ground into which the pile penetrates, caused by the transmission of pressure corresponding to R.

The total resistance offered by the ground, allowing for the weight of hammer and pile is :—

$R_2 = R + (W + P)$, and
L_w = working load on pile.
$= R_2/Q - P$, where Q = factor of safety.

Table I.

Values of η for ratios of P/w.

Ratio P/W	R.C. Piles driven by double-acting hammers.	In Situ Steel Tubes, 16-in. dia., driven by single-acting hammer.	R.C. Piles fitted with Helmet.	R.C. Piles with dolly deteriorated.
			Driven by single-acting or drop hammer.	
½	0·75	0·70	0·69	0·67
1	0·63	·55	·53	·50
1½	·55	·46	·44	·40
2	·50	·40	·37	·33
2½	·45	·36	·33	·28
3	·42	·33	·30	·25

It is found from experiment that the coefficient of restitution denoted by e has a constant value for any pairs of substances. In the above table the values of e have been taken for the four cases, reading from left to right, as 0·5, 0·32, 0·25 and 0, and the value of η has been calculated from the following formula :—

Where the pile is driven into penetrable ground

$$\eta = \frac{W + Pe^2}{W + \phi}$$

For the special case where a pile point meets with refusal on impenetrable rock instead of using the full value of P in the above expression, 0·5 P is substituted throughout, which gives a higher value to η. It should be noted that in the Dutch formula,

$$\eta = \frac{W}{W + P}$$

which is the same as above for $e = 0$.

Table II.

Values of total temporary compression (c) in inches.

Length of Pile. Feet.	p=500 lbs/in.² Easy Driving.			p=1000 lbs/in.² Medium Driving.			p=1500 lbs/in.² Hard Driving.			p= 2000 lbs/in.² Very Hard Driving.		
	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(4)	(1)	(2)	(3)
10	0·16	0·25	—	0·21	0·41	—	0·27	0·57	—	0·27	0·67	—
20	·19	·28	—	·27	·47	—	·36	·65	—	·39	·79	—
30	·22	·31	—	·33	·53	—	·45	·74	—	·51	·91	—
33	—	—	0·186	—	—	0·272	—	—	0·358	—	—	0·394
40	·25	·24	—	·39	·59	—	·63	·83	—	·63	1·03	—
46	—	—	·204	—	—	·308	—	—	·412	—	—	·456
50	·28	·37	—	·45	·65	—	·63	·92	—	·75	1·15	—
54	—	—	·215	—	—	·322	—	—	·444	—	—	·512
60	·31	·40	—	·51	·71	—	·721	·01	—	·87	1·27	—

p = driving stress per sq. inch on projected area of shoe to give the force R.

(1) for R.C. piles with 1-inch sacking material on head.
(2) for R.C. piles fitted with helmet and dolly.
(3) for 16-in. standard steel tubes for "in situ" piles.

The force impressed on the pile head will always be in excess of the force R transmitted by the energy of the blow through the pile. For a driving force R calculated from the formula the force R_1 acting on the pile head will not be less than that given by

$$R_1 = R \times \frac{1}{M\eta}$$

This force on the head must show a stress over the sectional area of the pile well within its safe limit of endurance and should generally not exceed one-third of the crushing strength of the material.

Dutch formula (reproduced by permission from Appleby-Frodingham "Steel Sheet Piling").

$$W_1 = \frac{w\,H\,\eta}{C\,(1 + R)}$$

C varies from 4 to 6. The value of 6 should be taken for piles, subject to dead load only and driven by winch-operated drop hammer. If no dolly is used, then value $C = 4$ could be used. $C = 6$ for vibratory loads.

The table given below (worked out on this formula) assumes final set to be 1/10-inch per blow, and hammer drop to be 3 ft. $C = 4$ for maximum load. $C = 6$ for minimum load.

Table giving values of W_1 when w and P/w are known.

P/w	Values of *w*													
	½ ton		¾ ton		1 ton		1½ tons		2 tons		3 tons		4 tons	
	Min. W_1	Mx. W_1	Min. W_1	Mx. W_1	Min. W_1	Mx. W_1	Min. W_1	Mx. W_1	Min. W_1	Mx. W_1	Min. W_1	Mx. W_1	Min. W_1	Mx. W_1
½	—	—	—	—	40	60	60	90	80	120	120	180	160	240
1	—	—	23	34	30	45	45	68	60	90	90	135	120	180
1½	12	18	18	27	24	36	36	54	48	72	72	108	96	144
2	10	15	15	23	20	30	30	45	40	60	60	90	80	120
2½	9	13	13	20	17	26	26	39	34	51	51	77	68	102
3	8	12	12	17	15	23	23	34	30	45	45	68	60	90
3½	7	10	10	15	13	20	20	30	26	40	40	60	52	80
4	6	9	9	14	12	18	18	27	24	36	36	54	48	72

APPENDIX B.

Reproduced by permission from Appleby-Frodingham Steel Co., "Steel Sheet Piling."

Timber Struts.

Safe load in tons on square, well-seasoned pitch-pine struts. Based on safe stress of 1000 lbs/in.^{2} on section of short strut.

Factor of Safety = 6.

L'gth in feet.	Side in Inches.											
	4	5	6	7	8	9	10	11	12	13	14	15
4	6·2	10·0	14·9	20·8	27·3	34·1	43·4	52·6	63·0	...	...	...
6	5·3	9·1	13·8	19·5	26·1	33·5	42·0	51·3	61·5	72·7	84·8	97·7
8	4·6	8·2	12·5	18·1	24·6	31·9	40·2	49·3	59·7	70·9	82·8	95·5
10	4·0	7·2	11·5	16·8	23·0	30·3	38·4	47·5	57·6	68·8	80·3	93·2
12	3·5	6·6	10·4	15·4	21·3	28·4	36·5	45·3	55·3	66·0	77·6	90·5
14	...	5·8	9·4	14·1	19·9	26·5	34·6	44·0	53·0	63·4	75·3	87·9
16	...	5·2	8·5	13·0	18·4	25·2	32·7	41·1	50·5	61·0	72·6	85·0
18	...	...	7·9	12·3	17·2	23·4	30·7	38·9	47·7	58·4	70·0	81·9
20	...	...	7·2	11·0	16·0	22·0	28·8	36·8	45·9	55·9	67·0	79·0
22	...	...	...	10·5	15·0	20·9	27·3	34·9	43·7	53·5	64·3	76·1
24	...	...	...	...	13·9	19·4	25·8	33·1	41·6	51·1	61·6	73·2
26	...	...	...	...	13·0	18·2	24·4	31·5	39·6	48·8	59·1	70·3
28	...	...	...	...	...	17·3	23·1	30·1	37·8	46·7	56·6	67·5
30	...	...	...	...	...	16·2	21·8	28·3	36·2	44·9	54·3	64·9
32	...	...	...	...	...	...	20·8	27·0	34·3	42·8	52·3	62·6
34	...	...	...	...	...	...	19·6	25·9	33·0	40·9	50·0	60·3
36	...	...	...	...	...	...	...	24·6	31·4	39·3	48·2	58·0
38	...	...	...	...	...	...	...	...	30·1	37·5	46·7	56·0
40	...	...	...	...	...	...	...	...	28·8	36·2	44·4	53·8
42	...	...	...	...	...	...	...	...	...	34·5	42·8	52·0
44	...	...	...	...	...	...	...	...	...	...	41·3	50·0
46	...	...	...	...	...	...	...	...	...	...	39·7	48·2
48	...	...	...	...	...	...	...	...	...	...	...	46·9
50	...	...	...	...	...	...	...	...	...	...	...	45·2

If struts are of white pine, take $\frac{2}{3}$ of tabular loads.

If struts are rectangular with long side = a'', short side = b'', then find safe tabular load for a square strut with side = b'' and multiply by a/b. For round struts, take diameter as side of square strut and multiply S.T. load by 0·77.

Strength of Pitch-Pine Walings as Simply-Supported Beams.

Safe distributed loads in lbs. per beams 1″ wide.
For other widths, multiply tabular load × width in ins.

Factor of Safety = 6.

Span of Beam in feet	Depth of Beam in Inches (width = 1″)										
	6	7	8	9	10	11	12	13	14	16	18
7	800	1089	1422	1800	...	...	...	...	...	...	...
8	700	953	1245	1575	1945	...	...	...	...	...	...
9	622	847	1106	1400	1728	2090	...	...	...	...	...
10	560	762	996	1260	1556	1882	2240	...	...	...	...
11	509	693	905	1145	1414	1711	2036	2390	2772	...	...
12	467	635	830	1050	1296	1568	1867	2191	2541	...	...
13	431	586	766	969	1197	1448	1723	2022	2345	3063	...
14	400	544	711	900	1111	1344	1600	1878	2178	2844	3600
15	373	508	664	840	1037	1255	1495	1753	2033	2655	3360
16	...	476	622	788	972	1176	1400	1643	1906	2489	3150
17	...	448	586	741	915	1107	1318	1546	1793	2342	2965
18	...	...	553	700	864	1046	1244	1461	1694	2212	2800
19	...	...	524	663	819	991	1179	1384	1605	2096	2653
20	...	...	498	630	778	941	1120	1314	1524	1991	2520
21	...	...	...	600	741	896	1067	1252	1452	1896	2400
22	...	...	...	573	707	856	1018	1195	1386	1810	2291
23	...	...	...	...	676	818	974	1143	1326	1731	2191
24	...	...	...	...	648	784	933	1095	1270	1659	2100
25	...	...	...	...	622	753	896	1052	1219	1593	2016
26	...	...	...	...	...	724	862	1011	1173	1532	1938
27	...	...	...	...	...	697	830	974	1129	1475	1867
28	...	...	...	...	...	...	800	939	1089	1422	1800
29	...	...	...	...	...	...	772	907	1051	1373	1738
30	...	...	...	...	...	...	747	876	1016	1327	1680

If the walings are continuous over two or more spans, the safe distributed load can be increased in the ratio 12 : 8, but safe shear stresses must not be exceeded. The timber is stressed to 1400 lbs/in.^{2} (extreme fibre stress).

APPENDIX C.

REINFORCED CONCRETE.

TABLE 1.

Ordinary Concrete (m = 15 throughout).

Concrete mix.	Working stresses (lb/in.²)					Design Factors				
	Steel tension (t)	Concrete: Bending (c)	Concrete: Shear (s)	Concrete: Bond	Concrete: Comp.	t/c	n_1	a_1	Q	r
1:2:4	18,000	750	75	100	600	24·00	0·385	0·872	125·7	0·008
	20,000					26·67	0·360	0·880	119·0	0·00675
	25,000					33·30	0·312	0·896	105·0	0·00467
	27,000					36·00	0·294	0·902	99·4	0·00408
	18,000	1000	100	$B_1$120 $B_2$180	760	18	0·455	0·848	193	0·0126
	20,000					20	0·428	0·857	183·7	0·0107
	25,000					25	0·375	0·875	164	0·0075
	27,000					27	0·357	0·881	157	0·0066
1:1½:3	18,000	850	85	110	680	21·20	0·414	0·862	151·5	0·00975
	20,000					23·55	0·390	0·870	143·7	0·00827
	25,000					29·40	0·338	0·887	127·5	0·00575
	27,000					31·70	0·321	0·893	121·8	0·00505
	18,000	1250	115	$B_1$135 $B_2$200	950	14·4	0·511	0·830	264·4	0·0177
	20,000					16	0·483	0·839	253·2	0·0150
	25,000					20	0·428	0·857	229·9	0·0107
	27,000					21·6	0·411	0·863	221·7	0·0095
1:1:2	18,000	975	98	123	780	18·50	0·447	0·851	185·7	0·0121
	20,000					20·50	0·423	0·859	177·0	0·0103
	25,000					25·70	0·369	0·877	158·0	0·0072
	27,000					27·70	0·351	0·883	151·5	0·00633
	18,000	1500†	130	$B_1$150 $B_2$200	1140	12	0·556	0·815	339·6	0·0232
	20,000					13·33	0·529	0·824	328·7	0·0198
	25,000					16·67	0·473	0·842	299·6	0·0142
	27,000					18	0·455	0·848	289·2	0·0126

† Same values for High Alumina Cement Concrete, 1 : 2 : 4 mix.

For other values of t, m and c

$$\frac{t}{mc} = \frac{1}{n_1} - 1\,; \quad a_1 = 1 - \frac{n_1}{3}\,; \quad Q = \frac{c}{2} \times n_1 a_1\,;$$

$$r = \frac{c}{2} \times \frac{n_1}{t}$$

TABLE 2.

Vibrated Concrete (m = 15 throughout).

Concrete mix.	Working stresses (lb/in.^{2})					Design Factors				
	Steel tension	Concrete								
	(t)	Bending (c)	Shear (s)	Bond	Comp.	t/c	n_1	a_1	Q	r
1:2:4	18,000	1100	110	$B_1$132 $B_2$198	836	16·36	0·479	0·840	221·4	0·0146
	20,000					18·18	0·453	0·849	211·3	0·0124
	25,000					22·73	0·398	0·867	190	0·0088
	27,000					24·55	0·379	0·874	182·8	0·0077
1:1½:3	18,000	1375	127	$B_1$149 $B_2$220	1045	13·07	0·535	0·822	303	0·0204
	20,000					14·54	0·508	0·831	290	0·0175
	25,000					18·18	0·453	0·849	264·1	0·0124
	27,000					19·61	0·433	0·856	254·8	0·0110
1:1:2	18,000	1650	143	$B_1$165 $B_2$220	1254	10·91	0·578	0·807	385	0·0264
	20,000					12·11	0·553	0·816	372	0·0228
	25,000					1·515	0·498	0·834	342·4	0·0165
	27,000					16·35	0·478	0·841	332·0	0·0146

TABLE 3.

Concrete : Variable Values of m.

	Concrete Mix	Modular ratio m	Working Stresses (lb/in.^{2}): Steel Tension (t)	Concrete: Bending (c)	Concrete: Shear (s)	Concrete: Bond	Concrete: Comp	Design Factors: t/c	n_1	a_1	Q	r
Ordinary Concrete	1:2:4	18	18,000 20,000 25·000 27,000	750	75	100	600	24·00 26·67 33·30 36·00	0·43 0·405 0·351 0·333	0·86 0·865 0·883 0·889	138 131·3 116 111	0·009 0·0076 0·0053 0·0046
	1:1½:3	16	18,000 20,000 25,000 27,000	850	85	110	680	21·15 23·50 29·40 31·75	0·43 0·405 0·353 0·334	0·86 0·865 0·882 0·889	156 148·5 132·5 125·8	0·0101 0·0086 0·0060 0·0052
	1:1:2	14	18,000 20,000 25,000 27,000	975	98	123	780	18·48 20·55 25·65 27·70	0·43 0·404 0·354 0·336	0·86 0·865 0·882 0·889	180 170 152 145	0·0116 0·0098 0·0069 0·0061
High-Grade Concrete	1:2:4	14	18,000 20,000 25,000 27,000	950	95	120	760	18·97 21·05 26·35 28·45	0·425 0·400 0·347 0·333	0·858 0·867 0·884 0·889	173 165 145·5 140	0·0112 0·0095 0·0066 0·0059
	1:1½:3	12	18,000 20,000 25,000 27,000	1100	110	135	880	16·37 18·17 22·70 24·55	0·423 0·398 0·346 0·333	0·859 0·867 0·885 0·889	200 189·5 168·5 162·8	0·0129 0·0110 0·0076 0·0068
	1:1:2	11	18,000 20,000 25,000 27,000	1250	125	150	1000	14·40 16·0 20·0 21·60	0·433 0·408 0·356 0·338	0·856 0·864 0·881 0·887	231·5 220 196 187·5	0·015 0·013 0·009 0·008

TABLE 4.

Areas of Round Bars (in.²).

Bar Diam. (in.)	Number of Bars										Bar Diam. (in.)
	1	2	3	4	5	6	7	8	9	10	
1/4	0·049	0·098	0·147	0·196	0·245	0·294	0·343	0·392	0·441	0·491	1/4
5/16	0·076	0·153	0·230	0·306	0·385	0·460	0·536	0·613	0·690	0·767	5/16
3/8	0·110	0·220	0·331	0·441	0·552	0·662	0·772	0·883	0·993	1·104	3/8
7/16	0·150	0·300	0·450	0·601	0·751	0·901	1·052	1·202	1·352	1·503	7/16
1/2	0·196	0·392	0·588	0·785	0·981	1·177	1·374	1·570	1·766	1·963	1/2
9/16	0·248	0·497	0·745	0·994	1·242	1·491	1·739	1·988	2·236	2·485	9/16
5/8	0·306	0·613	0·920	1·227	1·534	1·840	2·147	2·454	2·761	3·068	5/8
11/16	0·371	0·742	1·113	1·484	1·856	2·227	2·598	2·969	3·340	3·712	11/16
3/4	0·441	0·883	1·325	1·767	2·209	2·650	3·092	3·534	3·976	4·418	3/4
13/16	0·518	1·037	1·555	2·074	2·592	3·111	3·629	4·148	4·665	5·185	13/16
7/8	0·601	1·202	1·803	2·405	3·006	3·607	4·209	4·816	5·411	6·013	7/8
15/16	0·690	1·380	2·070	2·761	3·451	4·141	4·832	5·522	6·212	6·903	15/16
1	0·785	1·570	2·356	3·142	3·927	4·712	5·497	6·285	7·068	7·854	1
1 1/8	0·994	1·988	2·982	3·976	4·970	5·964	6·958	7·952	8·946	9·940	1 1/8
1 1/4	1·227	2·454	3·681	4·908	6·136	7·363	8·590	9·817	11·044	12·272	1 1/4
1 3/8	1·484	2·969	4·454	5·939	7·424	8·909	10·394	11·879	13·364	14·849	1 3/8
1 1/2	1·767	3·534	5·301	7·068	8·835	10·602	12·369	14·136	15·903	17·671	1 1/2
1 5/8	2·073	4·147	6·221	8·295	10·369	12·443	14·517	16·591	18·665	20·739	1 5/8
1 3/4	2·405	4·810	7·215	9·621	12·026	14·431	16·837	19·242	21·647	24·053	1 3/4
1 7/8	2·761	5·522	8·283	11·044	13·806	16·567	19·328	22·089	24·850	27·612	1 7/8
2	3·142	6·283	9·424	12·566	15·708	18·849	21·991	25·132	28·274	31·416	2

TABLE 5.

Areas of Round Bars (in in² per ft. width).

Bar Diam. (in.)	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	10	11	12
3/16	0·110	0·095	0·083	0·074	0·066	0·06	0·055	0·051	0·047	0·044	0·041	0·039	0·037	0·033	0·030	0·028
1/4	0·196	0·168	0·147	0·13	0·118	0·107	0·098	0·091	0·084	0·079	0·074	0·069	0·065	0·059	0·054	0·049
5/16	0·307	0·263	0·230	0·204	0·184	0·167	0·153	0·142	0·132	0·123	0·115	0;108	0·102	0·092	0·084	0·077
3/8	0·442	0·379	0·331	0·294	0·265	0·241	0·221	0·204	0·190	0·177	0·166	0·156	0·147	0·133	0·121	0·110
1/2	0·785	0·672	0·589	0·524	0·471	0·428	0·393	0·364	0·337	0·314	0·295	0·277	0·262	0·236	0·214	0·196
5/8	1·23	1·05	0·92	0·818	0·736	0·669	0·614	0·569	0·526	0·491	0·460	0·433	0·409	0·368	0·335	0·307
3/4	1·77	1·52	1·325	1·18	1·06	0·964	0·884	0·819	0·757	0·707	0·663	0·624	0·589	0·53	0·482	0·442
7/8	2·41	2·06	1·8	1·60	1·44	1·31	1·20	1·11	1·03	0·962	0·902	0·849	0·801	0·722	0·656	0·601
1	3·14	2·69	2·36	2·09	1·89	1·71	1·57	1·45	1·35	1·26	1·18	1·11	1·05	0·943	0·859	0·783
1⅛	3·98	3·41	2·98	2·65	2·39	2·17	1·99	1·84	1·70	1·59	1·49	1·40	1·33	1·19	1·08	0·994

TABLE 6.

Shear Value of Single Binders (Two Arms).

Diam. (in.)	Area (in.²) (foe one arm)	Stress (lb/in.²) t_w	Values in lb. per unit lever at varying pitches. 2″	3″	4″	4½″	5″	6″	7″	7½″	8″	9″	10″	11″	12″	15″	18″	24″	Pitch
3/16	0·028	18,000	497	331	248	221	199	166	142	132	124	110	99	90	83	...	...	...	...
		20,000	552	368	276	245	221	184	158	147	138	123	110	100	92	...	...	...	...
1/4	0·049	18,000	883	589	442	393	353	294	252	236	221	196	177	161	147	...	...	...	...
		20,000	982	654	491	436	393	327	281	262	245	218	196	178	164	...	...	...	...
5/16	0·077	18,000	1381	920	690	613	552	460	395	368	345	307	276	251	230	...	...	...	...
		20,000	1534	1023	767	682	613	511	438	409	384	341	307	279	255	...	...	...	...
3/8	0·110	18,000	1988	1325	994	884	795	663	568	530	497	442	398	361	331	265	221	166	...
		20,000	2209	1473	1105	982	883	736	631	589	552	491	442	401	368	294	245	184	...
7/16	0·150	18,000	2706	1804	1353	1203	1082	902	775	722	676	601	541	492	451	361	301	225	...
		20,000	3007	2004	1503	1336	1203	1002	859	802	752	668	601	547	501	401	334	250	...
1/2	0·196	18,000	3534	2356	1767	1571	1414	1180	1010	942	883	785	707	642	589	471	393	294	...
		20,000	3929	2618	1963	1745	1571	1309	1122	1047	982	873	785	714	654	524	436	327	...

$$\text{S per unit lever arm} = \frac{\text{area of two arms} \times t_w}{\text{pitch}} = s_1, \text{ where } t_w = \text{permissible stress.}$$

Effective shear value $S = s_1 \times a$ = shear strength, where a = lever arm.

TABLE 7.

Value of Bent-up Bars in Shear (lb.).

Diamtter (in.)	Area (in.^{2})	Stress t_w (lb/in.^{2})	Inclination and angle				
			1 in 2 26° 34′	30°	1 in 1½ 33° 41′	1 in 1 45°	60°
½	0·196	18,000 20,000	1,580 1,756	1,767 1,963	1,960 2,177	2,498 2,776	3,060 3,400
⅝	0·307	18,000 20,000	2,470 2,744	2,761 3,068	3,063 3,403	3,905 4,339	4,783 5,314
¾	0·442	18,000 20,000	3,556 3,952	3,976 4,418	4,411 4,900	5,623 6,248	6,886 7,653
⅞	0·601	18,000 20,000	4,840 5,378	5,411 6,013	6,003 6,670	7,654 8,504	9,374 10,415
1	0·785	18,000 20,000	6,323 7,025	7,067 7,854	7,840 8,712	9,996 11,107	12,243 13,603
1⅛	0·994	18,000 20,000	8,002 8,892	8,946 9,940	9,923 11,026	12,651 14,057	15,495 17,228
1¼	1·227	18,000 20,000	9,876 10,975	11,043 12,270	12,246 13,608	15,618 17,352	19,128 21,252
1⅜	1·485	18,000 20,000	11,956 13,283	13,365 14,850	14,827 16,475	18,902 21,002	23,150 25,722
1½	1·767	18,000 20,000	14,226 15,805	15,903 17,670	17,640 19,602	22,492 24,992	27,590 30,556
1⅝	2·074	18,000 20,000	16,695 18,550	18,666 20,740	20,706 23,004	26,397 29,330	32,329 35,921
1¾	2·405	18,000 20,000	19,360 21,512	21,645 24,050	24,010 26,680	30,611 34,012	37,490 41,656
1⅞	2·761	18,000 20,000	22,226 24,695	24,849 27,610	27,567 30,630	35,142 39,047	43,038 47,820
2	3·142	18,000 20,000	25,292 28,102	28,278 31,420	31,370 34,858	39,990 44,435	48,975 54,420

$S = A_w \times t_w \sin \theta$. A_w = area of bar. t_w = working stress. θ = angle.

APPENDIX D.

Notes on Earth Pressures.

The subject of pressure due to soils at the back of retaining walls and on foundations is one which has provoked a great deal of controversy among engineers for nearly a century. The first and perhaps the classic theory of earth pressure was that of W. J. M. Rankine, Professor of Civil Engineering and Mechanics at Glasgow University, 1855-72. Rankine deduced his formula by considering earth as an elastic solid in a state of strain and by a study of the principal stresses. If ϕ is the natural angle of repose of the soil and w its weight in lbs. per cub. ft., then by the Rankine theory the pressure p in lbs. per sq. foot at any depth h below the surface is given by

$$p = w\,h \times \frac{1-\sin\phi}{1+\sin\phi} \qquad (1)$$

and p acts horizontally where the ground line is horizontal. (See Fig. A (1).

In cases where the wall is surcharged. *i.e.*, the ground line slopes upward from the back of the wall at an angle θ,

$$p = w\,h \times \left(\frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}}\right) \qquad (2)$$

See Fig. A (2).

The total pressure P for case (1) is

$$= \frac{wh^2}{2} \times \frac{1-\sin\phi}{1+\sin\phi}$$

acting horizontally at $h/3$ above the base.

In case (2),

$$P = \frac{wh^2}{2}\cos\theta \times \left(\frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta + \cos^2\phi}}\right)$$

acting at angle θ to horizontal at $h/3$ above the base.

On the whole, it can be said that the pressures given by the Rankine theory are on the high side.

Wedge Theory. This theory is commonly used in practice and assumes that the earth tends to slide down a line called the line of rupture and finally takes up its natural angle of repose ϕ. (It also takes into account the friction between the earth and the back of the wall in some cases). See Fig. B (1). The wedge of earth is in equilibrium under the action of three forces—(1) the pressure P acting on the back of the wall at $h/3$, (2) weight of wedge of earth w, (3) reaction R inclined at ϕ to the normal to the line of rupture.

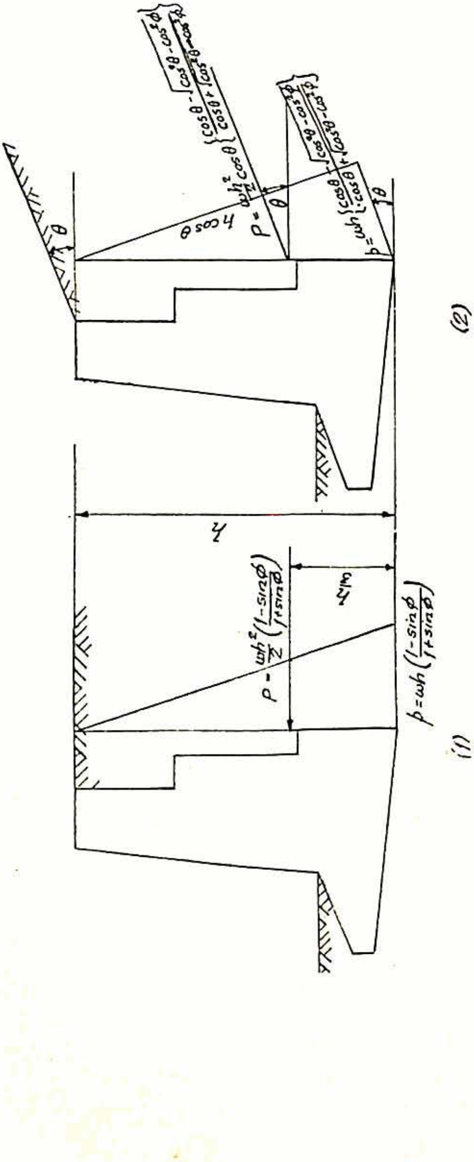

Fig. A.

For a wall with a vertical back and an angle of friction Z between the filling and the back of the wall (Z is also the angle between P and the normal to the back of the wall).

$$P = \frac{wh^2}{2} \times \frac{\cos^2 \phi}{\cos \frac{2}{3} \phi \left[1 + \sqrt{\frac{\sin 5/3\, \phi \sin (\phi - \theta)}{\cos \frac{2}{3} \phi \cos \theta}} \right]^2}$$

for $Z = \frac{2}{3} \phi$.

for $Z = \frac{2}{3} \phi$ and $\theta = 0$, *i.e.*, no surcharge.

$$P = \frac{wh^2}{2} \times \frac{\cos^2 \phi}{\cos \frac{2}{3} \phi \left[1 + \sqrt{\frac{\sin 5/3\, \phi \sin \phi}{\cos \frac{2}{3} \phi}} \right]^2}$$

In both cases the force P acts at $h/3$ above the base.
For $Z = 0$, *i.e.*, for no friction

$$P = \frac{wh^2}{2} \times \frac{\cos^2 \phi}{\left[1 + \sqrt{\frac{\sin \phi \sin (\phi - \theta)}{\cos \theta}} \right]^2}$$ for surcharged walls,

and

$$P = \frac{wh^2}{2} \times \frac{\cos^2 \phi}{(1 + \sin \phi)^2}$$ for no surcharge. See Fig. B (2).

The **Scheffler** theory assumes P to be inclined at ϕ to the horizontal. In other words, if P is taken for the wedge theory and resolved into two rectangular components, the component acting at ϕ to horizontal is taken as effective pressure (for level ground line). This theory is not used very much in practice.

The **Wedge** theory may be used for sand, gravel and light soils, the friction angle (Z) at back of the wall being varied from $\frac{1}{2}$ to $\frac{2}{3}$ of angle of friction of the filling. The late Professor C. F. Jenkin presented a paper before the Institution of Civil Engineers (Min. Proc. Inst. C.E., Vol. 234, 1931-2, part 2) on the results of experimental work and, so far as the tests go, they support a wedge theory of earth pressure for cohesionless soils.

When dealing with waterlogged filling, it should be borne in mind that the *total* pressure at the back of the wall is that due to the filling, *plus* the hydrostatic pressure and therefore the filling at the back of the wall should be drained. Where walls are subject to vibration, due to traffic, etc., no allowance should be made for friction between the wall and the filling. In cases where the earth behind the wall is subject to superimposed loads, due to roads, railways, etc., the condition can be met by assuming an equivalent

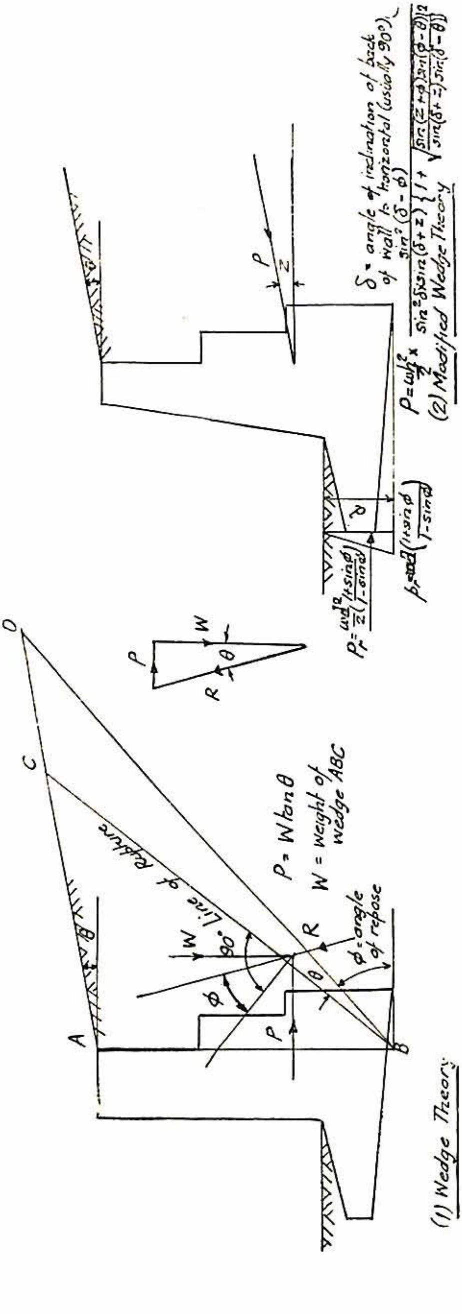

Fig. B.

horizontal surcharge of earth. For design purposes it is convenient to take an equivalent fluid pressure from earth pressure tables, but in all cases any formula should be used with discretion, having regard to the particular site conditions.

In dealing with clays or mixtures of clay and other soils, the stabilising force due to friction on the back of the wall may be greatly reduced, but there is a certain stabilising force due to cohesion. The pressures given by the wedge theory may be too high for the harder clays, but the pressures from soft clays may be in excess of those given by the wedge theory. In doubtful cases it is as well to use Bell's formula (see M.P.I.C.E., Vol. CXCIX.).

Bell gives the following value for the pressure,

$$p = wh \tan^2\left(45^\circ - \frac{\phi}{2}\right) - 2c \tan\left(45^\circ - \frac{\phi}{2}\right)$$

where w and h have the same values as before,
and c = cohesion per unit area,
and ϕ = angle of internal friction.

The foregoing remarks apply to the "active" pressure behind walls and footings. In opposition to the active pressure, there is a "passive" resistance in front of a wall, etc. For any depth d below the surface, the passive resistance in lbs. per square foot,

$$p_r = wd \left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$$ and the total resistance,

$$F = \frac{wd^2}{2}\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$$ (see Fig. B (2)).

Values according to Bell,

$$p_r = wd \tan^2(45^\circ + \phi/2) + 2c \tan(45^\circ + \phi/2).$$

The full passive resistance is not developed until some movement of a wall or foundation takes place and for that reason it is advisable to allow about 50% of above values. For clay or clayey soils which may vary in moisture content, drying out of the clay is accompanied by shrinkage which may cause the soil to shrink away from the front of the wall and a further movement of the wall is necessary before the passive resistance becomes effective again.

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1.	Safe Load on Machine-Cut Spur Gears.	
2.	Deflection of Shafts and Beams	Connected. (2–3)
3.	Deflection of Shafts and Beams (Instruction Sheet)	
4.	Steam Radiation Heating Chart.	
5.	Horse-Power of Leather Belts, etc.	
6.	Automobile Brakes (Axle Brakes)	Connected. (6–7)
7.	Automobile Brakes (Transmission Brakes)	
8.	Capacities of Bucket Elevators.	
9.	Valley Angle Chart for Hoppers and Chutes.	
10.	Shafts up to 5½-in. diameter, subjected to Twisting and Combined Bending and Twisting.	
11.	Shafts, 5¾ to 26 inch diameter, subjected to Twisting and Combined Bending and Twisting.	
12.	Ship Derrick Booms.	
13.	Spiral Springs (Diameter of Rd. or Sq. Wire).	
14.	Spiral Springs (Compression).	
15.	Automobile Clutches (Cone Clutches).	
16.	,, ,, (Plate Clutches).	
17.	Coil Friction for Belts, etc.	
18.	Internal Expanding Brakes. Self-Balancing Brake Shoes (Force Diagram)	Connected. (18–19)
19.	Internal Expanding Brakes. Angular Proportions for Self-Balancing.	
20.	Referred Mean Pressure Cut-Off, etc.	
21.	Particulars for Balata Belt Drives.	
22.	⅞″ Square Duralumin Tubes as Struts.	
23.	1″ ,, ,, ,, ,,	
24.	¾″ Sq. Steel Tubes as Struts (30 ton yield).	
25.	⅞″ ,, ,, ,, (30 ton yield).	
26.	1″ ,, ,, ,, (30 ton yield).	
27.	¾″ ,, ,, ,, (40 ton yield).	
28.	⅞″ ,, ,, ,, (40 ton yield).	
29.	1″ ,, ,, ,, (40 ton yield).	
30.	Moments of Inertia of Built-up Sections (Tables)	Connected. (30–31)
31.	Moments of Inertia of Built-up Sections (Instructions and Examples)	
32.	Reinforced Concrete Slabs (Line Chart)	Connected. (32–33)
33.	Reinforced Concrete Slabs (Instructions and Examples)	
34.	Capacity and Speed Chart for Troughed Band Conveyors.	
35.	Screw Propeller Design (Sheet 1, Diameter Chart)	Connected (35–37)
36.	,, ,, ,, (Sheet 2, Pitch Chart)	
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38.	Open Coil Conical Springs.	
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41.	Metric Equivalents.	
42.	Useful Conversion Factors.	
43.	Torsion of Non-Circular Shafts.	
44.	Railway Vehicles on Curves.	
46.	Coned Plate Development.	
47.	Solution of Triangles (Sheet 1, Right Angles),	
48.	Solution of Triangles (Sheet 2, Oblique Angles).	
49.	Relation between Length, Linear Movement and Angular Movement of Lever (Diagram and Notes).	
50.	,, ,, ,, ,, ,, ,, (Chart).	

51. Helix Angle and Efficiency of Screws and Worms.
52. Approximate Radius of Gyration of Various Sections.
53. Helical Spring Graphs (Round Wire) } Connected.
54. ,, ,, ,, (Round Wire)
55. ,, ,, ,, (Square Wire)
56. Relative Value of Welds to Rivets.
58. Graphs for Strength of Rectangular Flat Plates of Uniform Thickness.
59. Graphs for Deflection of Rectangular Flat Plates of Uniform Thickness.
60. Moment of Resistance of Reinforced Concrete Beams.
61. Deflection of Leaf Spring.
62. Strength of Leaf Spring.
63. Chart showing Relationship of Various Hardness Tests.
64. Shaft Horse Power and Proportions of Worm Gear.
65. Ring with Uniform Internal Load (Tangential Strain) } Connected.
66. Ring with Uniform Internal Load (Tangential Stress)
67. Hub Pressed on to Steel Shaft. (Maximum Tangential Stress at Bore of Hub).
68. Hub Pressed on to Steel Shaft. (Radial Gripping Pressure between Hub and Shaft).
69. Rotating Disc (Steel) Tangential Strain } Connected.
70. ,, ,, ,, Stress
71. Ring with Uniform External Load, Tangential Strain } Connected.
72. ,, ,, ,, ,, ,, Stress
73. Viscosity Temperature Chart for Converting Commercial to Absolute Viscosities. } Connected.
74. Journal Friction on Bearings.
75. Ring Oil Bearings.
76. Shearing and Bearing Values for High Tensile Structural Steel Shop Rivets, in accordance with B.S.S. No. 548/1934.
78. Velocity of Flow in Pipes for a Given Delivery } Connected.
79. Delivery of Water in Pipes for a Given Head
80. (See No. 105).
81. Involute Toothed Gearing Chart.
83. Variation of Suction Lift and Temperature for Centrifugal Pumps.
89. Curve Relating Natural Frequency and Deflection } Connected.
90. Vibration Transmissibility Curve for Elastic Suspension
91. Instructions and Examples in the Use of Data Sheets, Nos. 89 and 90.
92. Pressure on Sides of Bunker.
93-4-5-6-7. Rolled Steel Sections.
98-99-100. Boiler Safety Valves.
102. Pressure Required for Blanking and Piercing.
103. Punch and Die Clearances for Blanking and Piercing.
104. Nomograph for Valley Angles of Hoppers and Chutes.
105. Permissible Working Stresses in Mild Steel Struts with B.S. 449, 1948.
106. Compound Cylinder (Similar Material) Radial Pressure of Common Diameter (D1).

(Data Sheets are 3d to Members, 6d to others, post free.)

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