A LIST OF
"CONCRETE SERIES"
BOOKS IS GIVEN ON PAGE FACING PAGE 234

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PREFACE

This book is intended for use in everyday design and is restricted to work that would normally be undertaken by a civil engineer in general practice and would not entail calling in experts. It is practically restricted to work on land sites, the chapter on submerged sites being only a brief general survey.

A good practical design may be largely negatived unless it is supported by good drawings, a clear specification, and complete and fair bills of quantities. The author has tried to make this point clear in specific cases. Mistakes in the design of foundations are seldom due to mistakes in advanced theories or wrong assessment of known data, but are simple matters of oversight or non-observance of basic principles. Possibly ninety per cent. of the cases of structural distress reported in the technical press would have been avoided if the engineer had gone back and re-read the first few pages in the first chapter of any good book on design before completing his drawings.

The author is greatly indebted to the Chief Engineer of the North Thames Gas Board for permission to include descriptions of several foundations. Detailed acknowledgements are given on page 232.

G. P. M.

LONDON, 1961.
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CHAPTER I

INTRODUCTORY NOTES

Perhaps the most important point in designing foundations is to assess their importance relative to the structure as a whole, to estimate what may happen if they fail, or partially fail, and what could be done to remedy any such failure.

Most engineering problems involve human considerations that cannot be expressed in mathematical terms. Although not an inexact science, engineering is often a science of the inexact and the design of foundations often involves more inexact information and more human considerations than are involved in the design of the superstructure. While all materials of construction have to comply with some specification, the ground under the foundations is as nature made it and not as the engineer would have it.

"Soil mechanics" may be able to provide an approximate picture of site conditions at any one time but how these will be affected by seasonal changes, by constructional operations on the site, by constructional operations on adjoining sites during the life of the structure, by mining, earthquake, or even by heavy artesian pumping from under the site can only be roughly assessed by an intelligent guess.

It is usually practicable to strengthen members of the superstructure to carry unforeseen additional loading or to remove minor (or even major) members which are in the way when new and larger plant is substituted for old. This is particularly true with simple steel-framed superstructures, but to extend foundations or to lower them to accommodate new pits or trenches below the lowest floor level is usually a slow and costly major operation.

To design foundations successfully it is essential to bear all these points in mind, not merely at the outset when determining their type, extent and depth, but continually up to the completion of the detailed design.

This raises a point in the design of superstructures. A complicated, stiff-jointed, statically-indeterminate superstructure may be the best solution on a rock foundation but the worst on estuarine silt. On a clay foundation considerations of time may enter the design in a manner peculiar to this one problem.

If condition (a) and condition (b) in Fig. 1 represent a site before and after construction and if $W_1$ (the weight of ground removed) and $W_2$ (the total weight of the structure) are equal and $W_2$ is spread evenly by a raft foundation then the intensity of loading on plane BC is the same after construction as it was before. At first sight it might be assumed that conditions (a) and (b) were identical but there are three reasons why this may not be so.

(1)—The new construction may allow the ground water to percolate down the walls AB and DC and thus soften the clay near B and near C. This can and should always be prevented by casting concrete and driving sheet-piling tightly against the clay and avoiding all loose filling.

(2)—If the new structure is sufficiently large it may obstruct the free flow of
ground water through the ballast and alter the ground-water level. This effect is unlikely to be of much importance.

(3)—There must be an appreciable lapse of time between conditions (a) and (b) when the clay at level BC is relieved of most, if not all, of its load. This may result in more or less severe upheaval of the bottom and although the weight of

the new structure when completed and loaded will push the bottom down again this movement may take years to accomplish and be a great nuisance while taking place. If the clay varies in stiffness from one side of the site to the other the rising and subsequent settlement may vary in amount and velocity thus racking the building.

In one bridge pier on clay, this observed upward movement was about 3 in. The completed pier has settled a total of about 6 in., that is about 3-in. recovery of upheaval and about 3-in. net settlement.
If it were possible to convert condition (a) into condition (b) instantaneously no such trouble could arise and, of two possible alternative designs, that which offers the quicker construction will give less trouble in such a case.

In the design of foundations there is one attitude of mind that must be avoided—the tendency to regard the intensity of compression on a loaded area of ground in the same light as the compressive stress in a free-standing column. The bearing capacity of an area of soil depends on the lateral support of the surrounding soil and this lateral support depends on the height of soil above and extent of soil alongside the foundation. Before shearing failure of the soil can occur, solid particles of soil must escape from below the foundation and the longer and more difficult the minimum path of escape the greater the load any given area can support. It is therefore wrong to confine attention to the actual loaded area, as the minimum distance to the free surface of the ground in all directions should be noted.

The soil under a foundation is in a position remotely similar to the concrete in the core of a hooped column which is supported and prevented from escaping laterally by the steel hooping. It is also remotely similar to the water surface under a floating body whose bearing capacity is always in linear proportion to the depth below the free surface and is entirely dependent on the lateral support of the surrounding water.

All engineering design is, or should be, considered in the light of the methods of construction likely to be adopted. The design of long-span bridges and high radio towers may depend more on the method of erection than on the stresses in the completed structure but, apart from such examples, it is generally true to say that the design of small and medium-sized superstructures depends less on a consideration of constructional methods than does the design of their foundations.

It must also be remembered that most foundations are hidden from sight. Whereas deterioration of a superstructure is generally obvious or visible after removing small areas of walling or cladding, deterioration of foundations is usually not seen until failure occurs. A good mixture of concrete and ample cover to the steel reinforcement are necessary. The use of timber in foundations below the lowest permanent ground-water level is now unusual in Britain but is sound practice. All timber above this level must be easily accessible for inspection and renewal.

**Factor of Safety**

The question of an appropriate factor of safety is often raised, usually without defining what is meant by "factor of safety". In theory the factor of safety is the strength of the structure divided by the loading. Since both these values in practice are stochastic the result can only be statistically assessed within some arbitrary parameter. What is often miscalled the "factor of safety" is the ratio between the strength as calculated by one arbitrary Code of Practice and the loading stipulated by another arbitrary Code. Since these Codes vary from one edition to another and since there is no statutory guarantee that their compilers know what they are codifying, this ratio is variable and no more reliable than the opinions of those members of the drafting committee who were not too busy to take an active part in the drafting. Any interpretation must depend eventually on personal interpretation of what is "reasonable risk" which involves a
consideration of the results of structural failure. To suggest that the lead in a pencil should be heavily reinforced to eliminate breakage is ridiculous since the consequential damage is infinitesimal. If the draughtsman was severely injured every time his pencil point broke it would be quite a different matter. Was it reasonable to suppose that the Dutch and English coastal defence works that had stood for a century were "reasonably" safe the day before they were overtopped in 1953? We now know that the dam near Fréjus (or its foundations) had a factor of safety of less than unity. How many experts were satisfied that it was "reasonably" safe a week before failure occurred? It is clear that any structure whose failure must involve wholesale loss of life should have a larger factor of safety than one whose failure involves only petty annoyance, but what limits should we set? If we can increase the calculated factor of safety by 50 per cent. at a cost of only 5 per cent. increase in expenditure the decision may be easy but if the cost increases 75 per cent. it may be difficult. In doubtful cases the engineer should estimate what an increase in the strength of the foundations would cost and how this would compare with the total final cost of the whole building since failure of a foundation involves everything above it.

Between the years 1909 and 1959, the allowable stress in mild steel (28 to 33 tons per square inch) in the London area has increased from 75 to 10 tons per square inch (and now presumably to 10 ton) while the assumed imposed load on an office floor has decreased from 100 to 50 lb. per square foot, a variation in official outlook of \( \frac{10.5 \times 100}{7.5 \times 50} = 2.8 \), representing a mental readjustment of 180 per cent. Is a factor of safety then, to the official mind, only a matter of opinion? If a bearing pressure of 2 tons per square foot was considered reasonably safe on a certain type of ground in 1909 under an office building assumed to carry an imposed load of 100 lb. per square foot, should we limit the pressure to 1.5 tons per square foot if we now calculate for loads of only 50 lb. per square foot?

If the mechanical engineer estimates that his plant will weigh 50 tons, he may (and the author has often suspected that he does) if naturally timid or doubtful of the structural engineer's ability, increase this weight to 75 tons or even 100 tons. If the engineer for the superstructure mistrusts both the mechanical engineer and the foundation engineer he may pass this on as 150 tons. If the foundation engineer also has his doubts he may design the foundations to carry 200 tons. This is only one of the human considerations involved in foundation design. If the mechanical engineer's estimate of 50 tons is correct and if the foundation engineer correctly assesses the allowable bearing pressure we have a factor of safety of about 10.

The only factual step towards establishing a value for the factor of safety is to test-load the finished structure. After that we still, in most cases, do not know what maximum load the structure may reasonably have to carry during its effective life (after having, of course, determined how long its effective life can reasonably be expected to last). Would it now be reasonable to include in our estimated loading the risk of the superstructure being struck by an artificial satellite?

The disaster at Fréjus presents also the other side of the argument. Apparently the Roman amphitheatre withstood the onrush of water while modern buildings collapsed. Apart from emphasising the superb technique of the Roman
engineer, was the designer of this building grossly extravagant in providing a structure that outlasted its useful life by 1500 years? He could scarcely have foreseen that it would prove a source of attraction to tourists 2000 years after his death. In our present over-organised community where so many authorities have power over so many sites and—rightly or wrongly—see fit to interfere with the actions of individuals, a factor of safety must also include a "factor of interference" to allow for last-minute (official) intervention.

Levels

The engineer may be asked for advice about fixing the ruling level, such as the level of the main workshop ground floor, the general level of the factory roads, rail level, outfall-weir level, top water-level in reservoirs and water towers, etc. On many occasions clients' engineers have complained that the original choice of level was too low (usually 1 ft. too low) and subsequent extensions and developments on the site have been hampered thereby. Never has the author heard a client complaining that his levels are too high.

It appears then that the first structure on any site should be built about 1 ft. above the level indicated by short-term economics.

Soil Mechanics

There is a modern school of thought that apparently imagines that any problem may be solved by attaching to it a new and longer name. It might appear from some works on this subject that no one ever designed or constructed a foundation before the year 1930. The author is quite unable to forget that many thousands of engineers designed and constructed many millions of foundations centuries before the term "soil mechanics" was coined. Judging by some recent unfortunate happenings some of the ancients knew more about it than some of the moderns.

Soil mechanics, in so far as it applies to clays, is really a sub-division or simplified version of rheology. The rheologists are, in one respect at least, ahead of the engineer. Having failed to pierce the art of thumb-technique by scientific apparatus based on the principles of chemistry, physics and mathematics some rheologists have now turned to psychology. The author doubts if the centuries-old thumb-technique of foundation testing could be reduced to scientific measurement (or at any rate to some less personal parameter) by the introduction of a rheological psychologist (or psychological rheologist) but the results could scarcely be more misleading than one or two reports the author has seen from laboratory experts in soil mechanics.

Soil mechanics may have academic interest, but only in the field of plastic clays has it yet produced anything that was not already within the grasp of the practising engineer so far as foundations are concerned. It is claimed that long-term settlement in clay, on a site where no comparable buildings exist, may be more accurately estimated by soil mechanics than by hand examination of the borehole cores. Judging by some published calculations this claim may still be optimistic (doubts have recently been cast on how far an "undisturbed" sample is really undisturbed) but it should be capable of substantiation.
Elementary Approximate Theory of the Action of Soils

The science of rheology postulates that eight (some experts say nine) different physical characteristics of a solid must be known before its behaviour under loading can be deduced. In the simple theory that follows two theoretical types of soil are considered and in each case it is assumed that the strength depends on a single factor, thus ignoring seven (or eight) of the factors which must be considered before an accurate result could be expected.

Natural soils are divided into two main classes: non-cohesive and cohesive, many soils being a mixture of the two.

Non-cohesive Soils.—At one end of this range (or rather just beyond the end) stands the theoretical soil assumed by Rankine. This consists of fine uniform particles whose stability depends entirely on their weight and on friction between them, this friction being measurable by the instantaneous angle of repose and which, under all pressure conditions, has one definite and constant value. In a limitless expanse of such soil with a level surface the ratio between

![Fig. 2.](image1)

![Fig. 3.](image2)

the intensity of pressure on a vertical surface and that on a horizontal surface at any given point must lie between \( \frac{1 - \sin \phi}{1 + \sin \phi} \) and \( \frac{1 + \sin \phi}{1 - \sin \phi} \), where \( \phi \) is the instantaneous angle of repose. Rankine's Theory as such must be correct since it consists of unassailable mathematics. Its weakness lies in the fact that only one type of actual soil is even approximately similar to the theoretical soil postulated by Rankine, that is wind-blown sand under desert conditions, while some of the older sands, for example, Bunter and Thanet sands, will stand with a vertical face of 30 ft. or more and thus differ radically from Rankine's assumptions. It should also be remembered that the theory only applies exactly to a large undisturbed volume and not to overloaded local areas.

If in Fig. 2 we have Rankine's soil, the pressure on a horizontal surface at a depth \( h \) under undisturbed conditions is \( wh \). If we bury a small plate whose area is \( A \) and attempt to pull it out by applying a tension \( T \), the plate will not move when the tension reaches a value of \( whA \). If, however, the area of the plate is increased so that its linear dimensions are very large compared with \( h \) then the plate will, in the limit, lift when \( T = whA \).

Suppose we construct a foundation as shown in Fig. 3 at a depth \( h \) in Rankine's soil without disturbing the surrounding soil. Before we apply the load \( W \), the vertical pressure \( p_1 \) on a horizontal plane immediately outside the corner of
the foundation is $wh$ and, according to Rankine, the horizontal pressure $p_3$ on a
eroundly perpendicular pressure at a point immediately
inside the edge of the foundation becomes $p_2$ and the horizontal pressure $p_4$ at
this point lies between $\frac{-\sin \phi}{1+\sin \phi} p_2$ and $\frac{-\sin \phi}{1-\sin \phi} p_2$. Now, since $p_3$ must equal
$p_4$, if $p_3$ is greater than $\left(\frac{1+\sin \phi}{1-\sin \phi}\right)^2 wh$ then $p_2$ must exceed $wh$ and a small column
of soil immediately adjoining the foundation will tend to move upwards. This
tendency is local as the effect of applying the load $W$ must decrease further away
from the edge of the foundation and the soil is capable of resisting a local upward
pressure exceeding $wh$ (see Fig. 2). The amount by which the local upward
pressure may exceed $wh$ increases with the ratio $h/B$. (Estimated pressures under
the point of a pile driven in ballast may exceed 100 tons per square foot.) But
under the most unfavourable conditions the ultimate bearing capacity of Ran-
kine's soil will not fall below

$$\left(\frac{1+\sin \phi}{1-\sin \phi}\right)^2 wh.$$

If $w=112$ lb. per cubic foot, $\phi=30$ deg., and $h=2$ ft. then the ultimate bearing
capacity will not be less than $\left(\frac{1+0.5}{1-0.5}\right)^2 \times 0.05 \times 2$, that is not less than 0.9 ton
per square foot.

Owing, among other things, to variations in grading, compaction, moisture
content, and possible impurities that may add some cohesion, natural sands, even
quaternary sands, are much stronger than Rankine's soil, but the principles
of equilibrium in Fig. 3 apply to some extent to all non-cohesive soils. The
vertical pressure $p_2$ in Fig. 3 induces a horizontal pressure $p_4$. If the ground is to
resist this full pressure safely then it must be confined laterally by a stretch of
undisturbed level ground of width $4B$, by other foundations or retaining walls
cast tightly against the ground or, in extreme cases, by sheet-piling. The
foundation depends for its strength on the weight of the adjoining soil in a way
 remotely similar to the way in which the free end of a cantilever depends on the
weight of the tailing-down end. To remove part of this soil, even temporarily,
will reduce the safety of the foundation.

So long as the surrounding ground above foundation level has the necessary
weight it will act just as well as a bed of virgin sand. The bearing capacity of
sand may therefore be increased by adding a layer of consolidated filling, hardcore
or concrete above the original ground level.

Cohesive Soils.—At the extreme end of this range stands an ideal soft
plastic clay-type soil whose stability depends entirely on its shearing strength.
Few natural clays resemble this theoretical soil very closely although artificially
prepared and remoulded specimens approach more closely thereto.

Figure 4 shows a cross-section through a long foundation, in such a soil, of
width $B$ carrying a load $W$ per unit length. Before this foundation can fail, the
clay beneath it must squeeze out laterally and must push the surface up some-
where. (See Fig. 180 on page 171.) It is here assumed (incorrectly) that the
ground fails by shearing on the surface mnoq where mnoq is a semi-circle. Taking moments about the point s the load causing failure is \( \frac{1}{2}W \) at a distance \( \frac{1}{2}B \). The stabilising forces are the shearing force \( f \) on the surface mnoq and the weight of soil oqr.

\[
\frac{1}{2}W \times \frac{1}{2}B = \frac{1}{2}Bf\pi \times \frac{1}{2}B + \frac{1}{2}Bfh + \frac{1}{2}Bwh \times \frac{1}{2}B.
\]

\[
\frac{W}{B} = 2\pi f + 4\frac{f}{B} + wh = 6.28f \left[ 1 + \frac{h}{1.57B} \right] + wh.
\]

All these figures apply to unit length of foundation.

The net loading intensity at the ultimate bearing capacity is therefore

\[
6.28f \left[ 1 + \frac{h}{1.57B} \right] + wh.
\]

This result cannot possibly be regarded as a serious attempt to assess the bearing capacity of natural clays since seven out of the eight physical character-

![Fig. 4.](image)

istics are ignored and the plane of failure mno is not circular in section as assumed in Fig. 4. Slightly more exact theory and experiment suggest a value for the ultimate bearing capacity of \( 5f(1 + h/4B) \) for a long strip footing. If we have two foundations of width \( B \) spaced a clear distance of \( B \) apart, as in Fig. 5, the second expression in the approximate equation of equilibrium would disappear and it is suggested that the full value of \( 5f(1 + h/4B) \) should only be taken if the clear spacing exceeds \( 4B \).

Comparing the results for our theoretical clay with those for Rankine’s sand, we see that lateral confinement is not so important although we require an extent of \( 2B \) of level surface (or equivalent lateral support) outside our outer foundations if we are to take the full value of our increase-with-depth factor. The digging of trenches near the foundations is also not so dangerous but should be undertaken with caution and any such trenches are best filled back solid with lean concrete.
There is one notable difference between the cohesive and non-cohesive soils. If we are to rely on the shearing on the surface $q_0$ in Fig. 4, then all the surrounding soil above foundation level must be equal in shearing strength to the soil immediately below the foundation. Loose filling or much softer clay is only partly effective.

**Insufficient Lateral Confinement**

If foundations are placed close to a cutting or basement they may cause failure of the bank or overturning of the retaining walls as indicated in Figs. 6, 7 and 8. In Figs. 7 and 8 the foundations are best taken below a line drawn at 30 deg. to the horizon from the base of the wall. Fig. 8 is based on an example in the Pennant series where layers of hard rock were separated by very thin layers of softer material.

**Protection of Foundations against Extremes of Temperature**

Dry ground is an excellent insulator. Very hot installations (for example retort benches) or very cold (for example methane tanks) are protected by thick layers of insulating material but the minute amounts of heat that must escape through this insulation build up in the ground below them and can bake or freeze the soil to a depth of many feet if a large installation sits directly on the ground. Serious settlement or frost-heave may result. Ventilated air spaces or air ducts should be provided under all such foundations.
CHAPTER II

SITE INVESTIGATIONS

It is assumed that the reader has studied the Civil Engineering Codes of Practice No. 1 "Site Investigations" and No. 4 "Foundations". A précis of the author's view appears in part of the current edition of the latter Code (1954).

Sources of Preliminary Information

For a site in Britain the first step is to acquire the 6-in. ordnance map of the district. Unfortunately many of the geological versions of the 6-in. ordnance survey are not at present available and the plain contoured map is all that can be bought. The next step is to visit the nearest record office (for example the library of the Geological Museum in South Kensington) to consult the geological maps, both solid and drift, transferring this information in coloured pencil to one's own map over an area of, say, half a mile square so that the general nature of the ground is known for a distance of 400 yards on all sides of the site.

Records of boreholes exceeding 100 ft. deep are now preserved and details of any near the site should be copied and added. Some areas have been riddled with deep boreholes in search of water and a comprehensive picture of local strata is at once available to a depth of several hundred feet. Any geological memoirs should also be read through. These are generally written from the geologist's point of view and are mostly concerned with establishing the age and geological sequence of the strata and giving a comprehensive list of fossils. One may wade through many pages of information before finding any fact of engineering significance but there may be a vertical section passing right through the site which could be of great value. The next step is to inspect and survey the site and identify the geological data already collected. A camera is always useful as it retains impressions the human eye and brain forget. All existing structures closely adjoining the site should be examined for signs of settlement and a guess should be made as to the intensity of loading under them. The usual two-story villa residence weighs possibly 0.75 ton per foot run of external wall and 1.25 tons per foot of party wall or possibly a maximum of 0.5 ton per square foot on the ground. Local experts such as the borough, city or county engineer or the secretary of the local geological society may supply useful information. Nearby railway cuttings, clay-pits, gravel-pits and quarries should be noted. Data from retired builders and contractors' foremen are usually not only helpful but generally very much to the point. Even the "oldest inhabitant" at the local public house proved of great help in one case. The engineer should never ignore information—whatever the source. If the site is seen at the end of a long dry summer it should be envisaged at the end of a long wet winter and vice versa. All surface water should be noted or guessed at and the extent to which the new building will modify its amount and flow should be estimated.

The engineer now has all the information freely available and must proceed to make a preliminary drawing of the building showing the distribution and amount
of the loads to be carried. On a tricky site the loads should be sub-divided into
dead load, probable live load, improbable live load and probable but intermittent
live load.

Additional Information

A decision must then be made as to the amount of further site exploration
required. The ground must be explored down to the level of an "identifiable
stratum". In general all tertiary and older formations whose local extent and
strength are known, will fall within this category, but it may also include, particu-
larly in built-up areas, glacial deposits, beds of terrace gravel and even quaternary
clays when local records and nearby buildings show quite definitely their depth,
extent and strength. The upper part of older formations is often weathered down
to softer material. The top of the Cornish granite is extensively rotted down to
china clay and the black, chunky Oxford clay becomes light grey and putty-like
where it lies under water-bearing ballast. Perhaps the best-known example in
Britain is the deep stratum of tertiary clay which lies under the Thames basin
and is known as London Clay and which eventually carries nearly all the buildings
in Greater London. In its undisturbed state it is grey-blue to black but when
exposed it oxidises to brown and softens when in contact with ground or surface
water to depths up to forty or fifty feet. The transition from the softer brown
clay to the harder blue (or black) clay may be very gradual or quite sudden.
There was a rule of thumb in the London area which said "Blue clay 4 tons,
brown or yellow clay 2 tons per square foot"—a crude statement in the eyes of
soil mechanics experts but a shrewd practical assessment.

There are two reasons why site exploration cannot always be stopped as soon
as an "identifiable stratum" is reached. Firstly, as already stated, the upper
part of this stratum may have deteriorated badly and secondly, there may be a
softer stratum below. For example, if the "identifiable stratum" is a bed of
water-bearing ballast overlying clay, the upper part of this clay is certain to be
softened. Here the half-ton rule may decide. The only natural soils that will
not safely support a net loading intensity of half a ton per square foot are very
soft clay, silt, peat and running sand. If the local geology definitely precludes
these from the site then the level at which the net loading intensity falls to half
a ton per square foot is the limit to which we need go.

Square Footings

Figure 9 shows an isolated square footing carrying a load $W$ on an area $B^2$ at
$W/\frac{B^2}{\text{per square foot}}$. At a depth $D$ below the underside of this foundation the
intensity (assuming a spread of 1 in 2 as shown) is $W/(B+D)^3$ and if $D=B$ then the
intensity has fallen to a quarter. If $B$ is 10 ft. and the load $W$ is 400 tons the
net loading intensity falls to $\frac{1}{4}$ ton per square foot when $D$ is 18 ft. 4 in.

Piled Foundation

Suppose we have an isolated group of nine piles spread at 3 ft. 6 in. centres
and carrying 35 tons each as in Fig. 10. The intensity of loading at the pile
points (allowing for some side friction) is spread over an area of, say, 10 ft. 6 in.
by 10 ft. 6 in. giving 2.85 tons per square foot at this level. At half a ton per square foot it would cover an area of 25 ft. by 25 ft. which is reached at a depth of 14 ft. 6 in. below the pile points.

If the footings are more closely spaced, the slope lines of 1 in 2 may overlap and allowance must be made for this. It is not suggested that exploration must always be taken down to the half-ton line but it is always useful to draw this line on the cross-sections of the building.

Trial Pits and Boreholes

However complete local geological records may be and however small the building, a trial pit should be put down to the "identifiable stratum" unless copious ground water makes this impracticable, and this should be left open (and fenced or covered) for inspection by the contractors invited to tender. Assuming that there is no record of local geology and no nearby buildings, a programme of boring must be drawn up and defined by a specification and bill of quantities. On a firm level site well above the nearest river, lake or canal a trial hole 10 ft. deep in the middle of the site and three boreholes 20 ft. deep placed on the points of a triangle would be a reasonable start to the bill of quantities with provisional items for two more pits, for extending the borings to 40 ft. (possibly 60 ft. for a very heavy building) and for three additional boreholes. The Standard Method of Measurement should be used as a basis and all reasonable extras and all difficulties that can be reasonably foreseen should be thought out and billed. A legal point here should not be overlooked. Having received two or three prices for carrying out this work the engineer must either place an order on behalf of his clients or get his clients to place the order. Since this is only
preliminary work it is very easy for an engineer to place a direct order without realizing that this makes him personally responsible for the cost.

On an obviously poor site, say the flat bank of a wide shallow estuary, the boring programme may start with three boreholes 40 ft. deep with a provisional extension to 100 ft. If nothing really solid is found within 100 ft., and if the building cannot be carried on some type of raft or cannot be moved to a better site, then it really passes beyond the limits of normal everyday foundation work. Generally a complete review of the situation is necessary at this stage before deciding to bore to 200 ft. The engineer must keep continually in touch with records of the boring and should stop exploration as soon as he has a clear picture of the strata to the depth below which the soil cannot impair the safe bearing capacity or appreciably add to the settlement. It is normal practice to ask for samples at 5-ft. intervals (undisturbed samples from clay soils) so that the shearing strength can be measured.

If the site is likely to call for driven piles, for example if similar existing structures on nearby sites are on piles, test-piles should be driven. These, of course, should be located exactly so that they may be used as part of the permanent building. The detailed record of the driving when compared with the boreholes gives a much clearer picture of the strength of the site and, in this light, may be regarded as part of the site exploration. Test piles should always be of the same material and of the same size as the proposed permanent piles.

No matter how complete local records may be or how many trial pits or boreholes have been put down, every accessible foundation area should be inspected by the engineer or a responsible representative immediately before the preliminary concrete is placed. Local variations are always possible between boreholes. Some of these occur as a natural corollary of site or ground water conditions but some are sudden and apparently unreasonable. Some again, are due to human agency—an old canal basin full of black silt and obviously abandoned and forgotten long ago crossed the end of one site.

**Ground Resistance**

The author’s method of final inspection is mostly rule-of-thumb. It is assumed that the shearing strength of undisturbed samples of clay from the boreholes has already been measured. On a clay soil visual footprints are an indication of this. A man weighing 270 lb. and placing all his weight on a shoe heel with an area of $7\frac{1}{2}$ square inches imposes a pressure of 1.8 tons per square foot and an appreciably deep imprint suggests a safe pressure in the region of a half-ton per square foot. The first step is to dig up samples of the ground and to try to mould them in the fingers. The next step is to try the ground with the point of a bar. A man’s weight of 270 lb. on the area of a $1\frac{1}{2}$-in. diameter bar is 13.5 tons per square foot. If it is possible to push the bar easily, and definitely, into the ground a limit of 1 ton per square foot is indicated. If it needs all a man’s weight to penetrate a few inches then 4 tons per square foot is possible. The bar is then jabbed into the ground as hard as possible, worked round and jabbed down further. Finally a 6 ft. length of 4-in. diameter rod (or 1-in. diameter gas-barrel or something similar) is steadily driven down with a 7-lb. sledge hammer. If the resistance steadily increases all is well. If it decreases unexpectedly (an extremely unlikely event if the engineer has made even a moderately intelligent assessment
of his data) then it is necessary to dig down and find out why. If resistance definitely, and appreciably, increases a short distance below the surface, the foundation should generally be taken down to this level as a cheap insurance against possible future overloading of the building. On a ballast foundation the grading is visible and only the tightness of packing is in question. This can be tested by dropping the tip of a heavy bar on to the surface. If the bar rebounds without appreciable penetration anything up to 6 tons per square foot is indicated. Fine uniform sands and loose ballast are more akin to clays and the final checking of such foundations can be made by the same methods. With increasing mechanisation there is an increasing tendency to excavate all foundations by mechanical diggers. Careless handling of the bucket may let the tines run through the sand or ballast below foundation level, leaving a layer of loose materials some 6-in. deep. To prevent this the specification should definitely forbid it, and as a further safeguard an item should be added to the bill of quantities—"E.O. for removing last 6-in. depth by hand...square yards..."

Obstructions

Man-made features on the site must be included in the exploration, such as drains, gas, water, electricity and telephone services below ground and overhead mains and cables. The engineer must decide whether it is better to divert any of these as they may interfere with construction, before commencing or better still to support and maintain them insitu. All such features can cause very serious obstruction to the use of mechanical plant. If they can be avoided by minor alterations to the design of the foundations it may well be worth while to do so. For example, if a major diversion of drains can be avoided by making a foundation 8 ft. 0 in. x 12 ft. 6 in. instead of 10 ft. 0 in. x 10 ft. 0 in. it will save time and cost. On a built-up site trial pits should be dug around the perimeter to check the position and extent of the foundations of the adjoining buildings. Even if drawings of these are available the engineer should put down a few pits to make sure these drawings are final and accurate.

Dealing with ground water is an obvious hazard and testing for chemical contamination is necessary unless the site is definitely known to be free. Putting down a sump and carrying out pumping tests to see to what extent the ground water may be lowered, is not usually included in site exploration. There have been cases where such a precaution would have avoided major changes in design after the contract had been placed followed by claims for extras. How to deal with surface water from adjoining sites should also be considered. If work has to be done inside steel sheet-piling and if this piling has to be driven through coarse ballast, a few sheet-piles of different weights may be driven to find out what weight is required. This procedure is very unusual, but could save a lot of trouble later on.

It will be seen that site exploration consists of three stages. Firstly the gathering of all existing data and a superficial survey of the site and its surroundings over a large area. Secondly the procuring of more exact information of the strata underlying the actual building site from trial pits and borings. Thirdly a direct inspection of the actual area of ground to be loaded (in a piled foundation the driving record of each pile). The cost of site exploration may be more than the value of any information obtained. Suppose that it is definitely
established by preliminary examination that a site may be safely loaded to one ton per square foot and probably, but not certainly, to one and a half tons. If the cost of further site exploration, which is necessary to decide definitely whether or not the higher loading may be adopted, is £400 and if the saving in cost by reducing the size of the foundations is only £300 then, apart from the delay, this further exploration must result in a net loss of something between £100 and £400.

If visual and manual examination of trial pits and borehole specimens still leads to no definite decision, a loading test may be carried out providing that its cost does not exceed the value of the results (see Chapter III).

In Civil Engineering Code of Practice No. 1 the terms describing the intensity of pressure on the ground are confused and not always in line with the more correct terms in Code No. 4.

The art of foundation design includes not merely the ability to envisage and carry through a reasonably extensive investigation of the site at a reasonable cost but also in the ability to extrapolate hopefully, as information accumulates, and to foresee what to do if and when the final exposure of the whole foundation area contradicts the preliminary data. It is useless to add that engineering instinct is of supreme value in such cases for this is inborn and can neither be learnt nor acquired.

The salient point about the much-publicised failure of the foundations of the Transcona silo (see "Geotechnique", Vol. III, 1952 and 1953) is that it was obvious to anyone within miles of the site that similar ground would not support gravel embankments 30 ft. high. The ultimate bearing capacity at a depth of 12 ft. was therefore about 1 3/4 tons per square foot. The amazing thing is that anyone should have been surprised when the silo foundation failed at 2 1/2 tons per square foot. The intense smoke-screen of "scientific" investigation published since the failure merely serves to obscure the main facts.
CHAPTER III
LOADING TESTS

Loading tests fall into two categories, namely small-scale tests on pads or exploration piles much smaller than the actual footing slabs or permanent piles, and test loads on actual full-scale permanent work. In common with other types of site exploration, loading tests may cost more than they are worth. If the object of the test is merely to decide whether the allowable bearing pressure shall be either 2 tons or 2 1/2 tons per square foot, the cost of the test plus the cost of the delay may be more than any possible saving in the cost of the permanent foundations.

It is presumed that the reader is familiar with Appendix J of the Code of Practice for Site Investigations. This contains some confusion between settlement and shear-friction failure and between ultimate bearing capacity, safe bearing capacity and allowable bearing pressure. The author does not recommend the use of small test plates particularly if the excavations are exposed to the weather. (The site of the test should be covered by tarpaulins rigged over tubular scaffolding.)

Small-scale Loading Tests

The value of ground loading tests varies directly in proportion to the area of ground tested. If this area is approximately equal to the largest foundation indicated by the preliminary design, and if the foundations are widely spaced, loading tests may be regarded as evidence of safe bearing capacity. It is not, of course, suggested that a loading test alone is conclusive evidence. It must be preceded by a proper site investigation on the usual lines. The engineer must already have in his mind the upper and lower limits of the allowable bearing pressure. The loading test should fix a definite value within this range. If the area tested is small or if the foundations are to be closely spaced so that the spread lines soon overlap (see Figs. 9 and 10, page 13), loading tests can be misleading or even dangerous. One loading test on a small shallow foundation indicated a reasonably high bearing capacity. The ground consisted of dry topsoil and clay overlying peat with water-bearing ballast about 20 feet below the surface. Ignoring advice that the structure should be carried on piles driven into the ballast, shallow foundations were used with rather unhappy consequences.

Full-scale loading tests entail the spending of considerable time and money but loading tests on very small areas (say 1 ft. by 1 ft.) carried out with doubtful apparatus under conditions unlike the final foundations, seem scarcely worth while as rule-of-thumb bar testing plus laboratory examination of samples must be at least as reliable a guide.

A concrete pad 3 ft. by 3 ft. properly cast on the ground as a normal footing is the smallest area the author would readily accept for a serious test. If this is absolutely impossible one area 1 ft. by 1 ft. and one 2 ft by 2 ft. might just be worth while. Loading must be absolutely central. Measurement should be by a continuously-recording apparatus (with micrometer dial) and this apparatus
should be supported at least 10 ft., and preferably 20 ft., away from the loaded area. To justify a working load of 2 tons per square foot a pad 3 ft. by 3 ft. would require some 40 tons of kentledge. To check the loading on the 3 ft. by 3-ft. pad a second pad 6 ft. by 6 ft. should be tested on an adjoining spot. (To save moving apparatus the smaller pad could be extended.) This would require 150 tons of kentledge—an expensive item. Comparison of the tests on the 3-ft. square area with those on the 6-ft. square area would eliminate most “scale” effects. There is little object in leaving the loading in position more than a few days. Consolidation-settlement in non-cohesive soils develops almost at once while consolidation-settlement in clay may take years to reach even half its ultimate value. It is true that shearing failure of a soft clay overlain by a harder stratum may develop very slowly but a single loaded area of 6 ft. by 6 ft. would not be big enough to give reliable data in such a case.

Test loading is sometimes ineffectively done. The engineer may be to blame for this. At the back of his mind is the thought that, after all, traditional rule-of-thumb methods of judging ground have proved successful if intelligently used (and no system is really foolproof) in millions of cases spread over thousands of years. In this respect test-loading seems an unnecessary addition to his duties and worries to be dealt with and is often dismissed with the minimum of attention. Perhaps he has also seen the ludicrous results that can ensue if the theory of soil mechanics is applied with too much enthusiasm and too little imagination. On the other hand he may feel that he would like an additional check or may want to avoid appearing old fashioned. Being undecided he inserts a tentative and unspecified item in the bill of quantities with unhappy results. Test loading should be done properly or not at all. The engineer should issue a drawing showing details of what he requires together with a specification and detailed bill of quantities. Loading tests are usually associated with heavy and important structures. In the case of a shallow pavement much may be learnt by observing the effects of a motor roller passing over the foundation or hardcore layer, using the heaviest roller that the ground will carry without spewing. For important road work a mobile load-testing outfit should carry out proper tests every 100 yards or so.

**Large-scale Loading Tests**

The only direct results obtainable from a loading test are the value of the ultimate bearing capacity of the pile or foundation which is test-loaded and its immediate settlement. It is therefore only a complete check in sand or gravel soils where the piles or foundations are widely spaced and where the test pile or test foundation is of the same size, the same type and at the same depth as the other piles or foundations. It does not indicate:—(1) The ultimate bearing capacity or immediate settlement of a group of piles (unless the whole group is tested at once). (2) The ultimate bearing capacity or immediate settlement of a spread foundation larger than the one tested. (3) The ultimate bearing capacity or immediate settlement of a spread foundation equal in area to the one tested if there are to be similar foundations close by, that is if the minimum clear spacing between square foundations is less than three times their width. (See Figs. 5 and 9.) (4) The long-term settlement of a clay soil.

To arrive at a final estimate of the allowable bearing pressure on a clay soil
some expert assessment of, and extrapolation beyond the results of, the test is nearly always necessary. Even on a sand or ballast soil some careful consideration is usually required before the test results fall into their proper perspective. If the final assessment of the allowable bearing pressure may reasonably be based 80 per cent. on test results with only 20 per cent. extrapolation, such tests may well be worth while, but if the tests supply only 20 per cent. of the final answer and require 80 per cent. extrapolation there would seem little point in

making them, quite apart from their cost and a possible delay to commencement of the contract.

A test-loading set-up used on several contracts is shown in Figs. 11 and 12. In those particular cases the loading was applied to a reinforced concrete pile to a maximum of 200 tons. In case the pile did not settle absolutely vertically the connection between the pile-cap and the main lever of the recording apparatus had cup-and-ball ends (taken from the push-rod of a petrol engine). A similar arrangement could be used for test-loading a spread footing. The top flange of the joist carrying the measuring apparatus should be protected from the sun as a sudden burst of sunshine may cause the joist to hog. The measuring apparatus
Fig. 12.
is shown in Fig. 12. In this case the effective stroke of the dial was 1 in. increased to 2 in. by the 2-to-1 lever. Fig. 13 shows an arrangement for test-loading a bored pile. The pile to be tested is the central pile in one group, the group being rearranged to bring one pile dead central. The anchor piles are in adjoining groups and are only 11 ft. away. This is due to practical limitations. A test load of 80 tons was required. The largest joists readily available were 24 in. by 7\(\frac{1}{4}\) in. by 95 lb. with a \(Z\) of 211 in\(^3\). Using a pair of these stressed to 12\(\frac{1}{4}\) tons per square inch the maximum span is 22 ft. At 15 tons it would be 26 ft. 6 in. but on this site the stanchions were spaced at 11 ft. centres and 22 ft. was a natural choice. Arrangements of this type may possibly cause trouble from leaky pumps or leaky jacks but they are much cheaper than using kentledge. In Fig. 13 the jack is J. Four dials D register vertical movements, these dials being carried on two steel channels which rest on supports S.

The actual data from the test set-up in Fig. 13 are given in Chapter XIII.
CHAPTER IV
APPLIED LOADS

It is assumed that the reader has a copy of the British Standard Code of Practice CP.3, Chapter V (1952) or the current issue thereof.

In the theory of structures loading is defined as consisting of two parts; dead and live. Dead load consists of fixed constant loads always present. Live load is variable between zero and some definite maximum. In the art of engineering both these convenient conceptions prove to be myths. In a city building it is not unusual to increase the dead weight on an existing floor by increasing or varying the floor finish, possibly to accommodate new electric conduits. The addition of suspended heating coils, thermal insulation or acoustic ceilings may increase the total addition to some 10 to 15 lb. per square foot. Many buildings have partitions of coke breeze plastered on both faces. The removal of these and the substitution of aluminium alloy partitioning may reduce the average dead load by 10 to 20 lb. per square foot. Office floors in the County of London in 1909 had to be designed to carry an imposed load of 100 lb. per square foot. This figure is now reduced to 50 lb. per square foot. It is therefore possible to increase the dead load on all pre-1909 floors by as much as 50 lb. per square foot without increasing the maximum design load on the foundations. If the average dead weight, including walls, was originally 180 lb. per square foot we could have the following:

<table>
<thead>
<tr>
<th>Conception</th>
<th>Dead Load</th>
<th>Live Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original</td>
<td>180 lb.</td>
<td>100 lb.</td>
</tr>
<tr>
<td></td>
<td>per sq ft</td>
<td>per sq ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>New</td>
<td>230 lb.</td>
<td>50 lb.</td>
</tr>
<tr>
<td></td>
<td>per sq ft</td>
<td>per sq ft</td>
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<td></td>
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</tbody>
</table>

Since the average live load on office floors throughout a building will scarcely exceed 10 lb. per square foot we have actually increased the loading on the foundation from 190 lb. to 240 lb. per square foot of floor area carried, an actual increase of about 25 per cent. If the engineer who originally designed the building conceived the idea that 180 lb. per square foot was the fixed and immutable value of the dead load, he completely deceived himself. Dead loading in engineering, as distinct from the theory of structures, has no fixed value. The question of the assumed imposed load is discussed from another angle in Chapter I.

Those readers who have never had to estimate an imposed-load are advised to begin by making a rough estimate of the loading on the floor of the office in which they work. If this is 200 sq. ft. in area, four men each weighing twelve stones, average less than 3 1/2 lb. per square foot. Empty furniture can scarcely average more than 3 or 4 lb. per square foot. A row of text-books 2 ft. 9 in. long
in a sectional bookcase may weigh 45 to 50 lb. and a bookcase may weigh 90 to 100 lb. per square foot on the net area of the base. One hundred double elephant tracings on cloth may weigh 16 lb. and a 3 ft. high plan chest about 100 lb. on an area of 10 sq. ft. A plan chest 6 ft. high containing 2000 tracings would weigh about 50 lb. per square foot of net floor area. The introduction of computing machinery may increase loading. Legislation to insist on a higher minimum floor area per occupant may decrease it. Having estimated the loading on the office floor, the reader should then estimate the maximum load this floor has carried in the past and the maximum it will have to carry in the future. Imposed load on a factory or warehouse floor is higher and relatively more important than an imposed load on an office floor and its maximum value during the life of the building is more difficult to forecast. The only thing we really know is that the total loading will never exceed the point where incipient structural failure obviously develops. If we can make a reasonably close estimate of the total load the superstructure can carry before this point is reached, and compare this with our estimate of the ultimate bearing capacity of the foundations, we have at any rate eliminated most of the personal problems and in a difficult case this comparison may help towards a decision. Only in the case of containers that are filled full, such as water tanks and grain silos, can there be a reasonable certainty of the maximum imposed load.

When a non-technical client says he is going to install an item of plant weighing 50 tons, what does this mean? To an academic expert it means that the plant will weigh exactly 50,000 tons from the time it is installed until the structure finally becomes obsolete. If, as the building nears completion, the client decided to install an item weighing 75 tons he would generally consider his consulting engineer half-witted if the structure would not safely carry this "trifling" addition. Such a client (quite rightly) looks to his consultant for help and understanding and does not expect to be thwarted and frustrated by him.

If an architect's preliminary ¼-in. scale drawing shows the stone cornice at the fifth floor as 3 ft. 2 in. gross width and an average of 2 ft. 11 in. gross depth what does this indicate? To the academic expert it means that the stonework, at 160 lb. per cubic foot must weigh 0:6597 ton per foot run. An experienced engineer would not be surprised if such a detail, in the final building, weighed 1 ton per foot run. The author would not be surprised if it weighed 1½ tons. The engineer may fume and swear at such increases but the fault is often his own because he has assumed, entirely without justification, that figures put forward merely as a preliminary indication of the general order of magnitude, are exact, final and binding. The ability to understand what preliminary figures from non-engineering sources really mean is largely a matter of psychology. The author doubles all doubtful cases. Even if details of the weight of mechanical plant are provided by a qualified mechanical engineer the structural engineer should not jump to the conclusion that these include impact allowance unless this is stated in writing.

The problem of estimating dead and live load, as far as the superstructure is concerned, can safely, if not economically, be dealt with by assuming high maximum values for both. For foundations on yielding soils, a close estimate of actual load is required. Over-estimation of loading on any one column may lead to unequal settlement. Wind loading is usually quite unimportant. When it
becomes a major issue its determination is difficult. It would appear that the total wind effect varies with the area, the shape and the length of periphery of the exposed object. This point is not made clear in Code No. 3 which deals specifically with area and shape. It is generally conceded that the effect varies as the square of the wind velocity. An increase in velocity from 60 m.p.h. to 70 m.p.h. means an increase in wind pressure of 36 per cent.; from 60 to 80 m.p.h. an increase of 78 per cent. It is therefore safer to assume a velocity of 70 m.p.h. with a stability factor of 1.5 than a velocity of 60 m.p.h. with a stability factor of two.
CHAPTER V

SETTLEMENT

Downward vertical movement of a foundation may be due to (1) Shear-friction failure; (2) Short-term elastic compression; (3) Change in ground conditions; (4) Mining or other subsidence; (5) Consolidation settlement.

Shear-friction failure is not to be feared if the recommendations of the Foundation Code have been followed. Short-term elastic compression may be assessed by loading tests. It is unlikely to be important except under raft conditions or where the superstructure is particularly sensitive to small vertical movements. With soils capable of safely carrying 2 tons per square foot or more it can generally be ignored. Steps to be taken against changes in ground conditions are usually straightforward except for shallow foundations such as pavements, that necessarily lie above the freezing or drying-out levels and where the ground and surface water conditions are difficult. At the end of a long dry summer a number of houses on clay soils are in distress due to drying out and shrinkage of the clay. Foundations deeper than 4 ft. are generally not affected in Britain (possibly 6 ft. in hotter climates).

Mining subsidence may not be serious if the coal seams are deep and overlying strata strong. High seams in weak strata can cause serious local differences in level within the length of a building or long tank. The amount of subsidence to be expected may be ascertained from the local Coal Board authority. The total subsidence is, of course, independent of the size or type of foundation adopted. Types of raft are discussed in Chapter XI. The term settlement should be confined to downward movement due to the escape of air or water causing the ground to pack more tightly or decrease in volume under applied pressure. Where solid particles of the soil move an appreciable distance in a direction at right angles to the applied loading this should be classified as shear-friction failure.

Unfortunately it is not standard practice in Britain to take the exact levels of foundations before and after loading. In fact the writer has not a single complete record of levels of any one of the foundations he has designed over a period of many years. The question of settlement will never be on a really satisfactory basis until accurate measurement of actual settlement becomes the normal standard routine on all contracts.

In a built-up area examination of all nearby buildings for signs of settlement is part of the routine of site investigation and if the new structure is reasonably similar to those existing the amount of settlement to be expected should be easy to estimate.

The only conditions in practice in which we may need special investigation to estimate settlement are as follows. (a)—In built-up areas where the new building is appreciably heavier per square foot of site than nearby existing buildings and where the site is clay or silt whose safe bearing capacity is less than 2 tons per square foot. (b)—On all peat sites and made up ground. (c)—On virgin sites
in open country where the safe bearing capacity is less than 2 tons per square foot. (d)—Under raft conditions where the loading exceeds the allowable bearing pressure which would normally be chosen.

The only structures which would be allowed to stand on peat or made-up ground in this country, are of the type where settlement is not important. Steel oil-tanks on the Thames marshes (if one may call a steel tank a structure) are allowed to sink several feet. They are then emptied, jacked up and packed with chalk filling, this being cheaper than supporting them on piled rafts. We are, therefore, in practice really left with only one case to solve, that of a large building under or near raft conditions on a deep bed of soft clay or silt where there is no hard bottom that can reasonably be reached within 50 ft. of the surface (100 ft. for a very large and important building). The most reliable method of making an estimate of the settlement to be expected is to look for records of settlement of similar buildings on similar sites, but unless the engineer has some personal knowledge of where to look he is not likely to find much helpful data although there must be very much relevant data scattered through the proceedings of engineering bodies.

Failing all else laboratory compression tests on undisturbed borehole samples may be carried out and the settlement calculated therefrom. Unfortunately doubt has recently been cast on the extent to which "undisturbed" samples are really undisturbed. The author is appalled by some of the worked out examples he has seen in soil mechanics literature showing calculated settlements of 8 inches and upwards. In only one case has he experienced a settlement exceeding 6-in. (actually 9-in. against an expected figure of 12-in.) and only two cases where it has reached 3 in. In all other cases it has certainly not exceeded 1 in. and in many cases has not reached 0.1 in. A relative settlement of 0.1 in. is just sufficient to split an ornamental stone frontage and crack a reinforced concrete framed structure. A general settlement of 1 in. would show where the pavement meets the entrance to the building. In sharp contrast to these enormous theoretical settlements, we have a copious flow of theoretical and technical literature on framed structures (some exploiting the old French idea of introducing imaginary hinges) which is clearly based on the assumption of incompressible foundations. Do some of our structural experts then always build on outcrops of solid granite and some of our soil mechanics experts only on deep mudbanks? It is a pity that these opposite camps cannot get together and produce something useful within normal engineering practice.

The effect of settlement on a framed structure is discussed by the author in his books "The Displacement Method of Frame Analysis" and "Reinforced Concrete Arch Design".
CHAPTER VI

CONSTRUCTIONAL OPERATIONS

The author does not pretend to review modern methods of construction as such. This would require at least one complete volume in itself. The object is to review these operations as far as they may affect the design, specification and bills of quantities for foundation work. Design should always be correlated to construction and this is usually more important in work below ground level and most important in work below ground water level. Design should also be related to the plant which is likely to be available. On very large contracts the problem is simplified by the fact that the engineer can design and build special plant to meet the exact requirements of one particular job. On works of fairly high value, good, new, general-purpose plant is available and it is for works in this category that the engineer should have a general knowledge of modern plant. On very small jobs only odd items of old plant, not required on more important work, are usually found and on such jobs operations entailing heavy lifting or the placing of a large volume of concrete in a short time (unless ready-mixed concrete is available) should be avoided. If unusually heavy plant is required on a very small job this fact should be pointed out in the specification and covered by items in the bill of quantities.

Another handicap on small work is usually the lack of technical supervision which generally varies in amount directly in proportion to the value of the contract. This also should be remembered in designing small jobs. Straight rods running from end to end of all members and a uniform spacing of rods throughout are advisable on work where the only supervision is that of a junior ganger. This may increase the amount of reinforcement but is much cheaper than employing a qualified supervisor. Clarity and completeness of detailed drawings are essential on all contracts.

Foundations in Bulk Excavations

With heavy modern plant there is always the danger of excavating below the finished level or of damaging the ground by running heavy wheel loads over it. In both cases the specification must make it clear that the contractor has to put matters right at his own expense by removing damaged ground and filling back with selected material, hardcore or mass concrete as the case demands. On a sand or ballast foundation the damage may not be evident. The operator, in order to get right down to final level, lets the tines of the bucket drag below this level thus loosening the ground for some 6-in. or 12-in. below. In some cases this may be put right by running a heavy roller over the site but it is better to stop the mechanical excavation say 6-in. above finished level and trim by hand. This should be made clear in the specification, noted on the drawings and covered by an item in the bill of quantities.

It is standard practice in this country to measure excavation net and to specify that the contractor must quote rates that cover him for any necessary additional

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CONSTRUCTIONAL OPERATIONS

excavation and back filling, for timbering and for normal pumping (see "Standard Method of Measurement of Civil Engineering Quantities"). The author thinks the American "pay-line" method is much better where the engineer, who must have a working knowledge of constructional methods, draws a reasonable line for the gross excavation needed and "pays" for everything above that line. It is, however, difficult to break a practice once it has been established and however feeble this may seem, it may be better to conform to local custom. On a congested city site the rate of excavation is often controlled by the speed with which it can be carried away in lorries not by the speed at which it can be excavated.

Foundations in Small Pier Holes

Excavation down to lowest occupied level (say underside of basement floor) is removed mechanically but, unless the foundations cover a large percentage of the site, excavation below this line is usually by hand-operated tools.

As an example suppose we have a series of foundations 5 ft. by 5 ft. at 30 ft. centres both ways as in Fig. 14 and that foundation level is 8 ft. below the underside of basement floor. Normally the excavation is taken down in timbered pier holes 5 ft. by 5 ft. by 8 ft. deep. If mechanical plant is available it may be quicker to dig untimbered holes with sloping sides on lines bx, rough-shutter the piers and fill back. It may even be quicker to dig out the whole site to the level x-x-x-x and fill in again. Many points must be considered. If the foundation level is below ground-water level and if there is much water the prospect of producing a sizeable lagoon by excavating down to x-x-x-x, particularly in a built-up area, is unthinkable. If the ground between basement and foundation level is of doubtful quality it may be an advantage to clear it out and fill back. On a crowded site pier holes give most working and storage space as they may be tackled seriatim. Unless the back fill is consolidated hardcore or mass concrete some settlement and cracking of the basement floor is inevitable (unless the floor is suspended) and this point generally decides the issue.

In firm ground it was possible to dig pier holes 4 ft. square down to a depth of 30 ft. lining the sides with corrugated iron supported by light timber frames. With average unskilled labour placing more reliance on mechanical aid and less on manual skill, 5 ft. square or 4 ft. by 6 ft. is now the minimum size which should be attempted in England and 5 ft. 6 in. by 5 ft. 6 in. is a safer figure.

Timbering

Most regulations now (quite rightly) insist on the provision of timbering for all excavation exceeding 5 ft. deep. In dry firm ground boards 4 ft. long and of 1 4 in. by 9 in. section are supported by 3 in. by 9 in. walings and struts as in Fig. 15. This presents no obstacle to constructing the foundation as each lift of timbering can be removed immediately before each lift of concrete is placed.

In more difficult ground runners may be used. These are pointed lengths of 2-in. x 9-in. or 3-in. x 9-in. timber in lengths up to 12 ft. They are supported by small single timber wedges called "pages" bearing against a timber frame or frames. Progress is made by removing the pages from one runner at a time, driving it down a few inches, paging it out again, that is redriving the pages, excavating a few inches of ground and repeating the process for the next runner, thus working right round the pier hole. Runners were formerly driven by sledges
CONSTRUCTIONAL OPERATIONS

but are now usually driven by pneumatic tools. Runners longer than 12 ft. are difficult to handle and holes deeper than this require two flights as in Fig. 16 the lower flight being set in some 9 in. or 12 in. Timber frames are provided every 3 ft. or 4 ft. depth. These frames remain stationary. If the ground is worse than expected or the timbermen not too skilful the runners may be forced tightly against the frames and the system becomes locked together. If the excavation is within a foot or so of completion it is sometimes possible to reach it by arming all available men with sledges and driving everything down at once by sheer brute force. Skilled and experienced timbermen are becoming scarce and this system of timbering is dying out. When placing concrete the timber is usually removed. If the ground is not too bad the runners may be pinched up so that their toes are just about level with the concrete surface. In poorer ground the concrete may be placed in separate lifts of about 2 ft. and allowed to set, the runners being pulled up by pole and chain. If a 2-ton crane is available the lifts of concrete may be increased to 4 ft. If all else fails the runners are left in, only the frames being salvaged.

Steel trench-sheeting has now largely replaced timber runners. It is easier to drive and draw and is stronger than 2-in. timber. Details are given in the maker's handbook.

Even if reinforced concrete work has to be carried out at the bottom of a small deep pier hole, and this should be avoided wherever possible, the passing down of the rods usually presents no problem. Apart from dealing adequately with the ground water no particular points arise in connection with the design, the specification or the bill of quantities.

Foundations in Bored Pier Holes

A method of boring holes of 5 ft. diameter or even up to 10 ft. diameter, has now been developed. Equipment for boring up to 8 ft. diameter can be mounted on a lorry chassis and it is claimed that depths of 110 ft. may be reached in suitable ground. On a site requiring a large number of small mass concrete piers this method might be very useful and might also be better than groups of bored piles of small diameter.

Foundations in Large Pier Holes

Unless steel walings are used intermediate struts are needed. If the bottom of the pier hole is filled with mass concrete these present no difficulty but if it is essential to construct reinforced concrete work, both struts and walings may constitute serious obstructions. It is usual to work to a spacing of about 8 ft. for the struts, to allow crane buckets, etc., to drop down easily between them. Fig. 17 shows an example of what may, and unfortunately sometimes does, happen. An underground tank or chamber carries column loads at the four corners. The walls act as vertical beams the bottom being a square panel 16 ft. by 16 ft. overall, designed to spread the whole load evenly over the ground. This bottom is therefore a heavy slab strongly reinforced with sizeable splice rods running up into the walls. Following British custom the bill of quantities allows only for a hole 16 ft. square but allows for shuttering to the outside of the walls. The foreman digs sheets and timbers as shown in the left-hand half of the section,
puts in 3-in. mass concrete and then thinks what to do next. He discovers that the rods in the floor are nearly 16 ft. long whereas he has only 7 ft. by 7 ft. clear between his struts, or about 10 ft. maximum on the diagonal, through which to lower them. This is awkward but can be coped with. The splice rods are not so easy. Usually in such cases they are bent sharply down at right-angles and straightened (!) after the floor slab has been concreted and the lower walings and struts lifted or removed, the result being kinks in the rods and cracks in the newly-set concrete. A better solution is to increase the thickness of mass concrete from 3 in. to 12 in. and lift the bottom waling as soon as this hardens. This still leaves the difficulty of placing the vertical rods in the walls. By keeping the top waling 3 in. clear of the sheeting with hit-and-miss packing it may be possible to pass them all behind and in front without bending and without much re-spacing. But if this has not been thought of early enough it means another set of walings and struts. There follows the inevitable argument about who is going to pay for the extra excavation, mass concrete and timbering.

A better solution is to adopt the “pay-line” method. Show a “pay-line” hole 18 ft. square and include quantities to cover this. Add an item for shifting the timbering, or boxing out the struts where they pass through the walls, as concreting proceeds and include for filling back outside the walls (with mass concrete if necessary). In a very special or very difficult case the central struts could be omitted by using steel walings but this would certainly require a special item in the bill of quantities. Occasionally the sheeting cannot be recovered but it is standard practice to include a provisional item in the bill to cover this.
Foundations in Trenches

The problems that may arise when designing foundations to be constructed in trenches are similar to those in large pier holes—the interplay of the timbering and the reinforcement or structural steel. Speaking generally steel rods, including bent-up shear rods, are much easier to thread through struts than might at first be imagined. A mild steel rod with a yield point of 40,000 lb. per square inch may be curved to a radius of 375 bar diameters without causing permanent deformation, that is, a radius of 31 ft. 3 in. for a 1-in. diameter rod. Any one strut which is in the way may be removed by putting a new strut each side of it. Fig. 18 shows a foundation beam in a trench. It would be possible to concrete the slab with the lowest frame of walings and struts still in position but these might make it very difficult to get the bottom main steel down below them and the struts would interfere with the links and any bent-up bars. All this may be avoided by increasing the depth of the mass concrete from 3 in. to 6 in. to hold the toes of the sheeting. As soon as this is hard the lowest walings and struts can be withdrawn. The lateral pressure from average soil may be taken at 20 lb. per square foot per foot of depth. After the lowest walings are out the sheeters would span a distance $H$ ft. vertically. If the depth from ground level to the middle of $H$ is $D$ ft., the average pressure is about $20D$ lb. per square foot and the bending moment in the sheeters is about $20D \times \frac{1}{6}H^2 \times 12$ lb.-in. per foot width of sheeter.

With 3-in. timber runners the greatest value of $H$ is $24-6/\sqrt{D}$, with 2-in. timber runners $H$ is $16-3/\sqrt{D}$, and with steel trench-sheeting ($Z=0-53$ in.$^2$ per foot width) $H$ is $18-8/\sqrt{D}$. If $D$ is 10 ft. these values are 7 ft. 9 in., 5 ft. 2 in. and 6 ft. respectively for average soil, or about 6 ft. 3 in., 4 ft. 3 in. and 5 ft. for heavy soil.

If instead of a beam we have a continuous wall extending right up to ground level and if it is not practicable to remove all struts before concreting, those
remaining may be boxed round. If there is some strong objection to this, concrete struts could be placed and concreted in.

The writer is of the opinion that details of this kind should be discussed before work commences and items should be included in the bill of quantities to ensure that the contractor is paid for any additional work which he could not reasonably foresee.

**Foundations in Small Cofferdams**

Cofferdams, except on rock bottoms, are now of steel sheet-piling. The writer would here define "small" as not exceeding 25 ft. deep and not more than 25 ft. wide. Generally they are only an aid to foundation work but sometimes some or all of the piles are left in and constitute part of the permanent work. Having decided that steel sheet-piling is necessary the type of pile must be chosen. Very light sheet-piles weighing about 10 lb. per square foot are only suitable for driving in sand or clay. Attempts to drive them into ballast can lead to a complete fiasco. The writer knows of no rule by which the minimum weight of sheet-piles can be deduced from the coarseness of the ballast. In addition most quarternary and some tertiary gravel beds consist of alternating beds of coarse pebbles, sand and fine pebbles, and a borehole or trial pit may give a misleading picture of the driving resistance likely to be met. If the site investigation has included the driving of test sheet-piles the answer is known but this is not usual practice. Generally a pile of about 25 lb. per square foot (such as a No. 2 Larssen) will drive through the ballast of the "1 1/2 in. down" variety found in fairly flat river basins, without serious bending or deviation and piles of this weight are usually adequate up to a length of 30 ft. or 40 ft. in easier ground.

If the sheet-piles finish in solid clay the length of cut-off is easily determined. In Fig. 19, D may be made of 0.2H, or 3 ft. whichever is greater. But if the piles finish in water-bearing sand or ballast as in Fig. 20 a much deeper cut-off is required. The Foundation Code No. 4 suggests values of D from 0.4H to 0.7H depending on the ratio of H to W. The Code then continues "...Boiling is more likely to occur adjacent to the piling..." thus suggesting that the width has nothing whatever to do with the problem. The writer suggests a value of $D = 0.8H$ for all ratios of $W$ to $H$. 

![Fig. 19.](image)

![Fig. 20.](image)
The maximum spacing of walings for different types of pile will be found in the manufacturers' handbooks. With a reasonable section of pile there should be no difficulty in keeping the bottom waling 5 ft. above foundation level which will accommodate splice bars of normal height and no difficulty arises while concreting the floor. If the foundation comprises a hollow chamber, as in Fig. 21, it now becomes necessary to shift the bottom frame before further steel fixing and concreting can be done. If the height $H_2$ is not sufficient to overload the piling (say about 18 ft. for No. 2 Larssen in average ground) this lower frame can come out as soon as the bottom slab is a few days old.

Alternatively a temporary frame can be put in where shown arranged with hit-and-miss packing between the waling and the piles so that vertical reinforcement may pass behind and in front. The deciding point for this is the strength of the partly completed walls. If these walls are designed to span horizontally then they will support the sheet-piling when fourteen days old and the vertical
rods in them are merely light distributors. But if the walls span vertically from floor to roof without an intermediate floor, the vertical rods are heavy and spliced well into the roof and the walls cannot support the earth pressure until the roof is fourteen days old. Probably the easiest way is to make the cofferdam 2 ft. 6 in. wider than the minimum, increasing the billed quantities accordingly, and leave the original struts and walings in place until the end. Interference from the top waling may be eliminated by using steel joists placed outside the piles and welded or bolted thereto as in Fig. 23 (page 38).

It is possible to put new walings and struts against the inside face of the reinforced concrete walls (marked A in Fig. 21) but this is a delicate operation with newly-cast concrete and is not recommended. Boxing out for struts should be slightly wider outside than inside and slightly higher inside than outside. For a strut 9 in. square in a 12-in. wall the hole may be 14 in. wide by 12 in. high on the outside and 12 in. wide by 14 in. high on the inside of the wall.

Conditions on site must play a big part in deciding the exact method to be adopted. It may even be expedient to modify or strengthen the design. What is essential for smooth and rapid construction is that difficulties should be foreseen and covered by clear and fair items in the specification and bill of quantities. Plate I (facing page 58) shows a view inside a small cofferdam.

Ground Water

Mass concrete may be placed under still water and in some cases this is better than attempting to pump. Such a case (actually a mass concrete foundation for a gasworks plant) is shown in Fig. 22. A lot of water was pouring into the hole about half-way down. In some cases this situation might be dealt with by digging an intercepting drain round the site but here it was not possible. Steel sheet-piling was also out of the question. The hole was allowed to fill and the concrete was placed by tremie.

When placing concrete underwater the cement content should be increased (say 1:6 instead of 1:8) and any possible current stilled by baffles—timber, old corrugated iron or asbestos-cement sheets. Theoretically the lower end of the tremie pipe should be kept just below the surface of the concrete. Usually this chokes the flow and the end has to be pulled up above the surface and the pipe has to be shaken. Again a small diameter tremie pipe contains less water when starting operations and ensures a higher average velocity of downflow for any given output of mixer. In practice smaller pipes tend to choke. A 6-in. pipe for richer mixes with rounded aggregate and an 8-in. for poorer mixes with larger or angular aggregate are about the minimum sizes. Generally a better method is to place a series of 2-in. grouting tubes at different depths, fill round them with graded stone—say 6-in. to 4-in. and pump sand and cement grout (say 3 cement to 1 sand) into the voids. Tremied concrete tends to settle in layers with laitance between and will carry
reasonable vertical loads but little lateral thrust. Possibly the discharge end of a concrete pump or pneumatic concrete placer might prove superior to the ordinary gravity tremie but the writer has no data on this. Bottom-opening buckets may be used on larger works where a crane is available.

When founding in a water-bearing stratum the water may be cut off by driving steel sheet-piling round the site into an impervious clay stratum below, provided the sheet-piling is heavy enough to cope with the coarsest ballast or hardest stratum encountered and only a small number of piles spring out of their clutches. One exception to this general rule is worth noting. A dog-leg pit 31 ft. × 14 ft. × 21 ft. deep was built within a cofferdam on a site consisting of marsh overlain by 10 ft. of old made-up ground. (See Plate I facing page 58.) It was de-watered normally and easily by a single 4-in. pump. Shortly afterwards a second pit was built on a similar site a few hundred yards away by the same contractors using the same type and length of pile. Water gushed in through every open joint. It was necessary to pump three to four million gallons per day before the water could be lowered and the joints caulked. This second pit must sit over a geological fault or old watercourse now quite invisible from the surface. (See Fig. 188, page 177.)

If the bottom ends of the sheet-piles finish in hard chalk or shaley marl the driving may cause widespread splitting of the hard stratum and serious inflow through the cracked ground. These conditions are not likely on a land site but may be a major problem on a submerged site.

Reinforced concrete foundation work can only be successfully carried out if water is excluded. For the usual run of small foundations below ground water level this entails some kind of pumping. If the volume of water is very little a small drainage grip is dug round the outside and a small sump hole is dug in one corner of each excavation and the water is then baled or pumped out by a diaphragm pump. If these means are insufficient then an entirely independent sump should be put down at least 10 ft. away from and at least 3 ft. deeper than the foundation. With a group of foundations a central pumping sump will often lower the ground-water level for the whole group. The writer knows of no method of calculating beforehand what pumping will be required or over what area it will be effective. In some cases the ground water seems to be merely the accumulation of months of rain and once it is pumped down it rises again very slowly. The same variation is found in old wells sunk into apparently similar strata. All sand and gravel strata vary in coarseness and in compactness. On one site a torrent of water came through the top 3 ft. of a layer of ballast. One 4-in. and one 6-in. centrifugal pump working together in a hole 9 ft. by 28 ft. could only lower the water level by 12 in. The water was clearly coming from a river about 150 yd. away.

If water coming up through the bottom of a cofferdam cannot be lowered by pumping, the worst conditions could be dealt with as shown in Fig. 23. If a reinforced concrete foundation is needed at a depth of H ft. below ground-water level in an open water-bearing ballast stratum through which steel sheet-piles can be driven the ground inside the cofferdam could be removed by grabbing and a scaling layer of concrete put down underwater. To balance the upward pressure of water this layer should in theory be at least 0.8H deep. Alternatively grouting tubes may be driven down and the ballast grouted solid to form this
sealing layer. Theoretically this grouting (or chemical consolidation) could be extended and the sheet-piles dispensed with but it may be difficult to get a firm quotation for solidly grouting any specified volume of ground. It is usually easy to get a price for pumping a specified volume of grout into the ground at specified levels but if this finds a natural pipe leading away from the site the method may fail. On one site a ballast layer accepted a large volume of grout. This found its way into an existing deep drainage trench and grouted the back filling solid over a considerable length. Unfortunately this was not the object of the operation. If the conditions are not quite as bad as those in Fig. 23 but if the water coming through the bottom can only be lowered by continuous heavy pumping it can be eased by putting down about 2 ft. of concrete as shown in Fig. 24 leaving vent holes at intervals and leading the water to a sump.

It is everyday practice to pump drinking water from deep boreholes but the practice has not extended to dewatering foundations. With improvements in mobile boring and drilling rigs and the fact that most sites are now explored by boreholes it may be that pumping from site exploration boreholes will eventually be used for dewatering the site. Where a deep layer of running sand lies over water-bearing chalk, a large diameter borehole sunk into this chalk might prove a most effective pumping sump.

Well-points are sometimes effective in fine sand that yields its water slowly. They are excellent if driven into position. In the writer’s experience the practice of jetting the spear-points down disturbs the ground, brings down the timbering in nearby trenches and covers the site with sand and water. The pump suction is attached to the top end of the well-point tubes and they thus have a limiting theoretical depth of 32 ft. or say 25 ft. in practice. Continuous pumping always
draws a percentage of fine material from sand or ballast strata and this will in
time increase the amount of water to be pumped; it has been known to double.

Pumping confined to a single diaphragm or small centrifugal pump is usually
included in the billed rates for excavation. Pumping in excess of this should be
covered by provisional items in the bill of quantities. This again raises the point
whether site investigation on water-bearing sites should include pumping tests.
Technically the idea is a good one but there may be severe political repercussions.
If a general contractor arrives on the site to carry out a minor preliminary opera-
tion it may complicate the invitations to tender for the main contract. If the
volume of ground water may vitally affect the design of the permanent foundation
work then all reasonable steps should be taken to measure it as soon as possible.

If the contractor starts a deep excavation in timbering and is beaten by the
volume of water and then has to drive steel sheet-piling there is likely to be some
difficulty in dealing with the extra cost. If the engineer specifies steel sheet-
piling and the volume of water proves to be very small he has incurred unnecessary
expense. The present tendency to more frequent use of steel sheet-piling (driven
by automatic diesel hammers on those sites where the client cannot supply steam
or compressed air) has simplified this problem. Some engineers seem to think
the contractor should gamble on the outcome but a dissatisfied contractor, who
thinks that the engineer has misled him into quoting an impossibly low price,
may look for ways and means of getting his own back. The engineer should try
to draft a specification and bill of quantities fair to all parties.

It has often been suggested, although very seldom tried, that electro-endos-
mosis could be used to help de-water fine silts. With two electrodes buried in the
earth the water will tend to travel towards one, probably the cathode. This
method was applied to de-watering peat some fifty years ago but was not then a
commercial success.

Concreting

The importance of a minimum rate of continuous concreting for massive
work is discussed in Chapter VII. A minimum rate of rise in the concrete surface
of 1 ft. per hour for all reinforced work is advisable. Owing to minor delays on
site such as shifting runways or chutes, waiting for excavating or hoisting plant
to cross the site, rigging tarpaulin covers, etc., concreting plant does not average
its full working capacity. Twenty batches per hour from a mixer is a fair assump-
tion. A 4-in. concrete pump may be assumed to average 6½ cu. yd. per hour and
a 6-in. pump about 15. The engineer should decide the minimum rate he requires
and cover this by a clause in the specification and an item in the bill of quantities.
The allowable location of construction joints should also be made clear.

The question of timbering, waling and strutting in relation to reinforced
concrete work has already been discussed. It is always difficult to place concrete
through a forest of walings, struts, pump suctionss, existing drains, cables, etc.
If these cannot be eliminated or substantially reduced by reviewing construction
methods before commencing operations the contractor's attention should be
drawn to the fact in the contract documents. In a difficult case on a restricted
site, drastic revision and simplification of the design of the foundations may be
advisable.

Concrete in foundations is easily covered and kept damp.
The general problems of choosing the correct concrete materials, stacking, gauging, wheeling and placing are not peculiar to foundation work and are not discussed here. An elementary review of them is given by the writer in "Construction in Reinforced Concrete".

**Shuttering**

Even if the concrete cannot be cast against the earth or against the sheeting, any shuttering required may generally be supported off the timbering or off the sides of the excavation and is usually simpler and cheaper than shuttering for the superstructure. Tables of strength for timber shuttering are given by the writer in "Construction in Reinforced Concrete".
Chapter VII
Materials of Construction

Concrete

Of all materials for construction concrete is the most variable and the one about whose actual strength we know least. Structural steel to B.S. No. 15 has an ultimate tensile strength of 28 to 33 tons per square inch and it is virtually impossible for any material with a tensile strength of less than 26 tons per square inch to find its way into any structure. The writer has known of two cases where a batch of so-called concrete consisting of shingle, sand and water was placed in column boxes and not noticed until the shuttering was removed. For every case of this kind that comes to light there may be a hundred that are cut out, made good and hushed up.

The design of reinforced concrete is dealt with officially in the British Standard Code of Practice No. 114 and it is assumed that the reader has a copy of the current version. Unfortunately the Code assumes "...that the execution of the reinforced concrete work is carried out under the direction of a qualified supervisor". This means in effect that the Code does not apply to small and medium-sized contracts, as continuous technical supervision is not provided in this country for such work. On a very large contract with fully qualified and experienced engineers in continuous charge of all operations, advice from a Code of Practice seems unnecessary. On smaller jobs where advice is vitally needed this Code apparently does not apply. Is the reinforced concrete industry, therefore, better or worse off than it was before these Codes of Practice were published? The industry certainly began and developed on its own.

The processes of making concrete can be expressed by the "concrete chain" in Fig. 25. The first seven items are checked by the works test-cubes. Even if the engineer has aimed too high and these cubes do not come up to specification we do at least know where we are up to this point. The last five items are blind and we have only a vague idea of how the strength of the finished concrete compares with the works test-cubes. We know that structures which are reasonably well designed and built stand up to their specified working loads and we may therefore deduce that the average final strength of the finished concrete does not fall below half the average works test-cube strength. Provided that the concrete is adequately protected while setting, and this on a wet site may mean continuous pumping for some twelve hours, item 11 in the chain is usually simple in foundation work and little or no trouble is to be expected from item 12. (It is assumed that the ground has been tested for harmful impurities.) Items 8, 9 and 10 usually present simpler mechanical problems in foundations than in superstructures but time is often more important, for foundations are often massive and suitable places for construction joints are few. This means that larger volumes of concrete must be placed continuously. The problem is simple when the plan area is small as in deep piers, but most reinforced foundation work is spread over a large plan area. Some specifications call for concrete to be placed within twenty
minutes of first wetting but this may be wholly inadequate for large blockwork. The minimum setting time for normal Portland cement is now forty-five minutes at standard temperature. Most cements set in about three hours but the heat generated in a large block may shorten this time. Fig. 26 shows a slab foundation being concreted in slices working from the left-hand end. In order to make a solid block each slice must be worked down into and married with the slice below (analogous to the "sheeps-foot" roller effect in placing earth embankments) and each slice must therefore be placed within forty-five minutes of first wetting the previous slice. This means in effect only 22 1/2 minutes per slice. If the slab is \( b \) ft. broad and if the slices are taken as 6-in. deep, each slice contains \( \frac{1}{2}bl \) cu. ft. and it is necessary to place \( \frac{60}{22.5} \) slices per hour equal to \( \frac{60}{22.5} \times \frac{bl}{2} = 1.33bl \) cu. ft. per hour. If \( b = 12 \) ft. and \( l = 24 \) ft. then \( 1.33bl = 384 \) cu. ft. = \( 14 \frac{1}{4} \) cu. yd. per hour. Assuming a mixer turns out twenty batches per hour, although most mixers are reputed to be able to turn out thirty, this means 0.71 cu. yd. per batch. Two \( 14/10 \) mixers should turn out 20 cu. ft. or 0.74 cu. yd. per batch. If ready-mixed concrete is available in 3\( \frac{1}{2} \)-cu. yd. loads then four loads per hour are required.

For very large masses such as concrete dams special slow-setting low-heat cements are available but these are not usually available for normal foundation work. It is therefore necessary in work of this type for the engineer to specify the minimum rate of concreting required with a corresponding item in the bill of quantities, and to insist on a workable mixture that will ensure the knitting together of successive batches of concrete.

Another reason for insisting on a workable mixture is the high shearing stress and high bond stress due to short spans and heavy loading. Grave doubts have been raised about the bond strength of dry high-cube-strength concretes in this respect. For long-span superstructure members where self-weight and overall depth become increasingly important it is imperative to take all reasonable steps to ensure a high compressive strength. In many foundations a lower working stress in the concrete together with increased lever arm and less steel, is a positive advantage. This again places the emphasis on workability and not on test-cube strength.

Having decided on the type of concrete, the engineer has then to fix the permissible stresses in his design and here the main difficulty arises at the outset. What do we mean by the term "test-cube strength"? Even on a large easy contract where supervision and testing are at their simplest, great variations occur. With a small-section superstructure member a single weak batch will ruin its strength. With massive foundation work a single poor batch would make little or no difference. With superstructure design we are, therefore, mostly concerned with minimum cube strength; with foundations mostly with average
cube strength and this greatly simplifies the outlook for we are not so worried by
the odd batch that falls below all others. There are three more variables to be
taken into account, the mechanical method (if any) provided for consolidating
the concrete, the amount and quality of technical supervision available, and the
accessibility of the design. The strength of concrete as placed in the structure is

\[ S_\varepsilon = S_{te} \times f_1 \times f_2 \times f_3 \times f_4 \]

in which \( S_\varepsilon \) is the strength of finished concrete (Fig. 25), \( S_{te} \) is the works test-cube
strength, \( f_1 \) is the workability factor, \( f_2 \) is the consolidation factor, \( f_3 \) is the super-
vision factor and \( f_4 \) is the accessibility factor. These four factors all vary between
unity and zero. Only under ideal conditions do they all equal unity. Factors
\( f_1 \) and \( f_2 \) are interdependent. With mechanical vibration a 1-in. to 2-in. slump
may be best, while a 4-in. to 5-in. slump may be necessary for hand placing.
This may mean that the works test-cubes have a higher water-cement ratio than
the preliminary cubes (if any) and a lower strength. The factor \( f_3 \) varies with the
size and value of the contract, being 1·0 for a job worth £1,000,000, possibly
0·75 for one worth £10,000 and 0·5 for one worth less than £1000. For small work
in underdeveloped countries, values of 0·75 to 0·5 are reasonable. The factor
\( f_4 \) depends usually on the spacing of the reinforcement and to a lesser degree on
the minimum cover of concrete outside the steel. An engineer with contracting
experience normally ensures a high value for \( f_4 \) by arranging his bars with plenty
of space for the concrete to pass between and around the bars. Occasionally
circumstances make a high percentage of steel unavoidable and such cases may
occur in some odd corner difficult of access. If the difficulty cannot be side-
stepped by substituting structural steelwork a value of 0·75 to 0·5 for \( f_4 \) must be
faced. Although there is far too much literature on test-cube strength there is
little or none on the probable values of these four factors.

Engineers whose practice lies wholly in English cities have no experience of
coping with really poor aggregates some of which are barely usable in mixtures
leaner than 1:1:2, sometimes with alarming heat generation and shrinkage. In
the London area the author normally specifies a nominal 1:1\frac{1}{4}:3 mixture, that is
1 cwt.:1\frac{1}{4} cu. ft.:\frac{3}{4} cu. ft. with a permissible variation to 1 cwt.:2 cu. ft.:\frac{3}{4} cu. ft.
Works test-cube strengths at 28 days are specified as 3750 lb. per square inch
minimum, say 5000 average, with mechanical consolidation or 3000 minimum for
hand placing (allowing a higher water-cement ratio). A recent series of works
test-cubes in the London area showed a minimum of 2150 at seven days (3750
average) and a minimum of 3410 at 28 days (5200 average). A figure of 3750
lb. per square inch justifies a working stress in bending of 1000 to 1250 lb. per
square inch depending on the assessment of factors \( f_1, f_2, f_3 \) and \( f_4 \). In under-
developed countries preliminary test-cubes should be made and works tests
specified accordingly. With a minimum works test-cube strength at twenty-eight
days of 2500, a working stress in bending of 600 to 750 lb. per square inch may be
assumed depending on factors \( f_1, f_2, f_3 \) and \( f_4 \). Some engineers seem to think
that it is clever to demand very high strength in circumstances where it is doubtful
if these are attainable. When the works test-cubes fail to reach the specified
strength the contract is in trouble.

Tables of bending strengths of rectangular sections are given in Appendix A
(page 215).
Structural Steel

Structural steelwork is generally designed to British Standard No. 449 and it is assumed that the reader has a copy of the current version (1959). Although clear on most points it is most enigmatical on the questions prominent in foundation design, namely, shear, web bearing and web buckling. The strength of the webs of grillage joists under heavy concentrated loads must be well within safe limits.

Shearing Strength.—The shearing strength of compound joists or plate-girders may be limited by the flange rivets. Makers’ handbooks, from which most structural steelwork is “designed”, give a cryptic “Rivet Pitch Coefficient” and it may be as well to give the simple theory here.

If we have a symmetrical section as shown in Fig. 27 the shear per unit length of beam at plane $x_1-x_1$ is

$$\frac{F b_1}{2 I} (h_1^2 - h_2^2)$$

where $F$ is the total shear on the beam and $I$ is the net geometrical moment of inertia of the whole cross-section. The shear per unit length at plane $x_2-x_2$ is

$$\frac{F}{2 I} [b_1(h_1^2 - h_2^2) + b_2(h_2^2 - h_3^2)].$$

The shear per unit length at the neutral axis $x_3-x_3$ is

$$\frac{F}{2 I} [b_1(h_1^2 - h_2^2) + b_2(h_2^2 - h_3^2) + b_3(h_3^2 - 0)].$$

Actual rivetted compound joists or girders are, of course, not quite symmetrical as the compression flanges are solid, but the area of the tension flanges are “less holes” unless smaller plates are used on the compression flange, which is unusual.

Consider the net section as in Fig. 28; this figure shows only the top half of the section which is symmetrical about $x_4-x_4$.

Shear per unit length at $x_1-x_1$ is

$$\frac{F}{2 I} [12.125(26^2 - 24^2)] = 606 \frac{F}{I}.$$

Shear per unit length at $x_2-x_2$ is

$$606 \frac{F}{I} + \frac{F}{2 I} [10.75(24^2 - 23.375^2)] = 765 \frac{F}{I}.$$
Shear per unit length at $x_3-x_3$ is

$$765 \frac{F}{I} + \frac{F}{2I} [1.55(23.375^2 - 18^2)] = 937 \frac{F}{I}.$$ 

Shear per unit length at $x_4-x_4$ is

$$937 \frac{F}{I} + \frac{F}{2I} [0.625(18^2 - 0)] = 1038 \frac{F}{I}.$$ 

If $F$ is in tons and $I$ is in inch$^4$ units then these values are the shear in tons per linear inch of girder.

The net $I$ of the whole section in Fig. 28 is

$$\begin{align*}
2 \times 12 &\times 125 \times 2^3 \times \frac{1}{12} = 16.2 \text{ in.}^4 \\
2 \times 12 &\times 125 \times 2 \times 25^2 = 30,300 \\
2 \times 10 &\times 75 \times 0.625 \times 3^3 \times \frac{1}{12} = 0.4 \\
2 \times 10 &\times 75 \times 0.625 \times 23.69^2 = 7530 \\
2 \times 1.55 &\times 5.375 \times 3^3 \times \frac{1}{12} = 40.2 \\
2 \times 1.55 &\times 5.375 \times 20.68^2 = 7120 \\
2 \times 0.625 &\times 18^3 \times \frac{1}{3} = 2430 \\
\hline
\text{Net I} = 47,437
\end{align*}$$

The shears per 1 in. length of girder are as follows.

At $x_1-x_1$: \[606 \frac{F}{I} = \frac{606F}{47,437} = 0.0128F.\]

At $x_2-x_2$: \[765 \frac{F}{I} = \frac{765F}{47,437} = 0.01615F.\]

At $x_3-x_3$: \[937 \frac{F}{I} = 0.0198F.\]

At $x_4-x_4$: \[1038 \frac{F}{I} = 0.0219F.\]
If $F$ is 180 tons, the shearing forces per 1-in. length are as follows

- At $x_1-x_1$: 2.30 tons
- At $x_2-x_2$: 2.90 tons
- At $x_3-x_3$: 3.56 tons
- At $x_4-x_4$: 3.94 tons

The last result may be checked from first principles. The lever-arm of the section is something less than 50 in. so the shear per inch at the neutral axis $x_4-x_4$ is something more than $\frac{180}{50}$, that is, something over 3.6. If the web plate is sufficiently stiff to justify a shearing stress of 6.5 tons per square inch the shear per inch at the neutral axis must not exceed $0.625 \times 6.5 = 4.06$ tons per inch. Using $\frac{3}{4}$-in. rivets, and taking their net diameter, their safe bearing resistance in the $\frac{1}{4}$-in. web-plate is 6.56 tons each.

To take 3.56 tons per inch at section $x_3-x_3$ requires $\frac{3.56}{6.5} = 0.543$ rivet per inch. As there are two rows of rivets, this is 0.272 rivet per inch in each row or one rivet per 3.67 in.

In Fig. 28 the shear at $x_1-x_1$ is 2.30 tons per inch.

Taking the value of a $\frac{1}{4}$-in. rivet in single shear as 3.61 tons, we require $\frac{2.30}{3.61} = 0.637$ rivet per inch. As there are four rows of rivets this means one rivet per 6.27 in. It is standard practice to make the rivetting in both legs of the angles the same, so in this case $\frac{3}{4}$-in. rivets at 3$\frac{1}{4}$-in. pitch throughout would be used. This means that the value of $p$ in Fig. 28 is 3$\frac{1}{4}$ in.

In the simpler case of the compound joist in Fig. 29, the shear per inch length of beam at $x_1-x_1$ is

$$F \frac{[11.0625(13^2 - 12^2)]}{2I} = 138 F I.$$

If $F = 60$ tons and the net $I$ of the section is 5749 in.$^4$ then the shear per inch of beam at $x_1-x_1$ is $\frac{138 \times 60}{5749} = 1.44$ tons. With $\frac{3}{4}$-in. rivets, reckoned as $\frac{13}{16}$ in., at 4.14 tons each in single shear this needs $\frac{1.44}{4.14} = 0.348$ rivet per linear inch or 0.174 rivet in each of two rows; the spacing is $\frac{1}{0.174} = 5.75$ in. The "Rivet Pitch Coefficient" in the handbooks gives the same result.

The shearing strength of the web of a compound joist is greater than that of the web of the plain joist, since the lever-arm is slightly more. The approximate value of the lever-arm for this compound being 23 in. against a value of about 21 in. for the plain joist. At a stress of 6.5 tons per square inch this would increase the shearing strength from 6.5 x 0.57 x 2 = 77.5 tons to 6.5 x 0.57 x 23 = 85.5 tons.
Fig. 30 shows a secondary effect that occurs when a plate girder sits on a foundation. The calculated loading is 31.25 ton per foot of girder. It is standard practice to set the flange angles proud of the edge of the web plate but if this is done then the rivets not only have to transmit the horizontal shear from the flange plates to the web, as in Fig. 28, but here they must also transmit the vertical local shear 31.25 tons per foot which may dangerously overload them. If they are fully stressed vertically and horizontally at the same time then they are 41 per cent. overstressed in a direction at 45 deg. In this case the edge of the web plate was set proud of the flange angles and machined down to them. The flange plates then bear directly on the edge of the web. The cross bending on the flange plates is

15.62(3.5 - 0.25) = 50.8 ton-in.

Assuming the flange plates act as two separate plates, the modulus Z per foot of girder is \(2 \times 12 \times \frac{12}{6} = 4\) in.\(^3\), and the stress is \(\frac{50.8}{4} = 12.7\) tons per square inch.

Assuming the flange plates act together,

\[
Z = \frac{12 \times 2^2}{6} = 8\;\text{in.}^3; \quad \text{stress} = \frac{50.8}{8} = 6.35\;\text{tons per square inch.}
\]

The girder is, of course, provided with vertical web stiffeners at frequent intervals and is encased in concrete. Some indeterminate fraction of the load comes down through this casing and the web stiffeners and not all comes down through the web plate as we have assumed. Fillet welds were added as shown in Fig. 30 but their strength was not included in the calculations. It might be thought from the analogy of the "spread line" in a reinforced concrete slab footing, that a "spread" of 45 deg. could be assumed in the case of the shear on a grillage joist as shown in Fig. 31 and that only sections outside the lines a–a need
be checked for beam shear. This is not so and all sections between the lines a–a and b–b must be checked. If the joist in Fig. 31 is a 24-in. ¥ 7 1/2-in. ¥ 95-lb. section, the web area of which is about 22 in. ¥ 0.57 in. = 12.55 sq. in. at a shearing stress of 6.5 tons per square inch the total shear on section b–b should not exceed 6.5 ¥ 12.55 or 81.5 tons.

Web Bearing.—Grillage beams usually have to carry heavy concentrated loads as in Fig. 32. For a beam without stiffeners the highest intensity of compressive stress occurs at the root of the web on section x–x in the figure. The load may be assumed to spread at 30 deg. through a depth equal to the gross flange thickness plus the flange plates, if any. If the joists in Fig. 32 are 24 in. ¥ 7 1/2 in. ¥ 95 lb. and they carry a stanchion load W sitting on an 18 in. ¥ 18 in. base plate the effective spread is (18 + 6.56) in. As the webs are 0.57 in. thick the total bearing area is 2 ¥ 24.56 ¥ 0.57 = 28 sq. in. At a bearing stress of 12 tons per square inch the maximum permissible value of W is 28 ¥ 12 or 336 tons. If the joists had 3/4-in. flange plates, this would increase the spread by 2.6 in. thus increasing the bearing strength from 336 to 372 tons. This stress of 12 tons per

\[ \text{Fig. 32.} \]

square inch can only be worked to if the buckling strength of the web is sufficient or if the webs are definitely prevented from buckling under load. Provided there are two or more parallel beams with closely-spaced through-bolted separators all solidly encased in strong concrete, the latter condition may be achieved.

Web Buckling.—Clause 28(a) in B.S. No. 449 (1959) puts forward a formula for checking the web-buckling strength. Unfortunately, and most dangerously, neither explanation nor diagram is given of the assumptions on which this formula is based. It now appears that it was web failure of this type that caused the recent collapse of a major steel bridge structure. Apart from excessive direct bearing stress at the root of the web (Fig. 32) there are two conditions of collapse to be considered; practical cases usually lying between the two theoretical extremes. Suppose we have short bare joists without separators as shown in Fig. 33 carrying a vertical load W centrally applied. They may collapse as shown, the equivalent round-ended length, which is referred to as “effective” length in B.S. No. 449, being \( d \). Since the radius of gyration \( r \) is \( \frac{t}{2\sqrt{3}} \) the value of \( \frac{l}{r} \) in Clause 30(a) of the standard is \( \frac{2\sqrt{3}d}{l} \). If we have a double compound joist solidly encased in good concrete so that no sidesway is possible, as in Fig. 34, the two webs may buckle into the empty space, the equivalent round-ended length being 0.5d and
the ratio \( \frac{d}{r} \) being \( \frac{\sqrt{3}d}{t} \). This apparently is the only case envisaged in B.S. No. 449.

Taking a 24-in. \( \times \) 7\( \frac{1}{2} \)-in. \( \times \) 95-lb. joist with \( d = 22 \) in. and \( t = 0.57 \) in., \( \frac{d}{r} \) in Fig. 33 is \( \frac{2\sqrt{3} \times 22}{0.57} = 134 \), for which the safe stress is 2.8 tons per square inch.

In Fig. 34, \( \frac{d}{r} = 67 \); the safe stress is 5.98 tons per square inch.

The total length of web which may be considered as supporting a concentrated load for the purpose of assessing the web-buckling strength is shown in Figs. 35 and 36 which are based on B.S. No. 449. If a joist or compound joist carries a concentrated load \( W \) at its extreme end as in Fig. 35, the length of web effectively resisting the tendency to buckle may be arrived at by assuming a spread line at 45 deg. down to the centre of the joist. The effective spread in Fig. 35 is \( l \) plus \( \frac{1}{2} \) (joist depth) plus (flange-plate thickness). The author is prepared to accept this spread if the joist extends a distance equal to at least the depth of the joist beyond the edge of the concentrated load. If a concentrated load \( W \) is applied near the centre of a beam as in Fig. 36, the effective spread is \( l + \) (joist depth) + 2 (flange-plate thickness). The author is prepared to accept this spread if the joist extends at least the depth of the joist beyond both edges of the concentrated load. Assuming that the joist in Fig. 36 is 24 in. \( \times \) 7\( \frac{1}{2} \) in. \( \times \) 95 lb. with \( \frac{1}{4} \)-in. flange plates and that \( l \) is 18 in., the effective spread is 18 in. + 24 in. + 1\( \frac{1}{2} \) in. = 43\( \frac{1}{2} \) in.
If an uncased joist is under the conditions in Fig. 33, the safe web buckling load would be $43.5 \times 2.81 = 122$ tons. Under the conditions in Fig. 34 it would be $43.5 \times 5.98 = 260$ tons. The latter load cannot be worked to, as the intensity of web bearing immediately under this load would be too high. Referring to Fig. 32, the effective spread for web bearing is 27.16 in. and the maximum value for $W$ at a stress of 12 tons per square inch is $27.16 \times 0.57 \times 12 = 186$ tons.

Clause 28(a) in B.S. No. 449 seeks to limit the length $l$ in Fig. 35 to a length equal to half the joist depth and $l$ in Fig. 36 to a length equal to the joist depth. This condition is understandable for the end bearings of superstructure beams, particularly if of small depth:span ratio where the beam deflects and tends to bear on one edge only of its support. It is an unnecessary restriction in average foundation work where the whole length of the stiff bearing is effective. It is also unnecessary in the case in Fig. 37 where the beam is held down to its bearing by the upper stanchion.

Compare the strength of the two joists in Fig. 37 with the formula in B.S. No. 449. If these are 24 in. $\times 7\frac{1}{4}$ in. $\times 95$ lb. the spread for web buckling, since they extend well back beyond the centre-line of their support, is 30 in. The actual safe stress in buckling, as we have already shown, is 2.81 tons per square inch while the stress allowed by B.S. No. 449 is 5.98 tons per square inch. The actual total safe web-buckling load of two joists is only $2 \times 30 \times 0.57 \times 2.81 = 96$ tons. The value given by B.S. No. 449 is $2 \times 30 \times 0.57 \times 5.98 = 204$ tons. Buckling of the type shown in Fig. 33 could be prevented by adequate stiffeners and separators and if these were provided a safe load of 204 tons could be carried.

**Timber**

The days when the engineer could insist on pitch pine for all permanent work are now past. In this country timber, for permanent work, is now generally used only below ground-water level and then only as timber piles. The timber used is variously described as Douglas fir, Oregon or Columbian pine and is excellent provided that every baulk is carefully examined before purchase and all pieces where the grain runs out or curls suddenly are rejected.

Safe stresses for new timber are as follow

- Tension and compression along the grain: 1000 lb. per square inch.
- Compression across the grain: 400 lb. per square inch.
- Shear: 200 lb. per square inch.
- Elastic modulus: 1,200,000 lb. per square inch.

The engineer must make his own reductions to allow for deterioration according to site conditions and the expected life of the structure. A graph showing the safe strength of timber struts is given in Fig. 38.
Aluminium Alloy

Although now widely used for roofs there seems no immediate prospect of its adoption for foundation work. The use of aluminium alloy box-piles is a future possibility.

Prestressed Concrete

It is difficult to draw an exact line between the foundation proper and the substructure. For example in the case of a piled foundation, should we regard the piles alone as constituting the foundation and the pile-cap as the substructure. In a modern piled jetty the lower part of the pile is clearly a foundation but the upper part may be regarded as substructure or even superstructure. Taking the narrow interpretation the author has met no case where prestressed concrete could have been used with advantage for foundation work proper. This does not mean that cases cannot and will not arise as the technique improves. A loss of prestress of 10 tons per square inch, due to plastic yield of the concrete, with an elastic modulus of 13,000 tons per square inch means a change in length of nearly 1 in. in 100 ft.
CHAPTER VIII

MASS CONCRETE PIERS AND WALLS

Of all modern forms of foundation, mass concrete walls and piers are the simplest. They need no expert technical supervision and are not appreciably affected by weather or breakdown of plant. Usually the load is applied over the whole of the upper surface but formerly it was sometimes applied as in Fig. 39 to only the centre of its area and was assumed to spread through the mass of the pier. If the angle of spread is not less than 60 deg. to the horizontal this is safe for 1:8 concrete generally and for 1:10 with suitable aggregate under reasonable supervision.

Mass Concrete Foundations for Offices and Warehouse

Consider the example of a two-story office block with a single-story warehouse attached. The walls of both buildings consist of hollow precast blocks filled with reinforced concrete to form vertical columns with precast slabs spanning between these columns infilled with no-fines concrete. The office has hollow tile floor and roof. The warehouse has steel trusses with asbestos-cement roofing. (See Figs. 41 and 42.)

SITE INVESTIGATION.—The local geology is shown in Fig. 40. Recorded boreholes within a mile of the site show London Clay to be 200 to 250 ft. thick.
This is covered locally by a bed of Taplow gravel. The site originally sloped down towards the small river, falling about 14 ft. in a length of about 250 ft., but had been brought level by made ground of good quality. Buildings on neighbouring sites showed no signs of distress. Experience with driven piles on a site half a mile away was not very encouraging and indicated that virgin blue clay, capable of supporting 4 tons per square foot, was not to be expected within 30 ft. of the original surface and that the clay down to this level was oxidised and softened.

Loading.—The piers supporting the warehouse stanchions also carry the external wall, the combined load being 21 tons per pier at floor level. The load from the outside walls of the office is 2½ tons per foot run at ground level. The row of piers running down the centre of the office building carry 40 tons each at ground level. (See Fig. 41.)
GENERAL DESIGN.—It was thought that the clay immediately below the gravel might not be capable of supporting more than 1 ton per square foot. It was also to be expected that the bed of gravel, being so narrow, would tend to peter out. The possibility of large spread footings on the surface for the warehouse with a high-level raft for the office was considered. Preliminary levels indicated that no deep drainage trenches would be required near enough to this raft to disturb its bearing capacity but, as so often happens, subsequent events proved this idea to be completely wrong. The old rule-of-thumb for settlement of railway embankments allowed 1 in. per foot of height. As the depth of fill ran from about 5 ft. to about 14 ft. the settlement, assuming that half had already taken place, might vary from 2/4 in. to 7 in. Even if this estimate was pessimistic some relative settlement was inevitable and with an office building nearly 100 ft. long this idea was quite unacceptable. Driven piles, some of which might pull up in the ballast and some drive right through, were not attractive particularly as a minimum of two piles would be required under the warehouse stanchions to carry only 27 tons between them. Bored piles, sunk through the ballast into the clay, seemed a better proposition but were also not very attractive. This left mass concrete piers as the best choice. It was decided to take these down to ballast, probe and check the thickness of the ballast stratum at each pier hole and continue down into clay capable of supporting 2 tons per square foot if and where the ballast proved less than 3 ft. thick. The arrangement of piers and walls is shown in Figs. 41 and 42. Dimensions of 4 ft. 6 in. by 4 ft. 6 in. proved to be a bit tight for the deeper pier holes and these were actually made 5 ft. 6 in. square. The mass concrete foundation walls under the office reached a maximum depth of 14 ft. and were made 3 ft. thick. This is the minimum dimension recommended by the Foundation Code. Conditions below ground turned out as expected.

The fact that the original site is covered with made-up ground raises the question of what is meant by net loading intensity on such a site. A pier 5 ft. 6 in. square and 14 ft. deep carrying 27 tons at ground level has a self-weight of 27 tons and a gross weight of \(27 + 27 = 48\) tons. The gross loading intensity is \(\frac{48}{5\frac{1}{2}}\) or 1.58 tons per square foot. If we subtract the weight of the ground removed this reduces to 0.83 tons per square foot. If the made-up ground continues to consolidate and settle and clings to the sides of the pier as it does so, could it impose downward friction on the piers and thus increase the loading? The effect is probably very small in dry filling. In new wet filling likely to stick to the piers, a downward friction of 2 cwt. per square foot on a pier 5 ft. 6 in. square by 14 ft. deep would increase the load by 31 tons or 1.02 tons per square foot. The gross loading intensity on the central row of piers under the office is \(40 + 25\) = 65 tons on 28 sq. ft. or 2.32 tons per square foot. Subtracting the weight of ground removed leaves 1.57 tons per square foot net loading intensity. The mass concrete was gauged 1 cwt. of cement to 10 cu. ft. of ballast specified as “To be approved pit ballast or Thames ballast. To be clean and well graded. Maximum size of stone generally to be 2 in. to 3 in. with occasional stones up to 6 in.” The contractor mixed in a proportion of 3-in. to 6-in. stones and produced some excellent concrete.

The factory roads round the warehouse were concrete on rolled hardcore.
The made ground proved very elastic going down several inches under the roller but rising again as soon as the roller passed. There is a loading bank along both sides of the warehouse with a dwarf retaining wall of reinforced concrete sitting on top of the made ground. Neither the roads nor this wall show any obvious signs of settlement but no exact check levels have been taken.

Some years later a second warehouse was built parallel to and about 45 ft. away from the first. The roof is supported on fixed-end steel lattice shed frames standing on concrete blocks as shown in Fig. 43. The finished structure only

![Diagram](image)

Fig. 43.

requires anchor bolts on the inside where two bolts of \( \frac{1}{4} \)-in. diameter are provided. The frames were fabricated in three pieces with site joints at the points of contraflexure. The two side pieces were erected first. (See Plate II facing page 58.) and the centre lowered on to them. The two bolts on the outside must be strong enough to pull down the side pieces until the frames are finally jointed, tied and adjusted. Since the upper part of the foundation block in Fig. 43 stands in made-up ground no reliance was placed on any counter pressure that might develop on the sides of the block, and the feet of the shed frames were tied across by steel screwed rods buried in the floor. Plate III (facing page 59) shows the completed frames.
Tank Foundation on Mass Concrete Walls

The local geology in this example is as in Fig. 44. The site is about 300 ft. above sea level in a narrow valley, in which flows a stream, and is about 200 ft. from the watercourse. The general indication is that the ground consists of alluvium overlying valley gravel below which is the middle chalk. This is confirmed by the boreholes in Fig. 45. A heavy storm in the local valleys can produce a stream in a matter of minutes and the author guessed that the sand and gravel would produce copious water. The level of the edge of the foundation slab was fixed at 297:50 which is the highest reported water level in the river. Assuming a suitable bearing 1 ft. below the top of the ballast this placed the underside of the foundation walls between 290:00 and 293:00.

Fig. 44. (Based on a Crown Copyright Geological Survey Map by permission of the Controller of H.M.S.O.)

LOADING.—The main slab carries a uniformly imposed load of 0·7 ton per square foot with a small additional load round the periphery and a concentrated load of 25 tons at the centre. It was estimated that the stratum of sand and gravel would safely carry a net loading intensity of 3 tons per square foot after allowing for the fact that it is submerged but is also well below surrounding ground level.

DESIGN.—Two alternatives were considered; to remove all the topsoil and made ground down to the ballast, fill solid with lean concrete or flushed hardcore and provide a 9-in. slab reinforced with a nominal mesh; or to provide a thicker slab carried on mass concrete foundation walls. There seemed little between them in price but the latter scheme promised easier control of the pumping and placing of mass concrete. This concrete was 1:6 below level 292·00, 1:8 between 292·00 and 294·00 and 1:10 above 294·00.

Following the recommendation of the Foundation Code these walls were made 3 ft. wide and at 3 tons per square foot would carry 9 tons per foot run. The top surface of the structural slab carries a screed which is conical in shape
PLATE I. (See page 36.)

PLATE II. (See page 57.)
with a maximum variation in thickness of 6 in. Allowing for the fact that there must be 6 in. less depth of water in the tank where the screed is thickest, this adds on only about 40 lb. per square foot maximum.

Load: 0.7 ton per square foot = 1568 lb. per square foot
Self-weight of 12-in. slab: = 150 " " " "
Screed (minimum 1-in.): = 12 " " " "
Sloping screed, say: = 30 " " " "

1760 " " " "

= 0.785 ton per square foot.

Maximum spacing of 3 ft. walls = \( \frac{9}{0.785} = 11.48 \) ft. The nearest spacing to give an integral number of bays is 11 ft. 7 in. which is within 1 per cent. The general arrangement is shown in Fig. 46.

The mixture of concrete in the structural slab is 1 cwt.:1\( \frac{3}{4} \) cu. ft.:3\( \frac{3}{4} \) cu. ft. and working stresses of 1000 and 18,000 lb. per square inch were adopted.
Bending moment on slab = \(1760 \times 1.58^3 \times 12 \times \frac{1}{12} = 236,000\) lb.-in.

Minimum \(d_1 = \sqrt[3]{\frac{236,000}{193 \times 12}} = 10.1\) in.

With \(1\frac{1}{4}\)-in. cover and 1-in. diameter bars, \(d_1 = 10.25\) in.

\[A_{st} = \frac{236,000}{18,000 \times 0.85 \times 10.25} = 1.5\text{ sq. in.}\]

The steel provided is 1-in. bars every 6 in. (1.57 sq. in.). The steel was not increased in the end spans, firstly because of the width of the bearing walls which reduces the effective length of the end spans to 10 ft. 10 in. and secondly because...
the imposed load must act equally on all spans at once. Details of the reinforcement in this slab are shown in the section in Fig. 47 (pages 66 and 67) which is part of section A–A in Fig. 46. The distributors are $\frac{3}{8}$-in. diameter at 2-ft. centres or 0.106 per cent. of the gross area. This is less than the area recommended by the Code but is enough for a slab that sits on wet ground and is permanently covered by a deep water tank. No attempt has been made to reduce the steel at mid-span to resist a bending moment having the theoretical value of $wL^2/24$. The bars in the bottom are provided with hooks and have a 3-ft. lap. This arrangement is adopted partly because the high ratio of effective depth to span (1 to 15) and partly due to an instinctive feeling for the overall strength of the whole foundation as a unit. The reinforcement is shown in Plate IV (facing page 59).
CHAPTER IX
REINFORCED CONCRETE SLAB FOOTINGS

The plain slab footing in Fig. 48 is the simplest and usually the cheapest means of spreading the normal range of column or stanchion loads—say up to 1000 tons. It is clear that the stresses are highest immediately under the column and it might be thought that a stepped footing as Fig. 49 or a slab-and-beam footing as in Fig. 50 where the construction is also concentrated under the column, might be cheaper, at any rate for the larger sizes. Generally the footing has to be kept below the lowest floor level to accommodate pipes, ducts, trenches, etc., the distance h in Figs. 48, 49 and 50 being 2 ft. to 2 ft. 6 in. The space between the top of the footing and the floor has to be filled and this filling is usually hardcore, lean mass concrete or a mixture of the two. The stepped footing, if cast in two operations, requires vertical stirrups to tie the two lifts together. It may be deeper overall than a plain slab thus requiring more excavation and much more filling. The slab-and-girder design in Fig. 50 is usually uneconomical unless the footings are combined into a raft and a suspended floor supplied, the space between the footings and this floor remaining void. The level of the underside
of the footings is fixed by several considerations. (1) It must be below the frost line—say 3 ft. in Great Britain. (2) It must be level with or nearly level with neighbouring existing foundations, nearby lift-pits (present or future), drainage trenches or anything that tends to weaken the bearing capacity of the ground (see Figs. 6, 7 and 8 on pages 9 and 10). Footings at a higher level should be below the 30-deg. line as in Fig. 87 (page 83). (3) Footings must be taken down to sound ground. There may be a choice between large high-level footings at a small net loading intensity and small deep footings at a large net loading intensity. This case is surprisingly rare in practice and when there is real doubt it may be necessary to design two schemes and price them. Other things being equal, the small deep footings are better as they offer less obstruction and are less vulnerable to future construction. (4) The underside of any reinforced concrete or steelwork should, if possible, be kept above natural ground-water level and any work below this level should be in mass concrete. If not, dewatering and the draining of water entering the excavation are necessary.

The type of foundation in Fig. 51 must always be avoided. The earth fill collects any ground water, rain water or leakage, settles, and cracks the floor above it. On a clay site this water may soften the ground below the floor and in time destroy the bearing capacity of the ground below the footing. There is also the trouble of carrying out reinforced concrete work at the bottom of a dirty hole. Since the footing in Fig. 51 is not "concreted tight against the soil up to ground level" the increased bearing capacity allowed for increased depth below ground surface (see Foundation Code paragraphs 1.631 and 1.641) cannot be claimed. The footing in Fig. 52 not only qualifies for increased bearing capacity but any friction between the vertical faces of the mass concrete and the ground increases its effective area.

**Pressures under Slab Footings**

With a centrally loaded footing some experts say that the intensity of bearing pressure on the ground varies as Fig. 53. Others say it varies as Fig. 54. Others say it depends on the type of ground and again others say it depends on the relation between the flexibility of the base and the physical properties of the soil. The first and most vital step in foundation design is to assess the ultimate bearing capacity of the ground, that is the point at which shearing-friction failure develops. We are therefore concerned with the distribution of loading at the point of failure.
and not with the distribution under working loads. According to the rheologists, eight physical properties of the ground must be measured before the distribution at failure can be assessed. As far as the author is aware this calculation has never even been attempted. Figures that have been worked out are based on assumptions that have been so drastically simplified that the results are practically useless. By placing a thin flexible steel plate on the ground and applying a heavy concentrated load at the centre, the edges of the plate can be made to lift and

conditions similar to Fig. 54 can be produced under loads much less than the ultimate. Slab bases of normal thickness are very stiff and suffer very small deflections. Considering the conditions that must develop if the loading were increased until the ultimate bearing capacity is reached deflections of a slab footing of normal dimensions must be too small appreciably to affect the result. We are therefore thrown back on the assumption, made throughout the centuries in connection with millions of footings of this type, that we can safely assume that the ground pressure, for a centrally-applied load, is uniformly distributed
over the underside of all normal slab footings. It would now require thousands of long-term measurements under hundreds of full size bases on actual sites to establish the need for any other assumption. Small scale artificial laboratory experiments on a few handfuls of remoulded clay can scarcely be regarded as evidence in such a problem.

Where ground conditions vary the intensity of pressure might depend, under ultimate bearing conditions, on the relative paths of escape of the ground particles from the underside of the foundation to the free surface. In Figs. 6, 7 and 8 the ground could escape more easily from under the right-hand edge of the foundation towards the open basement or cutting than from under the left-hand edge and the intensity of loading under the right-hand side might be less than that under the left-hand side under ultimate loading conditions. This is another argument for the foundations in Figs. 7 and 8 (page 10) being below the 30-deg. line. A similar argument applies to the case where the surface of the site varies in strength and stiffness from one patch to another and it might be possible to have a shallow foundation standing half on a harder patch and half on a softer one. Variations of this kind almost always decrease or are obviated if the foundations are all taken deeper and this should be done if possible. If the base is eccentrically loaded the usual straight-line variation may be assumed. For foundations below ground-water level or those to which surface water might find access, the line of pressure should always lie within the middle third of the base to ensure positive downward pressure under all parts of the base under all loading conditions. An approximate attempt is made in Chapter XI to assess the distribution of pressure under a flexible raft but this can be ignored as far as this chapter is concerned.

**Bending Moments in Slab Footings**

With very small span-depth ratios we are a long way from our conception of a theoretical member whose depth is small compared with the span and we can more usefully spend our time working out sound methods of construction and simple steel arrangements than attempting to calculate "exact" stresses. For the base shown in plan and section in Fig. 55 the total moment on section x-x is $W \left( \frac{b}{2} - \frac{c}{4} \right)$, where $W$ is the total load on the column and $b$ and $c$ are the widths of the base and column respectively. For a base carrying a reinforced monolithic concrete column the Code suggests that the base may be designed for the moment on the section at the face of the column—an odd act of cheeseparing. For bases of normal thickness it may be assumed that the total moment is distributed uniformly across the section x-x. For a rectangular base ABCD shown in plan in Fig. 56 the Code suggests that reinforcement parallel to the short sides AB and CD should be more closely spaced under the column and spaced out towards the ends AB and CD. Yet if the base be enlarged to A'B'C'D' so that it becomes square the steel in both directions may be spaced uniformly—an odd piece of reasoning. Only in the case of abnormally thin footings would the author consider varying the spacing and then in both directions. With a very thin slab and a small column (using a very high strength concrete and heavy reinforcement) the maximum moment directly under the column could scarcely exceed 1.5 times the average moment. In an extreme case it might be assumed that the middle
Fig. 57.—Section AA.

(See also Fig. 46 on page 60)

the average moment on section x–x and the maximum intensity of moment on this section directly under the column is shown in Fig. 58 for two values of Poisson's ratio.

Shearing Stresses in Slabs

The standard English method of checking the shearing strength was similar to the problem of punching a hole in a steel plate and a "punching shear" stress of twice the safe "beam shearing" stress was allowed on the area of a clean punched hole as in Fig. 59. It was also stipulated that all sections outside the 45-deg. line in Fig. 60 should be designed for beam shearing. Apparently the conception of punching shear was never accepted by other European nations. The present version of the Code abandons it but retains the "45-degree line" idea. The actual failure of the base is more nearly represented by Fig. 61 which shows failure by tension on planes at 45 deg. The shearing strength of the slab in Fig. 62,
assuming a safe beam-shear stress of \( q \), a safe shearing tension of \( q \), or a safe punching-shear stress of \( 2q \) is:

\[
2q \times 4c \times d = 8cdq \quad \text{for punching}
\]

or

\[
q \times 4(c + 1.7d) \times 0.72d = (2.88c + 4.9d)dq \quad \text{for beam shear}
\]

or

\[
q \times 4(c + d)(\sqrt{2d}) \times 1/\sqrt{2} = (4c + 4d)dq \quad \text{for 45 deg. tension.}
\]

Comparison between these three methods of calculating the limiting shearing strength depends on the ratio between column size and slab thickness and the effect of varying this ratio is shown in Fig. 63. The maximum safe load on a reinforced concrete column is about \( 9c^2q \) and this value is also shown in Fig. 63. The curve showing the calculated strength based on tensile failure at 45 deg. as in Fig. 61 lies close to that marked “beam shear” in Fig. 63 and is shown by a dotted line. The issue is complicated by the amount of load passing directly from the column to the ground and not crossing any assumed shearing plane and by the fact that some criteria are based on effective and some on overall depth of the slab.

If we take a reinforced concrete column 18 in. x 18 in. carrying 144 tons and compare square bases for this at net loading intensities of 1 ton per square foot and 4 tons per square foot as in Figs. 64 and 65, the effective and overall depths required for “punching shear” at 200 lb. per square inch, beam shearing at 100 lb. per square inch and moment at 193d\( d^2 \) are shown in the following.
<table>
<thead>
<tr>
<th>Punching shear</th>
<th>Beam shearing</th>
<th>Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_1$</td>
<td>$d$</td>
<td></td>
</tr>
<tr>
<td>16.38 in.</td>
<td>22.0 in.</td>
<td>13.20 in.</td>
</tr>
<tr>
<td>13.50 in.</td>
<td></td>
<td>12.50 in.</td>
</tr>
</tbody>
</table>

Few designers would make the slab thickness less than the column diameter as this looks wrong and the author would not accept any base that does not comply with all three requirements.

**Rectangular Bases.**—*Figs. 66 and 67 show two bases.* In the square base, with a centrally loaded column, we may assume that one quarter of the column load is carried in shear on each of the four column faces. In the extreme case where the width of the base equals the width of the column it can only be carried in shear, half on each of the two faces. For intermediate ratios of length to width...
draw lines at 45 deg. and assume that the shearing force on each face equals
the net loading intensity on the hatched areas within these 45-degree lines.

**Eccentrically-loaded Bases.**—The distribution of shear to each face of
the column may be assumed to follow Figs. 66 and 67 remembering that the net
loading intensity is heavier than average under some areas.

![Fig. 66.](image)

![Fig. 67.](image)

**Cantilevered Bases**

In built-up areas or when extending an existing building it frequently happens
that there is not sufficient room to accommodate simple square or rectangular
bases under each of the outside columns. Sometimes it is possible to set the out-
side stanchions in from the building line. Sometimes it is possible to cantilever
in at ground-floor level but considerations of headroom may forbid this. Details
of a successive series of cantilevers are shown in Fig. 99 where the load on a
stanchion is carried to a foundation xx ft. 6 in. away. If the trouble affects a
large number of columns it may be worth while to adopt a different type of
foundation. For example, if the loading on the outside columns does not average
more than 10 tons per foot run of building line, then a single row of bored piles
carrying 35 tons each and spaced at 3 ft. 6 in. centres will take this (see Chapter
XIII). A capping beam about 2 ft. 3 in. wide will cover a 17-in. diameter pile
and project only 1 ft. 14 in. beyond the column centre-line. The arrangement is
shown in plan in Fig. 69 (page 72). If the ground at basement level will safely
take 2 tons per square foot a slab-and-beam foundation (see Chapter X) 5 ft. wide
would carry 10 tons per foot but would project 2 ft. 6 in. in front of the column
centre-line.

If it is not possible to pull the columns in at a higher level, not possible to
project sufficiently far to adopt a slab-and-beam foundation, and not practicable
to use piling, a cantilevered footing is indicated. The far end of the cantilever
should be tailed down by one of the interior columns (the author does not like the
use of piles in tension). The centre of gravity of the two columns is easily found.
If we have two columns as shown in Fig. 68 (page 72) carrying 110 tons (external)
and 180 tons (internal) and spaced at 25 ft. centres, the centre of gravity is
REINFORCED CONCRETE SLAB FOOTINGS

15 ft. 6 in. from the external column. If we can project only 1 ft. 6 in. beyond the centre-line of the external column we can go only 15 ft. 6 in. plus 1 ft. 6 in. or 17 ft. to the left of the centre of gravity. If we want a base of uniform width (this is simpler than the tapering type shown in Fig. 70, page 72) and want it to be uniformly loaded it must be symmetrical about the centre of gravity and must therefore be 34 ft. long. To carry 290 tons at 2 tons per square foot a width of 4·26 ft. is required (say 4 ft. 4 in.). The design reduces to a rectangular beam carrying 8·54 tons per foot. The greatest moment occurs where the shear force is zero 12·9 ft. from the left-hand end (11·4 ft. from the external column) and is 110 tons \( \times \left( \frac{11·4 - 6·45}{2}\right) = 543 \text{ tons-ft.} = 14,600,000 \text{ lb.-in.} \). The cantilever beyond the internal column carries 63·3 tons causing a moment of 63·3 \( \times 3·75 = 238 \text{ tons-ft.} \). The greatest shear force is on the centre-line of the external column and is \( \frac{110 - 8·54}{8·54} = 97·2 \text{ tons} = 218,000 \text{ lb.} \).

Working to a value of \( \frac{M}{bd_{1}} = 193 \) and a shearing stress of 100 lb. per square inch on the lever-arm area

\[
\text{Min. } d_{1} \text{ for bending } = \sqrt{\frac{14,600,000}{193 \times 52}} = 38·1 \text{ in.}
\]

\[
\text{Min. lever-arm for shearing } = \frac{218,000}{100 \times 52} = 42 \text{ in.}
\]

The beam could be 4 ft. deep overall for such a small span-depth ratio with \( d_{1} = 45 \text{ in.} \).

\[
\text{Top steel } = \frac{14,600,000}{18,000 \times 0·86 \times 45} = 21 \text{ sq. in.}
\]

This could be either 12 bars 1\( \frac{1}{4} \)-in. diameter in one layer or 27 bars 1-in. diameter in two layers (or 21 bars 1\( \frac{1}{4} \)-in. diameter). The bottom steel required under the internal column is

\[
\frac{238 \times 2,240 \times 12}{18,000 \times 0·90 \times 45} = 8·8 \text{ sq. in.}
\]

The shearing-bond stress at the external column is

\[
\frac{218,000}{\text{lever-arm} \times \text{bar perimeter}}
\]

If this value is taken at 180 lb. per square inch

\[
\text{bar perimeter } = \frac{218,000}{40·2 \text{ in.} \times 180} = 30 \text{ in.}
\]

To supply this we should need 7 bars 1\( \frac{1}{4} \) in. diameter or 10 bars 1 in. diameter. A bar of 1\( \frac{1}{4} \) in. is rather clumsy for the detail under the external column. The author would not use it and would prefer bars 1-in. diameter to bars 1\( \frac{1}{4} \)-in. diameter. If the external column carries more load than the internal one the arrangement in Fig. 68 will not work and a footing slayed in plan or widened at the outside end is needed as shown in plan in Fig. 70 or 71 (page 72).
Bending Strength of Slabs

Bending strengths of rectangular sections in accordance with Code No. 114 (1957) are given in Appendix A. Since the author does not recommend a stress of 20,000 lb. per square inch for foundation work and since compression steel is very rarely, if ever, needed we are really reduced to the "critical" points on the various curves, that is the points where both the concrete and the tension steel are stressed to their maximum permissible values. These "critical" values are given in the following table.

<table>
<thead>
<tr>
<th>Tensile Stress 18,000 lb. per square inch</th>
<th>Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{cb}$ lb. per square inch</td>
<td>$m = 15$</td>
</tr>
<tr>
<td>$\frac{M}{bd_1^2}$</td>
<td>Percentage of tensile steel</td>
</tr>
<tr>
<td>750</td>
<td>126</td>
</tr>
<tr>
<td>1000</td>
<td>193</td>
</tr>
<tr>
<td>1250</td>
<td>265</td>
</tr>
<tr>
<td>1500</td>
<td>340</td>
</tr>
</tbody>
</table>

Bond Stresses in Slab Bases

In the usual square base in Fig. 72 where $L$ is the half-width in feet the maximum unit moment is about $\frac{pL^2}{2} \times 12$ lb.-in. per foot width and the grip-length on a bar that goes right across the base, allowing for the hook, is about $12L$ inches. If the lever-arm is $l_a$ (inches) and the combined perimeters of the rods in unit width is $o$ (inches) then the average bond stress is

$$\frac{\text{moment}}{\text{lever-arm} \times \text{perimeter} \times \text{length}} = \frac{I}{l_a o}$$

This is nearly equal to

$$\frac{pL^2}{2} \times \frac{12}{l_a} \times \frac{I}{o \times 12L} = \frac{pL}{2l_a o}$$

The maximum unit shear is nearly $pL$ and the maximum shearing-bond stress nearly $\frac{pL}{l_a o}$ or about twice the average bond stress. It was usually assumed that the safe shearing-bond was about twice the safe average bond stress but the ratio between these two permissible stresses as given by the Code is only about 1.5 so that in this case the shearing-bond stress of approximately $\frac{pL}{l_a o}$ is usually the criterion. The total area of steel in square
inches per unit width at a stress of 18,000 lb. per square inch is \( \frac{M}{18,000l_a} \). If there are \( n \) rods per foot width each of diameter \( d \) (inches) then approximately

\[
6\phi \frac{L^2}{18,000l_a} = n\pi \frac{d^2}{4}
\]

The perimeter of these rods is \( n\pi d \) (inches). If the local shearing-bond stress is 180 lb. per square inch then

\[
\frac{\phi L}{l_{a0}} = \frac{\phi L}{l_a(n\pi d)} = \text{approx. 180}
\]

or \( \frac{\phi L}{180l_a} = n\pi d \) very nearly.

Dividing one equation by the other,

\[
\frac{6L}{100} = \frac{d}{4} \text{ very nearly}
\]

or \( L \) in feet = approximately \( 4.17d \)

or \( L \) in inches = 50 diameters approximately.

All the calculations are approximate as they do not allow for the width of the column.

The author is not satisfied that the standard pull-out bond test is a safe criterion of bond stresses in beams, particularly for dry high-strength concretes, and would not recommend any average bond stress over 120 lb. per square inch or local shearing-bond stress over 200 lb. per square inch.

Since the normal slab footing cantilevers about the column centre-line, the moment falls off rapidly as we get nearer to the edges. Halfway between the column centre-line and the outside edge the moment is only one quarter of the maximum and we could, as far as bending strength is concerned, stop off three quarters of the bars. In a small base the bond stresses usually make this impossible but in larger bases, say more than 10 ft. \( \times \) 10 ft., alternate bars may be stopped short. This should not be done on small contracts with little or no technical supervision where it is much safer to run all bars right through. In very large bases the moment diagram should be drawn as a guide to reasonable stopping places for the bars (see Figs. 96 and 97 on pages 94 and 95).

**Economical Depth of Slab Bases**

Although we must always calculate the minimum thickness required for bending and shearing it is sometimes neither convenient nor economical to work to these minimum sizes. First take the case on a dry site shown in Fig. 73 where the minimum structural thickness already takes the underside of the base down to sound ground. Deliberately increasing the depth of the base increases the excavation, increases the concrete and increases the edge shuttering (this may be actual shuttering, timbering or an extra couple of inches of concrete if the ground is firm enough to cast against). The increase in net loading intensity on the soil
caused by increasing the depth of the base is only the difference between the weight of the additional concrete and the weight of the soil it replaces. This is generally trifling. Next take the case in Fig. 74 where the safe foundation level lies well below the underside of the reinforced base and where the mass concrete and reinforced concrete are both cast against the ground. Provided we keep above the ground-water level, adopting a deeper reinforced base does not increase excavation or shuttering nor does it increase the total amount of concrete, although there is a greater amount of the more expensive mixture.

Alternative designs for a base to carry 140 tons at 2 tons per square foot are shown in Figs. 75 and 76. The base in Fig. 76 has been deliberately increased in depth. The quantities are as follows.

One base as in Fig. 75.—
4-7 cu. yd.; 6-9 cwt.;
and 6-62 sq. yd.

One base as in Fig. 76.—
6-7 cu. yd.; 4-25 cwt.;
and 9-45 sq. yd.
If we adopt the deeper base as Fig. 76 in the conditions shown in Fig. 73 we have

\[
\begin{array}{l}
\text{Extra 2 cu. yd. of excavation at 15s.} \\
\text{Extra 2 cu. yd. of concrete at 105s.} \\
\text{Extra 2.83 sq. yd. of shuttering at 20s.} \\
\hline
\text{Saving 2.65 cwts. of steel at 70s.} \\
\text{Increase in price} \\
\end{array}
\]

\[
\begin{array}{ccc}
10 & 10 & 0 \\
10 & 10 & 0 \\
2 & 16 & 6 \\
9 & 5 & 0 \\
5 & 11 & 6 \\
\end{array}
\]

If we adopt the deeper base in the conditions shown in Fig. 74 we have

\[
\begin{array}{l}
\text{Extra cost of richer concrete} \\
\text{2 cu. yd. at 105s. less 70s.} \\
\text{Saving 2.65 cwts. at 70s.} \\
\text{Saving in price} \\
\hline
\end{array}
\]

\[
\begin{array}{ccc}
3 & 10 & 0 \\
9 & 5 & 0 \\
5 & 15 & 0 \\
\end{array}
\]

The comparison must depend on the relative unit costs of concrete, steel, shuttering and excavation but generally no great advantage follows from squeezing the depth of a slab footing to the minimum.

**Stepped Footings**

The overall depth of a large footing is usually dictated by shearing, the minimum depth required for bending being only about three quarters of that required for shearing. It is therefore possible to save some of the concrete by making the footing thinner round the edge as shown in Fig. 77. Under certain unusual conditions this may be an advantage but in most cases this saving is fictitious since this space has to be filled back with mass concrete or flushed hardcore. The idea may be extended by making the upstand narrower and deeper as in Fig. 78. This is usually more expensive as it increases the excavation and considerably increases the mass concrete or hardcore backfilling. It may simplify the concreting if the concreting plant is small but should have a cage of
stirrups if the two lifts are to function as one. (The footing in Fig. 80 on page 77 has twenty-four links \( \frac{1}{2} \)-in. diameter marked A.)

As a matter of interest many foundations were, and are, constructed with sloping tops as in Fig. 79. These were usually calculated on the assumption that the load "spread" through the concrete at 60 deg. and the critical sections for both shearing and bending were on the planes x-x. This would not, and quite rightly, be allowed under modern regulations. Probably very few of such footings were actually built as drawn, as the cost of the mitred shuttering required to make the top slopes outweighed any saving in concrete. A design for a stepped footing to carry 800 tons at 2 tons per square foot is shown in Fig. 80.

**Steel-grillage Foundations**

Fifty years ago the loading from steel stanchions in steel framed buildings was spread by grillages consisting of two or three tiers of steel joists. Now a heavy slab base, bearing directly on a reinforced concrete footing is more usual.

Occasionally a single tier of joists is used to spread the load one way as in Fig. 81, the spread in the other direction being done by a reinforced concrete slab. In the figure this slab cantilevers 5 ft. 3 in. Since the whole arrangement is eventually filled solid with concrete when the steelwork has been plumbed and levelled, it has been claimed that this filling could be regarded as structural concrete, giving an overall depth of about 4 ft. 9 in. in the example in Fig. 81. This is not so, as it would be usual practice to attach the joists to the stanchion base and wedge and pack off the 2 ft. 3 in. slab. This means an appreciable and often a considerable interval between casting the 2 ft. 3 in. slab and concreting the remainder.

**Hollow-cone and Hollow-dome Foundations**

The top surface of a slab footing may be sloped or stepped to make the slab thinner at the edge (see Figs. 77 to 80) thus saving concrete. This idea is taken a stage further in Fig. 82 where the underside is also sloped to save more concrete.
The earth is trimmed to a cone and covered with a layer of mass concrete. The footing consists of a hollow cone reinforced with circular hoops. In this example a circular column carries a load of 236 tons. The footing is a 45-deg. cone 10 ft. diameter giving a pressure of 3 tons per square foot. The necessary hoop steel below any section can be found by the method in Fig. 83 which shows the lower
part of the cone held in equilibrium by three forces—a direct thrust down the cone, an upward load from the ground and a horizontal tying-in force provided by the steel hoops. The area of the annulus of ground under this portion of the footing is

$$\frac{\pi}{4} (10^2 - 6^2) = 50.4 \text{ sq. ft.}$$

The upward load, at 3 tons per square foot, is

$$50.4 \times 3 = 151.2 \text{ tons} = 338,000 \text{ lb.}$$

This is balanced by the vertical component of the thrust. As the angle in this example is 45 deg., the total thrust is $338,000 \times \sqrt{2}$ lb. The circumference on which this thrust acts is $\pi \times 6.5$ ft. or 20.4 ft.

$$\frac{338,000 \times \sqrt{2}}{20.4} = 23,500 \text{ lb. per foot run of circumference.}$$

If the cone is 8-in. thick this is

$$\frac{23,500}{8 \times 12} = 245 \text{ lb. per square inch.}$$

The horizontal component of this thrust is

$$\frac{23,500}{\sqrt{2}} = 16,600 \text{ lb. per foot run.}$$

The hoop tension caused by this on 6-ft. 6-in. diameter is

$$16,600 \times (\frac{1}{4} \times 6.5) = 54,000 \text{ lb.}$$

At a stress of 18,000 lb. per square inch, this requires 3 sq. in. of steel or, say three hoops of a $1\frac{1}{2}$-in. diameter bar below this section. These must be bent accurately to a true circle and should be welded to form a ring which is a difficult and expensive process. By moving the section up and down, the pressure on the concrete at all levels and the diameter and spacing of the hooped bars can be calculated.

Designs of this type often appeal to the beginner but he should avoid them until he has had sufficient site experience to appreciate how very expensive work of this type can be. There may always be the odd combination of site conditions that could make such a design economical and the principles underlying this analysis are worth remembering.

The same remarks apply generally to foundations shaped as part of a spherical dome. Some particular shapes of construction are covered by patents.

**Example of Plain Slab Foundations**

The slab foundations described in the following pages serve as the foundation to an eight-story steel framed building faced with Portland stone standing in the West End of London.

**Site Exploration.**—An existing building (see Fig. 84) standing on the north part of the site was to remain. Existing buildings on the south part were to be pulled down and a new building erected similar to and connected with the existing northern building to form one homogeneous building occupying the whole of the
island site. Except for an open square to the east of the site the whole district is paved or built over. The site slopes gently from north-east to south-west. Details of the ground are shown in Fig. 85. The main stratum is clearly London clay as shown by the records of two boreholes some hundred yards east of the site, originally overlain by gravel. Drawings of the foundations for the existing building on the north side of the site showed foundations a few feet below basement level sitting on clay and designed to carry 3 tons per square foot. The River Thames is less than half a mile away and there were originally a number of small streams flowing into it. The original beds of most of these are known and charted but there was always the chance of striking the bed of some small forgotten tributary.

The only apparent problem was to join a new building on newly-loaded clay foundations to an existing building which has stood there some twenty years. This is an overstatement for the old buildings on the south part of the site weighed possibly half as much per square foot as the new building and only the remaining half of the loading is really new. It was not possible to make an open vertical joint between new and old thus making two separate buildings as the main east face forms the whole side of the square and this joint would have split one of the main stone pilasters. The first thing was to put down a trial pit to check that the clay under the south part was the same as the clay under the north. This produced two nasty surprises. The clay was obviously not good enough for 3 tons

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Fig. 84. (Based on a Crown Copyright Geological Survey Map by permission of the Controller of H.M.S.O.)
per square foot and the trial pit filled slowly with water. It was thought that this water might have been trapped behind the old foundation walls or be due to a leaking main but was found not to be the case. The softness of the clay made the question of relative settlement a major issue and a borehole 40 ft. deep, that is 50 ft. below the pavement, was sunk in the middle of the site of the new building. This was also not encouraging. The stiffness of the clay increased very slowly with depth and even at 50 ft. below the pavement the clay had still a brownish tinge. Laboratory tests indicated that the clay at 35 ft. was capable of supporting about 3 tons per square foot and at 50 ft. possibly 4.5 tons. The author would not have put more than 30 tons on a 17-in. diameter bored pile sunk 40 ft. below the pavement. The possibility of supporting the new building on bored piles was seriously considered but was rejected on general grounds. Any widespread change in moisture content of the upper part of the clay stratum might affect the relatively shallow existing spread footings and not the proposed new piles. The author assessed the safe bearing capacity of the clay on the south part of the site, by visual and manual inspection and by probing with bars, to be 2 tons per square foot for shallow spread footings. Taking into account the question of relative settlement he decided on a net loading intensity of 1.75 tons per square foot for the internal stanchions and 1.5 tons per square foot for the external stanchions as these latter carry a higher percentage of dead load and part of the basement walls. Since the self-weight of the foundation slabs was about equal to the weight of clay excavated to construct them, the net loading intensity was taken to mean the load on the stanchion base-plate divided by the area of the footing. The sizes of the foundations for the first stage of the contract are shown in Fig. 86 opposite. The basement in the south-east corner is deeper than elsewhere and a section Z-Z through this is also shown in Fig. 86. Owing to the small spaces between neighbouring foundations it was not possible to step up quickly, and most of the foundations under the shallower part of the basement had to be taken down, or very nearly down, to the same depth as those under the deeper part, involving much extra excavation and mass concrete. Had the clay been stiffer and capable of supporting 3 tons per square foot there would have been clear spaces between adjoining bases and they could have been stepped up as shown diagrammatically in Fig. 87. The southern part of the new building had to be built and occupied before the north-east section, which was actually lying on the centre-line of the whole building, could be commenced. It was then found that the clay became much stiffer north of a line situated some 20 ft. south...
of the centre-line in Fig. 84. Trial holes had already confirmed that the existing foundations were actually as shown on the drawings. Subsequent construction, which is described later, showed that the ground water encountered earlier was flowing through a layer of ballast above the clay and was finding its way through the old basement walls on to the site. This water apparently came from rain falling on the open ground in the square to the east of the site augmented by the watering of flower beds. When the new basement walls were completed it ceased.

**Design of Slab Footing.**—As will be seen from Fig. 86 most of the footing slabs on the south part of the new building carry only one stanchion and have no unusual features. An example is the base for stanchion No. 50. The base is 13 ft. x 20 ft. 3 in. x 3 ft. 6 in. thick and carries 464 tons.

Net loading intensity = \( \frac{464}{13 \times 20.25} \) = 1.76 tons per square foot.

The stanchion base-plate is 3 ft. 9 in. x 3 ft. 9 in.

Total moment in long direction = \( \frac{464 \times (20 - 25)}{2} \) tons-ft. (see Fig. 55).

\[ = 955 \text{ tons-ft.} \]

\[ = \frac{955 \times 2240 \times 12}{13} \text{ lb.-in. per foot width} \]

\[ = 1,970,000 \text{ lb.-in. per foot.} \]

With stresses of 1000 and 18,000 lb. per square inch and \( m = 15 \) the safe

\[ \frac{M}{bd^2} = 193. \]

\[ \text{min. } d_1 = \sqrt[3]{\frac{1,970,000}{193 \times 12}} = 29.1 \text{ in.} \]

The actual \( d_1 \) is about 39 in.

Punching shearing strength at 150 lb. per square inch

\[ = 4 \times 45 \text{ in.} \times 42 \text{ in.} \times 150 \text{ lb.} = 506 \text{ tons.} \]

Lever-arm = say 0.849 x 39 = 33 in.

Total "spread" for beam shearing = 45 in. + 2(39 in.) = 123 in.

= 10 ft. 3 in. (see Fig. 60).

Fig. 88 shows this spread on plan.

Actual beam shearing on plane a-b is caused by the shaded area in Fig. 88 which is 63 sq. ft. at 3940 lb. per square foot.

Beam shearing on a-b = 63 x 3940 = 249,000 lb.

Beam shearing strength on a-b at 100 lb. per square inch

\[ = 123 \text{ in.} \times 33 \text{ in.} \times 100 = 405,000 \text{ lb.} \]

Total tension steel required in long direction = \( \frac{1,970,000 \times 13 \text{ ft.}}{18,000 \times 33 \text{ in.}} \) = 43.1 sq. in.

Actually provided are 49 bars \( \frac{14}{16} \)-in. diameter = 60 sq. in.

Total moment in short direction = \( \frac{464 \times (13 - 3.75)}{2} \) tons-ft.

\[ = 537 \text{ tons-ft.} = 14,400,000 \text{ lb.-in.} \]

Total steel required in short direction = \( \frac{14,400,000}{18,000 \times 32 \text{ in.}} \) = 25 sq. in.
Actually provided are 32 bars \(1\frac{4}{8}\)-in. diameter = 39.2 sq. in.

Total perimeter of bars provided in short direction = \(32 \times 3.14 \times 1.25 = 125\) in.

Total shearing-bond strength at 180 lb. per square inch

\[= 125 \times 32 \text{ in.} \times 180 \text{ lb.} = 321 \text{ tons.}\]

The maximum shearing must be less than half the total load, that is less than 232 tons. The reinforcement is shown in Fig. 89. The steel in the long direction is spaced at 3-in. centres leaving only \(1\frac{3}{8}\) in. between the bars. The design would have been better with 34 bars \(1\frac{4}{8}\)-in. diameter spaced at \(4\frac{1}{4}\)-in. centres but these were not available.

![Diagram](image_url)

**Fig. 88.**

The steel provided is more than is theoretically required. The base was actually designed to carry 2 tons per square foot to cover any possible unforeseen increase in loading between designing the foundations and completing the roof. In these calculations also, it has been assumed that the load from the stanchion is uniformly spread over a base-plate 3 ft. 9 in. × 3 ft. 9 in. As the base-plate is grouted before the stanchion is loaded there is a tendency for the load to concentrate under the centre as the load comes on the stanchion and the plate deflects. If the pressure on the ground should reach 2 tons per square foot and neglecting the width of the stanchion base-plate the areas of steel in the long and short directions would be 62 sq. in. and 40. This example is perhaps typical of what may happen if there is a certain amount of uncertainty at the beginning of a contract.

It is simplest if each stanchion can stand on its own independent footing but exigencies of stanchion spacing and available ground area sometimes make this impossible. On this contract, four stanchions Nos. 48, 49, 54 and 72 stand on one combined base (Fig. 86). The area of this base is 21 ft. × 36 ft. = 756 sq. ft.
The stanchion loads and the area of base occupied by each are as follows.

Stanchion 48 load 505 tons area occupied 292 sq. ft.

<table>
<thead>
<tr>
<th></th>
<th>313</th>
<th>182</th>
</tr>
</thead>
<tbody>
<tr>
<td>49</td>
<td></td>
<td></td>
</tr>
<tr>
<td>54</td>
<td>402</td>
<td>233</td>
</tr>
<tr>
<td>72</td>
<td>84</td>
<td>49</td>
</tr>
<tr>
<td>1304 tons</td>
<td>756 sq. ft.</td>
<td></td>
</tr>
</tbody>
</table>
REINFORCED CONCRETE SLAB FOOTINGS

The base is shown again in plan in Fig. 90; the average pressure is \( \frac{1304}{750} = 1.73 \) tons per square foot. The largest stanchion load is 505 tons on a base-plate 3 ft. 9 in. square. The base is 4 ft. thick.

The punching shear strength at 150 lb. per square inch is

\[
4 \times 45 \times 48 \times 150 \text{ lb.} = 578 \text{ tons};
\]

\[ d_1 = \text{say 44 in.}; \quad \text{lever-arm} = 0.849 \times 44 = 37.5 \text{ in.} \]

Total "spread" for beam shearing at 45 deg. under stanchion No. 48 is

\[ 45 \text{ in.} + 2(44) \text{ in.} = 133 \text{ in.} \]

Total beam shearing strength at 100 lb. per square inch is

\[ 4 \times 133 \times 37.5 \times 100 \text{ lb.} = 890 \text{ tons.} \]

Although the average calculated pressure under the final stanchion loads is 1.73 tons per square foot the base was designed to carry 2 tons per square foot. This allows for unequal live loading on the stanchions, small inequalities in the ground under such a large base, and possible additions of load between the time of designing the foundations and completing the roof.
SECTION A - A

SECTION B - B

106 $\frac{1}{2}$" L = 25' - 9"

ex. 4" crs. in bottom middle layer.

18 $\frac{1}{2}$" $\frac{1}{2}$" L = 22' - 9"

ex. 2½" crs. in top lower layer.

57 $\frac{1}{2}$" $\frac{1}{2}$" L = 37' - 9"

ex. 4½" crs. in bottom lower layer.

12 $\frac{1}{2}$" $\frac{1}{2}$" L = 26' - 9"

ex. 4½" crs. in top upper layer.

BASE 48 49 54 72

Fig. 91.
REINFORCED CONCRETE SLAB FOOTINGS

Unit cantilever moment on line b-b in Fig. 90

\[ = 4480 \times 12 \times 87^2 \times 12 \times \frac{1}{2} = 4,450,000 \text{ lb.-in.} \]

\[ \text{min. } d_1 \text{ required } = \sqrt{\frac{4,450,000}{193 \times 12}} = 43.8 \text{ in.} \]

\[ A_{st} = \frac{4,450,000}{18,000 \times 37.5} = 6.6 \text{ sq. in.} \]

Total steel in 21 ft. width = 21 \times 6.6 = 138 \text{ sq. in.}

= 113 \text{ rods } 1 \frac{1}{4} \text{-in. diameter}

The reinforcement is shown in Fig. 91. There are 57 bars marked (11) and 57 marked (12).

The load on stanchions Nos. 48 and 49 is 62.5 per cent. of the total and on line d-d in Fig. 90 which is 62.5 per cent. of 36 ft. from the lower edge; the shearing is zero. Actual upward moment on d-d = 818 tons \times 11.25 ft. whilst the actual downward moment = 818 tons \times 9 ft. 9 in., that is, there is no tension on the top of the slab in the centre of the foundation. Consider the skew section a-a in Fig. 90.

Triangular area = \( \frac{1}{2} \times 19 \text{ ft.} \times 1 \frac{1}{2} \text{ in.} \times 36 \text{ ft.} = 344 \text{ sq. ft.} \); distance of centre of gravity of this area from line a-a is 5 ft. 7\frac{1}{2} in. Total upward moment on section a-a is

\[ 4480 \times 344 \times 5.625 \times 12 \text{ lb.-in.} = 104,000,000 \text{ lb.-in.} \]

The length of section a-a is 40 ft. 9 in. and the average moment on section is

\[ \frac{104,000,000}{40.75} = 2,550,000 \text{ lb.-in. per foot width.} \]

The effective depth is, say 45 in.;

\[ A_{st} = \frac{2,550,000}{18,000 \times 0.85 \times 45} = 3.71 \text{ sq. in. per foot.} \]

This result neglects downward load on the small stanchion No. 72 which stands on this triangle. The steel provided is 1\frac{1}{4}-in. bars at 4-in. centres, which is equal to 3.68 sq. in. (marked 13 in Fig. 91).

The shearing in this triangular area is not simple but we may follow the idea in Fig. 67 and draw the beam shearing lines for this foundation in Fig. 92. In sharing the area of this base between the four stanchions we can take it that an area of 21 ft. \times 22 ft. 6 in. supports stanchion Nos. 48 and 49 (see Fig. 90) and the remaining 21 ft. \times 13 ft. 6 in. supports stanchion Nos. 54 and 72 (see Fig. 92). The "spread" for beam shearing at 45 deg. is 44 in. The stanchion base-plates plus this "spread" are shown in Fig. 92. From this it appears that the most shear crossing any one plane outside the 45-deg. line is the upward load on an area 8 ft. 7 in. wide by about 11 ft. 6 in. average length lying to the left of stanchion No. 54. The length of shearing plane through which this loading is transmitted back to stanchion No. 54 is 9 ft. 9 in. At 2 tons per square foot total upward load = \( 13.5 \times 8.6 - \frac{3.75 \times 3.75}{2} \) \times 2 = 218 tons; net upward load after deducting the load on stanchion No. 72 is 218 - 84 = 134 tons. Strength of shearing face 9 ft. 9 in. long at 100 lb. per square inch = 117 \times 37.5 \times 100 \text{ lb.} = 196 \text{ tons.}
One of the difficulties of such a combined footing is the volume of concrete, namely, \((21 \text{ ft.} \times 36 \text{ ft.} \times 4 \text{ ft.})^{\frac{1}{3}} = 112 \text{ cu. yd.}\).

An obvious place for a construction joint is at section d–d in Fig. 90 with 70 cu. yd. below and 42 cu. yd. above this level. The lower part may be split again about 8 ft. from the right-hand edge thus dividing the 70 cu. yd. into 27 and 43 cu. yd. A single 10/7-mixer turning out twenty batches per hour would take 8.3 hours to complete 43 cu. yd. Two 10/7-mixers would take 4.15 hours, the concrete rising very nearly one foot per hour and this would be satisfactory.

The foundations in the north-east part of the new part of the building (see Fig. 84, page 81) were more complicated. Foundation space was restricted by the foundations of the existing portion. This meant picking up the most northerly of the new stanchions on steelwork and transferring the load on to what clear areas were available. A plan of the arrangement is shown in Fig. 93. Working space was very limited and the first operation was to drive steel sheet-piling outside the external foundations (those to stanchion Nos. 38, 40, 41 and 44) to support the roadway. This sheet-piling was supported by raking struts carried down to existing stanchion foundations (Fig. 94). As soon as the external foundations were completed they supported the steel sheet-piling which was strong enough to cantilever up to pavement level thus leaving the basement free of all strutting which was a great advantage to everyone concerned.
One of the new foundations is tight against the old and considerably lower. It would have been possible to underpin the old foundations but this would have been slow and would have meant finding workmen experienced in this kind of work. The scheme adopted is shown in Fig. 95. Steel trench sheeting was driven along the sides of the new foundation before any excavation was begun. This
was easy, the ground being medium stiff clay. A single trench was then dug across the new foundation down to a level of \(-19\) ft. 6 in. Short lengths of steel channel were spot welded to the trench sheeting and a strut made up of three steel scaffold tubes welded together in a clover-leaf pattern was put in, wedged tight with steel wedges and spot welded in place. This process was repeated until the whole length was strutted and the excavation was then taken down to \(-21\) ft. 3 in. These struts were located at the mid-depth of the new foundation and both struts and sheeting were left in. The calculated safe load on these struts of \(12\)-ft. length and made of three nominal \(1\frac{1}{4}\)-in. diameter 8-gauge (0·160 in.) tube (area \(2·625\) sq. in., \(k=0·904\) in.) is \(6\frac{1}{2}\) tons each. A plan and sections through this long foundation are shown in Fig. 96 (page 94). It is 42 ft. long, 11 ft. 9 in. wide and 4 ft. deep. (See also Fig. 93.) The loading is brought on to the top surface partly by a pair of plate girders with a reaction of 734 tons and partly by four joists with a total reaction of 129 tons. (See also Figs. 97 and 99.)

\[
\text{Net loading intensity} = \frac{734 + 129}{42 \times 11\frac{7}{8}} = 1.75 \text{ tons per square foot}
\]

Centre of gravity of load is \(\frac{129}{863} \times 8 \cdot 2 \text{ ft.} = 1 \cdot 22 \text{ ft. from the larger load.}

The concrete for this base is nominal \(1:1\frac{1}{4}:3\) mixture grade IIa. With stresses of \(1250\) and \(18,000\) lb. per square inch and \(m=15\).

\[R = \frac{M}{bd_1^2} = 265; \quad \text{lever-arm} = 0.83d_1; \quad 1.75 \text{ ton} = 3920 \text{ lb. per square foot.}
\]

Clear cantilever = 18 ft. 6 in. (no "spread" taken) (Fig. 97).

Unit \(M = 3920 \times 18.5^2 \times 12 \times 1\frac{1}{4} = 8,050,000\) lb.-in. per foot width.

Theoretically there is a greater moment where the shear changes sign in the diagram in Fig. 97 but with strip loading of this kind and a thick slab the usual practice of designing for the moment on the clear cantilever is fair.

Overall depth = 48 in.; \(d_1 = 44\) in.

\[\frac{M}{bd_1^2} = \frac{8,050,000}{12 \times 44^2} = 347.
\]

With \(\rho_{cb} = 1250\), \(\frac{R}{\rho_{cb}} = \frac{347}{1250} = 0.278\), \(\frac{\rho_{st}}{\rho_{cb}} = \frac{18,000}{1250} = 14.4\).

From the charts of bending strength (Chart 6 on page 220 in Appendix A) the reinforcement required is

\[A_{st} = 2.22 \text{ per cent. and } A_{st} = \text{just over } \frac{1}{2} A_{st}.
\]

\[A_{st} = \frac{2.22}{100} \times 12 \times 44 = 11.7 \text{ sq. in. per foot width.}
\]

Total \(A_{st} = 11.7 \times 11\frac{7}{8} = 137.5\) sq. in. in \(11\) ft. 9 in.

\[A_{st} = \text{just over } \frac{1}{4} A_{st}, \text{ say 3 sq. in. per foot width.}
\]

Total \(A_{sc} = 3 \times 11\frac{7}{8} = 35.3\) sq. in.

As an approximate check on \(A_{st}\), \(\frac{8,050,000}{18,000 \times 0.83 \times 44} = 12.25\) sq. in.
The actual tensile steel provided is as follows

bottom layer 38 bars 1\frac{1}{4}-in. diameter = 67.2 sq. in.
middle layer 38 bars 1\frac{1}{4}-in. diameter = \frac{67.2}{134.4} sq. in.

The top layer of 38 bars of 1\frac{1}{4}-in. diameter [shear bars marked (\Xi\Xi\Xi) in Fig. 96] are nearer the neutral axis and not fully effective if we take d_1 as 44 in. The actual compression steel is 38 bars of 1\frac{1}{4}-in. diameter = 46.6 sq. in.

As a check we can assume that all the tensile steel is effective and d_1 is only 42.75 in.

201.6 sq. in. of tensile steel = \frac{3.34}{2} per cent.

46.6 sq. in. of compression steel = 0.772 per cent.

\[ A_{st} = \frac{0.772}{3.34} A_{st} = 0.231 A_{st}. \]
REINFORCED CONCRETE SLAB FOOTINGS

From Chart 6 (page 220) this would give a safe bending strength of

\[ 405bd_1^2 = 405 \times 12 \times 42 \times 75^2 \]

\[ = 8,850,000 \text{ lb.-in. per foot width.} \]

Maximum shear force = 3920 \times 18.5 \text{ ft.} = 72,500 \text{ lb. per foot width.}

Shearing strength of concrete alone at 115 lb. per square inch

\[ = 115 \times 12 \times 0.83 \times 44 = 50,300 \text{ lb. per foot width} \]

\[ = 3920 \times 12 \times 85 \text{ ft.} \]

The concrete alone is strong enough to take the whole shear for a distance of 12.85 ft. from the extreme ends of the foundation. Total shear = 72,500 \times 11.75 ft. = 852,000 lb. = 380 tons (see Fig. 97). Shear reinforcement is 38 bars 14-in. diameter bent up at 45 deg.

Total tensile strength of bent-up bars = 38 \times 1.767 \times 18,000 = 1,210,000 \text{ lb.} \]

\[ = 540 \text{ tons.} \]

Vertical component = \frac{540}{\sqrt{2}} = 382 \text{ tons.} \]

Vertical component = \frac{540}{\sqrt{2}} = 382 \text{ tons.} \]

Total perimeter of steel, ignoring shear bars = 76 \times 3.14 \times 1.5 = 358 \text{ in.;}

lever-arm = 0.83 \times 44 = 36.6 \text{ in.} \]

Shear-bond strength at 200 lb. per square inch = 358 \times 36.6 \times 200 = 2,620,000 lb. = 1170 tons.

The reinforcement is shown in Fig. 96. The loading comes on to the top surface in strips stretching right across the width of the footing and there is therefore, theoretically, no cross bending. The cross steel supplied top and bottom is nominal. There is always the chance that the girders, through inequalities of packing or relative deflection may bear unevenly until the building settles down. The cross steel also serves to tie the main steel together. This beam has no vertical links. These would have assisted in tying the steel together and provided anchorage for the compression steel. They would also have interfered with and slowed down concreting. No anchorage is normally provided for compression bars in floor slabs and this is merely a large slab. Assuming all three layers of main tension steel are effective and ignoring the compression steel we should have a section with 3.34 per cent. of tension steel and a safe bending strength of 305bd_1^2 (see Chart 6, page 220).

\[ 305bd_1^2 = 305 \times 12 \times 42 \times 75^2 \]

\[ = 6,600,000 \text{ lb.-in. per foot width.} \]

If we accept the increased compressive stress allowed by the Code after 12 months maturing, and the author does not accept it, this is increased to

\[ 1.24 \times 6,600,000 = 8,200,000 \text{ lb.-in.} \]

which is less than our calculated moment. Notice the great length of the rods compared with the requirements of the moment diagram in Fig. 97. The end hooks have been spaced by eye with the idea of picking up oblique thrust from the load as shown in Fig. 98. Unfortunately modern theory of shearing strength is still not in line with the instinctive knowledge of experienced engineers. One criticism of a long narrow footing is its relative flexibility which might mean slightly more yielding under load than that of the smaller foundations. In this
Fig. 100.
case there was no reasonable alternative. (The use of bored piles had been considered.)

This foundation contains 73 cu. yd. of concrete and there was no safe place for a construction joint. Happily this basement has a broad permanent ramp and access for ready-mixed concrete was easy. Using loads of 3\(\frac{1}{2}\) cu. yd. a total of twenty loads was required. Plate V (facing page 122) shows the foundation before concreting commenced and Fig. 99 shows the construction bringing the load down to it. Fig. 94 (page 92) includes a section through the foundation adjoining the road. Construction of this was easy as it spans only one way and could be concreted in sections.

**Chimney Foundation**

*Fig. 100* (opposite) shows the foundation slab for a reinforced concrete chimney. The total height of the stack from underside of foundations is 184 ft. The total weight of the stack is 525 tons including lining and the weight of the foundation is 137 tons. The weight of the filling over the foundation (the foundation is 20 ft. below ground level) is 320 tons giving a total gross weight of 525 + 137 + 320 = 982 tons.

The stack was designed for a wind load of 25 lb. per square foot of elevation plus a horizontal earthquake load of 0.1 g. The ground is hard conglomerate. With no wind, the loading intensity is 1.71 tons per square foot. With maximum wind the edge pressure is 2.91 tons per square foot. With maximum earthquake shock the edge pressure is 3.92 tons per square foot. The tensile stress in the steel under dead load plus wind load is 18,000 lb. per square inch. Under dead load plus earthquake load it is 27,000.
CHAPTER X

BEAM-AND-SLAB FOOTINGS

The first variation from the plain slab footing where each column has its own separate base, is to connect the foundations for a row of columns to make a beam. In the simplest case this may be a rectangular beam as in Fig. 101 but is more usually an inverted T-section as in Fig. 102. They are often employed for columns standing in external walls where the top of the beam also serves as a footing for the wall panels between the columns. On many sites the amount by which the footings may project outside the building line is strictly limited and a long narrow footing of this type is then suitable.

Fig. 101.

Fig. 102.

footing for the wall panels between the columns. On many sites the amount by which the footings may project outside the building line is strictly limited and a long narrow footing of this type is then suitable.

Loading on Beam

The effective loading on the beam depends on (a) The stiffness of the superstructure and its relation to the stiffness of the beam; (b) The arrangement of live load on the superstructure; and (c) Variation in ground conditions from one end of the beam to the other.

If we take the case of a grain silo with rectangular bins supported on inverted T-section foundation beams, the superstructure is stiff enough to ensure that any settlement of the building is on a straight line variation as in Fig. 103. If all the bins were full the applied loading on the ground would be uniform and equal to \( \frac{W_D + W_L}{L} \) where \( W_D \) is the total dead load and \( W_L \) the total live load on one beam.

If only half the bins were full this would increase to a maximum of about \( \frac{W_D + 1.25W_L}{L} \) assuming that \( W_D \) is not less than 0.25\( W_L \). If \( W_D \) were equal to \( W_L \) then this would be only 12\% per cent. above fully loaded conditions and it might be argued that such an unusual distribution of live load could well be allowed to encroach on the safety factor of the beam provided the ground will safely carry the increased intensity of net loading. A more serious consideration arises if the ground varies. Small local shallow pockets of soft ground are,
of course, removed and replaced by mass concrete. General variation may be
due to variation in the ground immediately below the foundations or, more
rarely, due to variation in deep underlying strata or to variations in adjoining
ground levels which increase or decrease the amount of lateral support. A case
actually occurred under a silo standing on a clay stratum overlying chalk where a
large circular patch of clay was much softer than all the rest of the site; almost
certainly due to an old swallow-hole in the chalk below. In extreme cases the
maximum upward loading per ft. run on the foundation beams over the harder
areas of the site, might be twice the average. The net loading intensity could be
kept down by increasing the width of the slab over the harder areas. If the

![Diagram of beam and slab footings]

**Fig. 103.**

location of the softer patches is not known before the design is completed this
may mean designing all the slabs and beams for twice the average loading on a
very patchy site. With a stiff superstructure, such as shown in *Fig. 103*, the
foundation beams span a distance $l$ between incompressible supports and moments
of $\pm \frac{wl^2}{12}$ and $\pm \frac{wl^2}{10}$ for interior and end spans may be assumed.

The other extreme is shown in *Fig. 104* where the superstructure consists of
simply-supported beams and has no overall stiffness. The live loading on alternate
columns for this arrangement is $0.75w_LL$ and $0.25w_LL$ but if we substitute a
continuous beam of varying depth these could, at their limit, be $1.0w_LL$ and zero.
If we wish to keep a uniform pressure per ft. run on the ground throughout then
we have unbalanced live loading of $0.5w_LL$ per column on alternate columns or
BEAM-AND-SLAB FOOTINGS

uneven settlement it is usual to assume that the building may be supported entirely on the middle half of the site or entirely on the two outside quarters. If all this unequal loading comes on the foundation beams the moment for the applied load in Fig. 106 would be 1,000,000,000 lb.-ft. and would require a beam 5 ft. wide by 16 ft. deep. To support a building of this size over such a site would require a deep double basement with heavy cross walls in both directions. On the other hand if the walls of a silo are 7 in. thick and 100 ft. high and spaced at 12 ft. 6 in. centres in both directions they could be made to carry a silo building 100 ft. (8 bins) wide if all the load is taken on the centre 50 ft. on the two cut-off strips of 25 ft. If severe settlement conditions are to be met the superstructure must be planned to have regular rows of columns in both directions or the shearing forces soon get out of hand.

A beam-and-slab footing in the limit merges into a raft though beam-and-slab footings are usually found under lighter structures or on firmer sites. Uneven distribution of dead loading can be met easily by varying the width of the slab. (See Fig. 106, page 106, where the width of slab varies from 3 ft. 6 in. to 9 ft.) Settlement due to structural loading can be reduced by a general increase in width of all slabs. Neither of these arrangements can be used in designing a raft which already covers the whole site completely. Although the effect of uneven distribution of live loading on a beam-and-slab footing is usually much less severe than on a raft (generally to the point where conditions of partial loading may be ignored) there is some general resemblance and what is said in Chapter XI on raft foundations, applies to some extent here.

Some interesting examples of reservoirs in mining areas are given in a book "Reinforced Concrete Reservoirs and Tanks".

Example of Beam-and-Slab Footings

The foundations in this example support a brick building, the southern end of which has two stories, the northern end one story and the centre a basement with three stories above.

Site Investigation.—The site is low-lying and about 600 yards from the River Thames and almost certainly was originally marsh land. Local geological conditions are shown in Fig. 107. This indicates flood plain gravel overlying London Clay. The site was already occupied by small buildings, and a nearby building with a semi-basement and three upper stories which showed no signs of uneven settlement. The general ground level of the site is about 12-00. A trial pit on the south part of the site showed soft and variable soil with alternating patches of clay and sand down to level 6-00 which is about the level of the top of the old marshes that fringed the Thames. Probing with a bar found nothing
more solid than clay five feet below the surface and it was decided to found the southern end of the building at level 6-00 with a net loading intensity of \( \frac{1}{4} \) ton per square foot. It is difficult to distinguish between old made-up ground and alluvium when the made-up ground contains no human products, for example bricks, and has been dug from an adjoining site. The underside of the basement foundations under the central portion is at level 2-08. This ground was not appreciably harder, but as about 10 ft. of overburden had been removed a gross loading intensity of \( \frac{1}{4} \) ton per square foot was allowed under the basement. The ground under the northern single-story end was better and a level of 7-00 was fixed for the underside of the foundations for this end. No appreciable ground water was encountered. When the whole of the foundation area for the southern end of the building was exposed a few shallow soft pockets below level 6-00 were removed and filled back with mass concrete.

**Loading.**—When a building has no structural skeleton it is necessary to draw floor plans showing which way the floors and roofs span and the loading they impose on the walls. It is then necessary to draw elevations of all walls showing the loading bearing on them from all floors and roofs and showing all openings through the walls. By dividing each wall into sections it is then possible to calculate the total load carried on each pilaster or section of wall at each floor level and consequently at foundation level. The foundations for the southern end of the building are shown in Fig. 108 (opposite). North to south the building has a module of 12 ft. 6 in. In the southern portion are three reinforced concrete portal frames spanning 43 ft. (Section A—A in Fig. 108) bringing loads of 30 tons on to each point of support. Otherwise all loads are carried to the foundations by brick walls and piers. A gas main was encountered running across the site almost under the centre line of the north portal frame and the foundation had to bridge over it (see Section F—F in Fig. 108).

**Design.**—The top level of all foundation beams is fixed at 10-00. those under the southern section being of inverted T-sections and those under the northern
end of plain rectangular sections. The most heavily loaded strip of footing under the inverted T-section beams carries $4\frac{1}{4}$ tons per foot and is therefore 9 ft. wide. The section through this beam is shown in Section E–E in Fig. 108 and again in detail in Fig. 109.

Effective cantilever of slab say 4 ft. 3 in.

$$M = 1120 \times 4.25^2 \times 12 \times \frac{1}{2} = 121,500 \text{ lb.-in.}$$

Shearing force $= 1120 \times 4.25 = 5170 \text{ lb.}$

With stresses of 1000 and 18,000 lb. per square inch and $n = 15$,

$$\text{minimum } d_1 = \sqrt[12]{\frac{121,500}{12 \times 193}} = 7.24 \text{ in.}$$

With $1\frac{1}{4}$-in. diameter bars and $1\frac{1}{4}$-in. cover, a 9-in. slab has an effective depth of 7.25 in.

Shearing stress $= \frac{5170}{12 \times 0.85 \times 7.25} = 70 \text{ lb. per square inch.}$

$$A_{et} = \frac{121,500}{18,000 \times 0.85 \times 7.25} = 1.09 \text{ sq. in.; } 1\text{-in. diameter bars at 8-in. centres } = 1.18 \text{ sq. in.}$$

Shear-bond stress $= \frac{5170}{(0.85 \times 7.25) \times 3.14 \times 1.5} = 177 \text{ lb. per square inch.}$

The thickness of 9 in. for the slab was made standard for all inverted T-sections and a standard spacing of 8 in. was kept for the bars in the cantilever, the diameter being progressively reduced for the narrower slabs.

Beam span 12 ft. 6 in. (Fig. 109).

Load $= 4\frac{1}{4}$ tons $= 10,000 \text{ lb. per foot.}$

$$M = 10,000 \times 12.5^2 \times 12 \times \frac{3}{4} = 1,565,000 \text{ lb.-in.}$$

$$A_{et} = \frac{1,565,000}{18,000 \times 4 \text{ in.}} = 2.12 \text{ sq. in., say three bars of } 1\text{-in. diameter.}$$

Shearing force $= 10,000 \times 6.25 = 62,500 \text{ lb.}$

Shearing stress $= \frac{62,500}{13.5 \times 4} = 113 \text{ lb. per square inch.}$

The effective depth of the beam is 45.5 in.; therefore

$$\frac{\text{effective depth}}{\text{span}} = \frac{45.5}{150} = 0.302.$$

Safe shearing stress $= 1.5 \times 100 = 150 \text{ lb. per square inch.}$

The top of the foundation beams was fixed at level 10.00 to give clearance for services entering the building below ground. (Ground floor level is 13.75.) Shallower beams would have saved concrete and shuttering but would have increased the brickwork; they would have been much less effective in spanning over the softer patches of ground. All the beams had vertical $\frac{1}{3}$-in. links spaced throughout at 8-in. centres (see Fig. 109). This was the same spacing as the spacing of the main bars in all the slabs. Standardisation of spacing, wherever reasonably possible, not only simplifies and expedites the work but reduces the chance of mistakes occurring. The basement under the central part of the building
SECTIONAL PLAN OF BASEMENT

Fig. 110.
is shown in Fig. 110 and a detail of part of Section A–A in Fig. 110 is shown in Fig. 111 (opposite). Here the loading is heavier and the reinforced concrete basement walls are utilised as beams. On the staircase side of the basement there is a total of 648 tons on an area of 43 ft. 2½ in. by 22 ft. 4½ in. or 0.67 tons per square foot. This area is covered by a foundation slab 14 in. thick with a maximum span of 12 ft. 8½ in.

Since 0.67 tons = 1500 lb., the negative \( M = 1500 \times 12.7^2 \times \frac{12}{12} = 242,000 \) lb.-in.

Minimum \( d_1 = \sqrt{\frac{242,000}{12 \times 193}} = 10.22 \) in.

With 1½-in. cover and ½-in. diameter bars, \( d_1 = 12.375 \) in.

\[
A_{st} = \frac{242,000}{18,000 \times 0.85 \times 12.375} = 1.28 \text{ sq. in.}
\]

![Diagram of foundation showing 4φ 5/8" bars, 3/8" links, and 10'-6" centres](image)

This is provided by two rows of ½-in. bars marked (81) and (82) in Fig. 111 at the standard spacing of 8-in. centres, which are overlapped under the most heavily loaded walls to give ¾-in. bars at 4-in. centres (1.32 sq. in.).

The positive moment of \( \frac{wL^2}{24} \) is covered by ¾-in. bars at 8-in. centres marked (83) in Fig. 111.

Shearing force = \( 1500 \times 6.35 = 9530 \) lb.

Shearing stress = \( \frac{9530}{12 \times 0.85 \times 12.375} = 75.5 \) lb. per square inch.

The foundation beams carrying the northern (single-story) portion of the building are all of the simple rectangular type shown in Fig. 112.

Example of Anchor Foundation

The purpose of the foundation shown in Fig. 113 (page 110) is to serve as a base for the test erection of masts. Its purpose is more to resist uplift than to support downward loads.

Site Investigation.—The site which is on the top of a hill some 1100 yards from a tidal estuary and about 170 ft. above sea level, is almost level and is covered
SECTION A–A

SECTION B–B

PLAN

DETAILS OF BOLTS

FIG. 113.
with grass. Two-story buildings with brick walls near the site appear sound. A check on visible manhole covers indicated that the site was clear of drains, mains and cables although this was subsequently found to be erroneous. The local geology is shown in Fig. xiv (page 112). The ground consists of plateau gravel overlying the Hamstead beds, Bembridge marls, Bembridge limestone and Osborne marls in descending order. The Hamstead beds and Bembridge marls consist of layers of clay and marl and have the local reputation of softening badly when wet. Happily this site lies high and is clearly well drained. The plateau gravel is said to vary in depth up to about a maximum of 20 ft. A small gravel pit in an adjoining field confirmed this. A trial pit put down on the site showed about two feet of topsoil then gravel. When the excavation was complete it showed gravel capable of supporting 3 to 4 tons per square foot safe bearing capacity with a deep narrow drainage trench crossing the middle of the site. The pipe was well below the underside of the new foundation which was easily capable of spanning over the trench.

**Loading.**—The foundation shown in Fig. xiv must resist an upward or downward load of 30 tons applied at the centre of any one of the three blocks or at any point on the beams joining the blocks. The downward load can be discounted at once. The beams are 3 ft. wide and at only 2 tons per square foot a 5 ft. length of beam would suffice to carry 30 tons. Since the beams are 5 ft. deep only nominal steel is required to spread over a 5 ft. length. The upward load of 30 tons was increased to 45 tons to give a safety factor of 1.5.

**Stability and Moments.**—The beams weigh 1 ton per foot run, the large block 36 tons and the smaller blocks 26 tons each. Five conditions of loading are shown in Fig. xiv.

*Condition (a).*—Uplift of 45 tons on large block.
Net upward moment about left-hand end for stability

\[(45 - 36) \times 26 = 234 \text{ tons-ft.}\]

Downward moment \[23 \times 11.5 = 264 \text{ tons-ft.}\]  Bending moment in beam say 1 ton per foot on a span of 26 ft.

\[+ \frac{26 \times 26}{8} \text{ tons-ft.} = +2,270,000 \text{ lb.-in.}\]

*Condition (b).*—Uplift of 45 tons at mid-span on centre beam.
Upward bending moment

\[-\frac{45 \times 26}{4} \text{ tons-ft.}\]

Downward bending moment

\[+\frac{26 \times 26}{8} \text{ tons-ft.}\]

Net bending moment

\[(-293 + 84.5) \text{ tons-ft.} = -5,600,000 \text{ lb.-in.}\]

*Condition (c).*—Uplift of 45 tons on small block.
Net upward moment about right-hand end for stability

\[(45 - 26) \times 30 = 570 \text{ tons-ft.}\]
Fig. 114.
(Based on a Crown Copyright Geological Survey Map by permission of the Controller of H.M.S.O.)
BEAM-AND-SLAB FOOTINGS

Fig. 115.

Downward moment

\[(24 + 14) \times 15 = 570 \text{ tons-ft.}\]

The reaction of 14 tons is supplied by the central beam. The bending moment on the beam is, say, 1 ton per foot on 30-ft. span plus a central "point" load of 14 tons.

\[
\frac{30 \times 30}{8} + 14 \times \frac{30}{4} \text{ tons-ft.} = +5,850,000 \text{ lb.-in.}
\]

The uplift of 14 tons on the end of the centre beam causes a moment of about +2,100,000 lb.-in. on this beam.
**Condition (d).—** Uplift in centre of end beam.

Bending moment

\[-(45 - 14) \times \frac{30}{4} + \frac{30 \times 30}{8} \text{ tons-ft. } = -3,230,000 \text{ lb.-in.}\]

**Condition (e).—** Uplift on end beam 7 ft. 6 in. from small block.

Bending moment at 45-ton load

\[-14.7 \times 7.5 - 7.5 \times 3.75 \text{ tons-ft. } = -3,730,000 \text{ lb.-in.}\]

Positive bending moment near right-hand end is approximately

\[+\frac{7.7 \times 7.7}{2} \text{ tons-ft. } = +797,000 \text{ lb.-in.}\]

The stresses under the actual applied uplift of 30 tons are restricted to 750 and 18,000 lb. per square inch. Under the uplift of 45 tons they are allowed to reach 1125 and 27,000 lb. per square inch.

The beams are 36 in. wide and 60 in. overall.

\[d_1 = \text{say 56 in.}; \quad \text{lever-arm } = 48 \text{ in.}\]

Safe moment at 750 and 18,000 with \(m = 15\) is \(126bd_1^2\)

\[126bd_1^2 = 126 \times 36 \times 56^2 = 1,420,000 \text{ lb.-in.}\]

This is much higher than any of our calculated moments and the concrete stress in bending is nowhere in question.

Safe shearing resistance at 75 lb. per square inch.

\[75 \times 36 \times 48 \text{ lb. } = 58 \text{ tons}\]

which is much higher than any of the calculated shearing forces provided there are no weaknesses at construction joints. The reinforcement is shown in Figs. 116 and 117.

Bottom steel for condition (a) \[= \frac{2,270,000}{27,000 \times 48} = 1.75 \text{ sq. in.}\]

Steel provided is four \(\frac{1}{4}\)-in. bars marked (41).

Top steel for condition (b) \[= \frac{5,600,000}{27,000 \times 48} = 4.32 \text{ sq. in.}\]

Steel provided is four \(\frac{1}{4}\)-in. bars marked (45).

Bottom steel for condition (c) \[= \frac{5,850,000}{27,000 \times 48} = 4.52 \text{ sq. in.}\]

Steel provided is four \(\frac{1}{4}\)-in. bars marked (46).

Top steel for condition (c) \[= \frac{3,730,000}{27,000 \times 48} = 2.88 \text{ sq. in.}\]

Steel provided is four \(\frac{1}{4}\)-in. bars marked (48).
Construction joints are only allowed where shown in Sections A–A and B–B in Fig. 113, the whole foundation being cut into five sections, namely three blocks and two beams. This was made clear in the specification which calls for each section to be concreted in one continuous operation and to be completed in six hours. This is repeated as an item in the bill of quantities for the contractor to

put an extra price against which was in fact done. The largest section is 20 cu. yd. which, in six hours, needs 90 cu. ft. per hour or 20 batches of 4.5 cu. ft. The concrete mixture is 1 cwt.: 12 cu. ft.: 3½ cu. ft. and a one-bag mixture is 7-25 cu. ft. which would overfill some nominal 7/5 mixers but could easily be managed by a 10/7-mixer. The construction joints are reinforced according to the maximum shearing forces across them.
The large block weighs 36 tons. To pick this up on steel bars inclined at 45 deg. needs

\[
\frac{36 \times 2240 \times \sqrt{2}}{27,000} = 4.22 \text{ sq. in.};
\]

steel provided is three \(\frac{1}{4}\)-in. bars marked (44).

The small blocks weigh 26 tons and need 3.05 sq. in.;
steel provided is three \(\frac{1}{4}\)-in. bars marked (47).

The vertical links are mostly provided to take the tension from the long bolts down to the bottom steel and to tie the top surface of the beams to prevent any side shearing that may come on the bolts. The blocks require steel in the top to cantilever from their centre-lines when they are lifted up off the ground. The completed base is shown in Plate VI (facing page 122). The foundation just described is a testing base to take masts of different shapes and sizes. Permanent bases to take specific cantilever masts have been built as shown in Fig. 118. This base consists of three reinforced concrete beams 4 ft. by 4 ft. each about 40 ft. long. Each mast has three legs at 20 ft. centres. Each beam is equal in weight to half the maximum uplift (including a factor of stability) from the windward leg of the mast. The two other leeward beams holding down the ends of the windward beam. Details of one of these bases are shown in Fig. 119 (page 118).
CHAPTER XI
RAFT FOUNDATIONS

Many structures in the past were built on layers of tree trunks laid criss-cross to form something that looked like a raft. When buried in suitable soil below ground-water level these rafts lasted for centuries. On suitable sites in countries where timber is abundant this could still be the best solution to some foundation problems. The term "raft" is now very loosely applied to any shallow foundation covering a large area and is sometimes even used to denote a ground floor slab or pavement. As already discussed in Chapter II the first process in designing foundations is to determine the "ultimate bearing capacity" of the ground, that is, the intensity of loading at which shear-friction failure would develop. This value is divided by a factor of safety (say 2.5) to give the "safe bearing capacity". If the total weight of the structure exceeds the total safe bearing capacity of the site, then a shallow foundation is impossible and a deeper foundation, or lighter building, is necessary. If the total weight of the building is within the safe bearing capacity of the site it is then necessary to estimate what settlement would result if the ground were loaded to the safe bearing capacity and what effect this settlement would have on the superstructure. If this settlement, local or general, is more than the superstructure can safely, or reasonably resist or absorb, it is reduced to within safe limits by making the foundations larger and reducing the pressure to the "allowable bearing pressure". If the individual footings at the allowable bearing pressure touch, or very nearly touch, one another they may be combined into a continuous foundation which would nowadays be called a raft. If the individual footings at the allowable bearing pressure are so large that they overlap and deep foundations are either impossible, impracticable or far too expensive, then a raft in the true sense of the word becomes necessary; that is a foundation where it is no longer sufficient to study each column load separately but where the strength and settlement of the combined foundation must be considered. The loading on the ground under a true raft is therefore somewhere between the safe bearing capacity and that value of the allowable bearing pressure which the engineer would adopt if the site were large enough. In other words the general settlement is larger than most engineers like and all that can be done is to provide some structural means, such as a raft, to keep the relative settlements within reasonable limits. If the structure carries an appreciable live load we must not only make an estimate of the expected settlements under long-term loading but we must also carry out loading tests to determine the compressibility of the soil under short-term loading. (Settlement is discussed in Chapter V.)

The next thing to study is the superstructure. Every effort must be made to adopt a regular and reasonably close spacing of columns and, if possible, the same spacing in both directions to give square panels. Loading must be reduced to a reasonable minimum and distributed as evenly as possible over the site. (Loading is discussed in Chapter IV.) Superstructures fall into two very distinct
categories, those with very great overall strength and stiffness, typified by a battery of rectangular silos with deep walls in both directions, and others such as the normal steel-framed structure with no available overall strength. Theoretically we can imagine an intermediate type—a stiff-jointed framed building that could act as a deep Vierendeel girder with a limited amount of overall strength and overall stiffness available to resist general flexure, but this type would only be likely in districts subject to earthquake tremors. The next step is to calculate the loads on the columns as accurately as is possible, dividing this load into three categories. (1) Dead load. (2) Probable live load. (3) Improbable live load. These values should be tabulated and written on a drawing of the raft. (4) We must also estimate the probable intensity of unbalanced imposed load.

(5) The probable worst arrangement of this unbalanced imposed load. (6) The length of time during which this unbalanced imposed load is likely to remain in its worst probable arrangement.

The raft, or the superstructure, must be made strong enough and stiff enough to spread the loading and restrict relative settlements when all columns are fully loaded. Whether and to what extent the raft, or superstructure, should be made strong enough to spread uneven loading when this is entirely due to different columns carrying different percentages of their maximum live load, must be decided for each case.

The engineer's first conception of a raft is usually to utilise the lowest floor as one flange of a deep box girder as shown in Fig. 120. With modern buildings this is usually impossible as all sorts of services have to pass below this floor and the arrangement in Fig. 121 has to be accepted. Rafts are usually used on soft sites and beam-and-slab construction is used rather than a solid plain thick slab to save weight. On most sites where rafts are necessary, usually low-lying flat
land, the ground is uniform over the site. Where the ground varies appreciably from one part of the site to another it is almost certain that it varies quickly with depth and a deeper foundation will provide a better solution. If the raft has to spread large variations due to uneven soil or spread uneven mining subsidence, the problem becomes very much more difficult.

Flexible Raft under Stiff Superstructure on Uniform Ground

With a stiff superstructure, such as the battery of rectangular silos outlined in Fig. 103, on a uniform site the maximum intensity of loading under the worst arrangement of live load is, theoretically, about \( w_D + 1.25w_L \) where \( w_D \) is the intensity due to dead load and \( w_L \) is the intensity which would be caused by live load if the whole structure were loaded. Whether we should cover this case under normal working stresses or whether, and to what extent, we should allow the calculated stresses to exceed the normal values depends on the degree of probability of such a distribution of live load occurring and the length of time for which it might persist. In an actual silo, where such a distribution is extremely probable, it should be covered. When the superstructure is stiff enough and strong enough to spread uneven loading uniformly over the site the function of the raft is merely to span from column to column and no overall strength or overall stiffness of the raft beams is necessary. A silo building with its close and uniform column spacing makes an ideal superstructure for a raft foundation. Curiously enough the author can recall only one such combination. The other silos are either on spread footings, footing beams or, most often, on piles. Suppose we have a battery of silos with bins of \( 12 \text{ ft.} \times \text{12 ft.} \) with \( 7\text{-in.} \) walls, the columns under the bins being \( 30\text{-in.} \times 30\text{-in.} \) spaced at \( 12 \text{ ft.} \) \( 7\text{ in.} \) centres and carrying \( 475 \) tons each. This means a net loading intensity of \( 3 \) tons per square foot.

We can try the variety of flat-slab raft shown in Fig. 122 (page 122). The load passing directly to the ground under the column is \( 3 \) tons per square foot on an area of \( 30\text{ in.} \times 30 \text{ in.} = 3 \times 2240 \times 2.5 \times 2.5 = 42,000 \text{ lb.} \)

The total column load = \( 475 \times 2240 = 1,060,000 \text{ lb.} \)

The minimum overall thickness for punching shear at 200 lb. per square inch = \( \frac{1,060,000 - 42,000}{4 \times 30 \times 200} = 42.3 \text{ in.}, \) say \( 3 \text{ ft.} \ 7\text{ in.} \)

There have been many different regulations governing the design of flat-slabs and many opinions expressed. This is not the place to discuss them. In this particular case there can be very little variation in loading from one panel to the next but the foundation is certain to carry its full design load. Adopting the following moments: strip A: \( + \frac{WL}{40} \) and \( - \frac{WL}{28} \); strip B: \( + \frac{WL}{72} \) and \( - \frac{WL}{72} \), as \( W = 1,060,000 \text{ lb.} \) and \( L = 12.6 \text{ ft.} \) these moments are as follows.

On strip A: \( + 4,000,000 \text{ lb.}-\text{in.} \) and \( - 5,720,000 \text{ lb.}-\text{in.} \)

On strip B: \( + 2,220,000 \text{ lb.}-\text{in.} \) and \( - 2,220,000 \text{ lb.}-\text{in.} \)

The width of each strip is half of \( 12 \text{ ft.} \ 7\text{ in.} \), that is, \( 75.5 \text{ in.} \)

With stresses of \( 1000 \) and \( 18,000 \text{ lb. per square inch} \) and \( m = 15, \frac{M}{bd^2} = 193 \).

The minimum thickness of slab at mid-span of strip A is \( \sqrt{\frac{4,000,000}{193 \times 75.5}} = 16.6 \text{ in.} \).
say 1 ft. 8 in. overall. In order to cover the 45-deg. shearing line, the drop-panel must be 7 ft. wide (see Fig. 122). The minimum thickness of drop is \( \sqrt{\frac{5,720,000}{193 \times 84}} = 18.8 \text{ in.} \); we require much more than this for resistance to shearing.

We have still to check the shearing strength of the 1-ft. 8-in. slab at the lower edge of the 45-deg. "spread line".

Effective depth \( d_1 = \text{say, 17.5 in.}; \) lever-arm = 15 in.; total width of spread = 30 in. + 2(40.5 in.) = 111 in. = 9.25 ft.

Effective shearing force = 1,060,000 - 9.25 \times 6720 = 484,000 lb.

Total shearing resistance = 4 \times 111 \times 15 \times 100 = 666,000 lb.

\[ A_{sh} \text{ at mid-span of strip } A = \frac{4,000,000}{18,000 \times 15} = 14.85 \text{ sq. in.,} \]

say 19 bars of 1-in. diameter.

\[ A_{sh} \text{ under column } = \frac{5,720,000}{18,000 \times 0.86 \times 40.5} = 9.13 \text{ sq. in.,} \]

say 12 bars of 1-in. diameter.

\[ A_{sh} \text{ in strip } B = \frac{2,220,000}{18,000 \times 15} = 8.25 \text{ sq. in.,} \]

say 11 bars of 1-in. diameter.

The design so far worked out is shown in Fig. 122. This may be compared with the stepped footing in Fig. 80. Two criticisms at once arise. Firstly the slab only 1 ft. 8 in. thick looks thin compared with the 1-ft. 11-in. upstanding block. The design would look more substantial if the slab were increased to 1 ft. 10 in., thus reducing the upstand to 1 ft. 9 in. Secondly the slab would be concreted first and the upstanding blocks the next day. Even if they are tied together by a cage of heavy links (similar to those in Fig. 80) we still have the feeling that we could have two separate layers instead of one solid whole. If we change to a solid slab 3 ft. 7 in. thick as in Fig. 123 (page 122) we save about 8 cwt. of steel per panel but use about an extra 8 cu. yd. of concrete. We should also save the shuttering for the upstanding block, about 6 sq. yd., but we should lose some of this in extra shuttering for construction joints. Most silos have conveyor trenches below the ground floor and the empty passageways between the upstanding blocks in Fig. 122 might be very useful for these, whereas the solid slab might have to be pushed down bodily to accommodate them. We should, of course, have the overriding satisfaction of knowing that we had one solid block of concrete under each column and not two separate layers. Each panel for a thick raft requires 2x cu. yd. of concrete. A job of this size would normally have a concreting plant capable of turning out this amount per hour.

If we do not insist on covering for punching shear the minimum effective depth \( d_1 \) for beam shearing on the 45-deg. line for a plain slab raft at 100 lb. per square inch (taking the lever-arm as 0.86\( d_1 \)) is given by

\[ 0.86d_1 \times 4(30 + 2d_1) \times 100 = 1,060,000 - \frac{6720(30 + 2d_1)^2}{144}, \]

giving \( d_1 = 26.2 \text{ in. or, say 2 ft. 5 in. overall.} \) Compared with the 3-ft. 7-in. raft in Fig. 123 this would decrease the concrete by nearly 7 cu. yd. per panel but
would increase the steel by about 9 cwt. This design complies with the requirements of the B.S. Code.

**Loading on Superstructure.**—The maximum intensity of loading on the ground occurs when only half the building is loaded as shown in Fig. 124 and is equal to \( w_D + 1.25w_L \) where \( w_D \) and \( w_L \) are the intensities of loading on the ground that would occur if the whole building were fully loaded. This not only increases the loading on the outside panels (and outside columns) but, if we have a flexible raft of the type shown in Fig. 122 all the general moment due to this arrangement of loading must bear on the stiff superstructure. Moments of \( \frac{w_LL^2s}{216} \) and \( \frac{-w_LL^2s}{216} \) occur in the walls, or frames, of the superstructure where \( s \) is the spacing of these walls.

Assuming that we have a spacing of 12 ft. 7 in. as in Fig. 122 and taking \( w_L \) as 5000 lb. per square foot, then if \( L \) in Fig. 124 is 101 ft., these moments are

\[
\pm \frac{5000 \times 101^2 \times 12.6}{216} = \pm 2,980,000 \text{ lb.-ft. per wall.}
\]

A much more severe moment is caused in the stiff superstructure by the loading in Fig. 125 where only the middle half of the building is loaded. This moment is \( \frac{w_LL^2s}{32} \), or \( \frac{5000 \times 101^2 \times 12.6}{32} = +20,100,000 \text{ lb.-ft. per wall.} \) If
only the outside quarters of the building were loaded and the middle half left empty a moment of \(-\frac{w_LL^2}{32}\) would be caused in each wall.

If a silo wall is 7 in. thick and 100 ft. high it can be used as a beam having an effective depth of about 97 ft., the moment of resistance of which is \(193bd_1^2 = 193 \times 7 \times 1164^2 = 1,825,000,000\) lb.-in. The tensile reinforcement necessary to resist this moment would be difficult to accommodate in a 7-in. wall but we could reasonably insert about 30 sq. in. which would resist a moment of about 630,000,000 lb.-in. We must also remember that we may have to allow for some compression on the concrete in the walls due to local bending. If \(w_L\) in Fig. 125 is 5000 lb. per square foot, \(M = \frac{5000L^2 \times 12.6}{32}\) lb.-ft. and \(\frac{5000L^2 \times 12.6}{32} \times 12 = 630,000,000\) or \(L = 163\) ft., meaning that we could utilise this wall to spread uneven live loading of the type shown in Fig. 125 if the building were not wider than 163 ft.

If the ground were not quite uniform the load would tend to be higher on the harder parts of the site, particularly when the structure was first loaded. This would not only increase the load on those panels of the raft that stand on the harder areas but would, by so doing, increase the load on the columns above those panels. If the variation is slight it could be ignored particularly if we have already made full allowance for the worst possible arrangements of live load.
Stiff Raft under Flexible Superstructure on Uniform Ground

Some very soft wet sites have been dealt with by building a deep cellular basement crossed in both directions by deep reinforced concrete walls. Such substructures are close to the loading conditions on a floating caisson and it is this type of raft which is here meant by the term "stiff".

Live Loading.—The values of the dead loads and their distribution may be determined within close limits. In many cases the maximum live load can only be determined by processes little better than guesswork and its distribution is usually not controlled. All rafts under flexible superstructures should be designed to cover arrangements of a reasonable amount of live loading. The interpretation of what is reasonable must be determined for each particular case.

If we have a building with columns regularly spaced at $l$ centres loaded as shown in Fig. 126 and standing on a raft which is very stiff compared with the superstructure and very stiff compared with the elastic compressibility of the soil,

the intensity of loading on the soil is $(w_D + 0.5w_L)$ where $w_D$ is the intensity due to dead load and $w_L$ the intensity due to full live loading. The loading on the beams in the raft may be split into two, $w_D + 0.25w_L$ acting on a span $l$ and $0.25w_L$ acting on a span $2l$ as in the similar case in Figs. 104 and 105. The moment under the more lightly loaded columns is $-(w_D - 0.25w_L) \frac{Ps}{12}$, and that under the more heavily loaded columns is $-(w_D + 1.25w_L) \frac{Ps}{12}$ where $s$ is the spacing of the raft beams. If the live load covered the whole of each floor these moments would both be $-(w_D + w_L) \frac{Ps}{12}$. The values of $0.75W_L$ and $0.25W_L$ in Fig. 126 are for simply supported beams. For continuous beams of uniform section these coefficients become $0.773W_L$ and $0.227W_L$. For continuous beams of varying section they could be, in the limit, $1.0W_L$ and zero. If we have a building of length $L$ carrying live load over a central length $kL$ where $k$ is a fraction (see Fig. 127) and assume that the raft spreads this uniformly over the site then the intensity of pressure due to this loading is $kw_L$ where $w_L$ is the intensity per square foot caused by live load on the whole building. The moment at the mid-
point of the beams of the raft due to this loading is \( w_L \frac{kL}{2} \left( \frac{L}{4} \frac{kL}{4} \right) s \), where \( s \) is the spacing of the raft beams. This moment is a maximum when \( k = 0.5 \) and \( M_{\text{max}} = \frac{w_LL^2s}{32} \) as in Fig. 125.

Similarly if we have live load only on the outside quarters as in Fig. 128 the maximum moment is \( -\frac{w_LL^2s}{32} \).

If the live load covers the right-hand half of all floors as shown in Fig. 129 the intensity of live loading on the soil varies from \(-0.25w_L\) to \(+1.25w_L\) (assuming \( w_D \) is greater than \( 0.25w_L \)) and moments of \(-\frac{w_LL^2s}{216}\) and \(+\frac{w_LL^2s}{216}\) occur on the raft beams at points distant \( 0.333L \) from the left- and right-hand ends (where \( s \) is the spacing of the raft beams). Most rafts have beams in both directions and if
one beam running from east to west is more heavily loaded than the nearest two parallel beams then the beams running from north to south will carry the load from the more heavily loaded to the more lightly loaded beams. To get the full effect of the arrangements of live loading in Figs. 126 to 129 this loading must continue right across the building in a direction at right-angles to the plane of the diagram.

With an office building the average imposed load is unlikely to exceed 10 lb. per square foot per floor and the distributions shown in Figs. 126 to 129 are most unlikely to occur. If Fig. 127 shows a building with columns at 20 ft. centres in both directions with eight floors carrying 10 lb. per square foot, \( w_L = 8 \times 10 = 80 \) lb. per square foot and \( s = 20 \) ft. If \( L = 160 \) ft., \( \frac{w_L L^2 s}{32} = \frac{80 \times 160^2 \times 20}{32} = 1,280,000 \) lb.-ft. If the local authority insisted on calculating for an imposed load of 50 lb. per square foot on all floors distributed as shown in Fig. 127, this moment would be 6,400,000 lb.-ft.

If the effective loading intensity on the soil under fully loaded conditions (live plus dead) were 1500 lb. per square foot the average loading on the beams, assuming beams run in both directions, is 15,000 lb. per foot and the maximum local moment on the beam immediately below each column would be about \( \frac{15,000 \times 20^2}{12} \) or only 500,000 lb.-ft. The worst possible arrangements of live loading shown in Figs. 127 and 128 are generally so unlikely that they need not usually be fully covered at normal working stresses. The author suggests the modified arrangement in Fig. 130, the live load increasing from nothing at the ends to a maximum of \( w_L \) in the centre. If this loading were carried on a stiff raft on yielding soil the moment at the mid-point would be \( \frac{w_L L^2 s}{48} \) or two-thirds of the moment caused by the arrangement in Fig. 127 when \( k = 0.5 \).

If the loading were zero at mid-point increasing to \( w_L \) at both ends the
moment would be \(-\frac{wL^2s}{48}\) or two-thirds of the moment caused by the arrangement in Fig. 128. To what extent this should be covered at normal working stresses must be decided for each particular case.

It must be emphasised that these very high moments due to a live load unevenly arranged can only be fully realised if we have a very stiff raft or superstructure on a yielding foundation.

![Diagram](image)

**Fig. 130.**

**Combined Live and Dead Load.**—With a stiff raft it must be assumed that the loading on the ground is distributed uniformly, or with a straight-line variation if the centre of gravity of the total load is an appreciable distance from the centre of the raft. The definition of "stiff" depends theoretically on the compressibility of the soil but on most sites where a raft is really necessary any raft whose beams or cross-walls are deeper than one tenth of the greater length of the raft may be classed as "stiff". If the loading is extremely irregular the resulting moments may be reduced by adding kentledge (brick walls, mass concrete, clean ballast, etc.) to the more lightly loaded sections, provided that the total load is not thereby increased beyond the safe bearing capacity. Additional loading always increases the total settlement but on sites where rafts are really needed, considerable settlement has to be faced in any case.

**Flexible Raft under Flexible Superstructure on Uniform Ground**

**Live Loading.**—The term "flexible" applied to a raft carrying short-term live loading means that its ability to deflect is of the same order of magnitude as the elastic yielding of the ground. If we have a reinforced concrete beam of effective depth \(d_1\) and length \(L\) as shown in Fig. 131 so loaded that the compression stress at the top is everywhere 1500 lb. per square inch with \(E_s=3,000,000\times 144\) lb. per square foot and the tensile stress at the bottom is everywhere 20,000 lb. per square inch with \(E_s=30,000,000\times 144\) lb. per square foot, the radius of curvature \(R\) would be \(855d_1\) and the deflection \(\Delta\) would be \(\frac{L^2}{6850d_1}\).

If \(L=160\) ft. and \(d_1=4.5\) ft., then \(\Delta=\frac{160^2}{6850\times 4.5}=0.83\) ft. = 10 in. = \(\frac{L}{193}\).
The deflection shown in Fig. 131 is the theoretical maximum. All practical raft beams have local bending moments and only a fraction of their strength is available to cope with general flexure. On some sites uniform settlements of 12 in. have been expected and allowed for but a relative settlement of 10 in. between the middle and ends of a building would seriously jeopardise any superstructure not specifically designed to accommodate this movement. If we accept the idea that vertical downward movement of the ground, within the safe bearing capacity, is directly proportional to the intensity of loading, then a flexible raft cannot spread a concentrated load uniformly over the ground. Fig. 132 shows the calculated moment at mid-point of a flexible beam of uniform section resting on compressible ground. It is assumed that the upward reaction of the ground on the underside of the beam is everywhere proportional to the downward movement of the beam. The coefficient \( C_2 \) is equal to the intensity of loading per square foot that will cause a compression of one foot, multiplied

![Diagram](image-url)

by the spacing of the beams in feet. If a vertical downward compression of the ground of 1 ft. produced an upward reaction of 2000 lb. per square foot and if the beams were spaced at 15 ft. centres then \( C_2 = 2000 \times 15 = 30,000 \).

If the raft beam deflected 10 in. then the ends would move down only 2 in. when the centre moved down 12 in. and if the upward reaction under the centre of the beam were 2000 lb. per square foot, the reaction under each end would be only \( \frac{2 \text{ in.}}{12 \text{ in.}} \times 2000 = 333 \text{ lb. per square foot} \).

If we assume that the raft beams run right across the building and are of uniform section throughout, we can calculate the moments in the beam for simple
arrangements of loading if we assume that upward ground reaction is always proportional to downward movement of the beam. If a downward movement of 1 ft. produces an upward reaction of 5000 lb. per square foot and if the beam in Fig. 132 is 2 ft. wide and 6 ft. deep, \( I = \frac{2 \times 6^3}{12} = 36 \text{ ft.}^4 \). If the beams are spaced at 18-ft. centres, \( C_2 = 18 \times 5000 = 90,000 \). If \( E = 3,000,000 \times 144 \text{ lb. per square foot} \) and \( L = 100 \text{ ft.} \),

\[
L \times \sqrt[4]{\frac{C_2}{EI}} = 100 \times \sqrt[4]{\frac{90,000}{3,000,000 \times 144 \times 36}} = 4.92.
\]

From the curve the moment \( M \) at mid-span is \(-0.077WL\). If \( W = 44,800 \text{ lb.} \), then \( M = -0.077 \times 44,800 \times 100 = -345,000 \text{ lb.-ft.} \). If the beam were infinitely stiff instead of being flexible the central moment would be \(-560,000 \text{ lb.-ft.} \). The flexing of the beam has therefore relieved it of 38 per cent of its moment.

The negative sign indicates tension on the underside and this will be made clear in the detailed analysis that follows.

Suppose we have a uniform raft beam of length \( L \) as shown in Fig. 133 carrying loading of the type suggested in Fig. 130 varying from nothing at each end to a maximum of \( \frac{1}{4}C_2L \) at the centre such that the intensity of applied loading at any point distant \( x \) from one end is \( C_1x \); and further suppose that this raft beam is supported on ground of such a nature that the upward reaction from the soil at any point is directly proportional to the downward movement of the beam at that point. Moreover we suppose that the compression of the soil is immediate and that there is no time factor involved as there is with long-term settlement in natural clays. We have in fact removed the natural soil and substituted a deep stratum of theoretical elastic jelly whose Poisson’s ratio is zero. It is not suggested that any natural soil comes very near to behaving in this way but the solution of this problem indicates the type and order of magnitude of the moments and deflections we could expect in practice. If the vertical downward movement of the beam at point \( x \) in Fig. 133 is \( y \) then the upward reaction of the ground at this point is \( C_2y \) where \( C_2 \) is a constant for any one given type of our theoretical soil. The values \( C_1 \) and \( C_2 \) are therefore constants for any one problem. The net intensity of downward loading is \( C_1x - C_2y \) and if \( I \) is the geometrical moment of inertia of the cross-section of the beam then \( EI \frac{d^4y}{dx^4} = C_1x - C_2y \)

\[
y = \frac{C_1x}{\alpha EI} + \left[ e^{\alpha x/\sqrt{2}} \left( A \sin \frac{\alpha x}{\sqrt{2}} + B \cos \frac{\alpha x}{\sqrt{2}} \right) \right] + e^{-\alpha x/\sqrt{2}} \left( C \sin \frac{\alpha x}{\sqrt{2}} + D \cos \frac{\alpha x}{\sqrt{2}} \right)
\]

where \( \alpha = \sqrt[4]{C_2/EI} \)

The values \( A, B, C \) and \( D \) are found from the conditions

\[
EI \frac{d^2y}{dx^2} = 0 \quad \text{when} \quad x = 0; \quad \text{therefore} \quad A = C.
\]

\[
EI \frac{d^2y}{dx^3} = 0 \quad \text{when} \quad x = 0; \quad \text{therefore} \quad 2A = B - D.
\]

Also \( EI \frac{d^3y}{dx^3} = 0 \) when \( x = \frac{L}{2} \), and \( EI \frac{dy}{dx} = 0 \) when \( x = \frac{L}{2} \).
Alternatively if we take these conditions into account we can write

\[ y = \frac{C_1}{C_2} x + y_0 \cos \frac{\alpha x}{\sqrt{2}} \cosh \frac{\alpha x}{\sqrt{2}} + \frac{y_1}{\alpha \sqrt{2}} \left[ \sin \frac{\alpha x}{\sqrt{2}} \cosh \frac{\alpha x}{\sqrt{2}} + \cos \frac{\alpha x}{\sqrt{2}} \sinh \frac{\alpha x}{\sqrt{2}} \right], \]

where \( y_0 \) and \( y_1 \) are the values of \( y \) and \( \frac{dy}{dx} \) when \( x = 0 \) and \( \alpha = \frac{C_1}{\sqrt{2}C_2} \) as before.

The deflected shape of the beam is a periodic curve of diminishing amplitude. Theoretically we may have a case where part of the beam rises above the original ground surface and loses contact with the ground. This might happen with a pavement carrying a concentrated load (see Fig. 164) but cannot happen with a raft where there is always sufficient dead load at all points to ensure that the beam is everywhere below the original surface. The amount to which the beam bends and deflects depends on the ratio between \( C_2 \) and \( I \). If the line AA in Fig. 134 shows the level of the underside of the beam in Fig. 133 before

[Figure 133]

[Figure 134]

loading, the level of the underside after loading must lie somewhere between the lines BB and ACA depending on the value of \( C_2/I \). The total area of the sinking curve below level AA is a constant for any given values of \( C_1 \) and \( C_2 \) whatever the value of \( I \). The total applied load is \( \frac{1}{2} \frac{C_1 L}{2} \times L = \frac{C_1 L^2}{4} \). The total upward reaction of the ground is \( C_2 L \times \text{(average value of } y) \) and these two must be equal.

If the beam were infinitely stiff it would sink uniformly to the line BB in Fig. 134 the vertical distance AB being equal to \( \frac{C_1 L}{C_2} \), the moment at mid-point being \(-0.0104 \frac{C_1 L^3}{4}\) (tension on underside). If the beam had no structural stiffness the loading would pass straight through and the beam would consist of two straight lines ACA, the vertical sinkage between level AA and point C being \( \frac{C_1 L}{2C_2} \). With a relatively stiff beam we should have a curve D approximating to line BB. With a relatively flexible beam we should have a curve E approximating to the lines ACA and the bending moment at mid-point in this beam would approximate to \(-\frac{E I \alpha C_1}{\sqrt{2}C_2}\). The moment at mid-point for various values of \( \alpha \) in
terms of $EI\alpha \frac{C_1}{C_2}$ is shown in Fig. 135. Having regard to the directions in which the loads and co-ordinates are measured the negative sign indicates tension on the underside.

If $\frac{1}{4}C_2L$ is the loading in pounds per foot run on the beam at mid-point and $C_2$ is the intensity of loading in pounds per square foot that causes a sinkage of 1 foot multiplied by the spacing of raft beams in feet, and

$$\begin{align*}
L &= \text{length of beam in feet}, \\
E &= \text{elastic modulus in pounds per square foot}, \\
I &= \text{geometrical moment of inertia in foot units},
\end{align*}$$

then the moment will be in pounds-feet. For example if $L=160$ ft., and if the intensity of applied loading at mid-point is 1600 lb. per foot, $C_1(\frac{1}{4}L)=1600$ or $C_1=20$.

![Diagram](image)

If the ground sinks 1 ft. under a pressure of 2000 lb. per square foot and if the raft beams are spaced at 20-ft. centres, $C_2=2000 \times 20=40,000$. If $E=3,000,000 \times 144$ lb. per square foot and if the beam is 18 in. wide and 60 in. deep,

$$I = \frac{1.5 \times 5^3}{12} = 15.6 \text{ ft}^4.$$

$$\alpha = \frac{4}{\sqrt{3,000,000 \times 144 \times 15.6}} = \frac{1}{20.25}.$$

The value of $\frac{\alpha x}{\sqrt{2}}$ at mid-point is $\frac{1}{20.25} \times \frac{80}{\sqrt{2}} = 2.79$. Thus the value of $\frac{\alpha x}{\sqrt{2}}$ varies from zero at the end of the beam to 2.79 or 0.89\pi at the centre, which is less than half a complete periodic cycle.

Using the two equations which ensue from the conditions that $\frac{d^3y}{dx^3}=0$ when
\( x = 80, \) and \( \frac{dy}{dx} = 0 \) when \( x = 80, \) we have \( A = +0.000263, B = +0.000563, C = A \)
and \( D = +0.000376. \)

\[
M \text{ at mid-point} = \frac{EI\frac{dy}{dx}}{dx^2} \text{ (when } x = 80) \\
= \frac{EI}{20.25^2} \times [-0.00717] \\
= -\frac{3,000,000 \times 144 \times 15.6 \times 0.00717}{410} \\
= -118,000 \text{ lb.-ft. (tension on underside)}. \\
\]

Taking a value from Fig. 135, \( aL = \frac{I}{20.25} \times 160 = 7.9. \)

\[
\text{Moment at mid-point} = -0.7EI \alpha \frac{C_1}{C_2} \\
= -0.7 \times 3,000,000 \times 144 \times 15.6 \times \frac{I}{20.25} \times \frac{20}{40,000} \\
= -117,000 \text{ lb.-ft.} \\
\]

If the beam were infinitely stiff and went down to line BB in Fig. 134, the moment would be \(-0.0104C_1L^3 = -0.0104 \times 20 \times 160^3 = -855,000 \text{ lb.-ft.}\) This beam, by flexing, has relieved itself of 86 per cent. of the moment.

The shape of the beam is nearer to the curve E than to the curve D in Fig. 134. The expression \( EI \alpha \frac{C_1}{C_2} \) in Fig. 135 is a little difficult to visualise but we can convert it into terms of \( C_1L^3 \) for any given value of \( aL \) and can thus give the moment directly in such terms. This is shown in Fig. 136.

![Diagram](image-url)
If we check back on the last example,

\[ L \times \frac{4C_2}{\sqrt{EI}} = La = \frac{160}{20 \times 25} = 7.9. \]

Moment at mid-point = 

\[ -0.0014C_1L^3 \]

\[ = -0.0014 \times 20 \times 160^3 \]

\[ = -115,000 \text{ lb.-ft.} \]

Again if a pressure of 1 ton per square foot causes a sinking of 2 in. and the beams are 24 in. \( \times \) 72 in. deep spaced at 25-ft. centres and 75 ft. long and loaded to a maximum of 4,000 lb. per foot at the centre point

\[ \frac{1}{4}C_1L = 4000 \quad \text{or} \quad \frac{1}{4}C_1 \times 75 = 4000 \quad \text{or} \quad C_1 = 107. \]

\( C_2 = 6 \) tons per square foot \( \times \) 25 ft. spacing \( = 6 \times 2240 \times 25 = 336,000 \text{ lb. per linear foot}; \]

\( L = 75; \quad I = \frac{2 \times 6^3}{12} = 36 \text{ ft.}^4 \)

\[ L \times \frac{4C_2}{\sqrt{EI}} = 75 \times \frac{4\times336,000}{3,000,000 \times 144 \times 36} = 5.12. \]

From Fig. 136, the moment at mid-point is 

\[ -0.0048C_1L^3 = -0.0048 \times 107 \times 75^3 = -217,000 \text{ lb.-ft.} \]

The additional steel to resist this moment would be \( \frac{217,000 \times 12}{20,000 \times 60 \text{ in.}} = 2.17 \text{ sq. in.} \); this is in addition to the steel required for local moments for spanning from column to column. If we now take the case of a raft beam carrying a load arranged as in Fig. 137 and add this to the loading in Fig. 133 we get uniform loading, uniform sinking and no general bending moment in the raft beam. The effect of the loading in Fig. 137 is therefore equal and opposite to that of the loading in Fig. 133 and the moment at the mid-point of the beam is equal to that given in Figs. 135 and 136 but of opposite sign (tension on top). If the raft beam in the first example has columns at 20-ft. centres and the combined live and dead loads are equal to 20,000 lb. per foot run of beam, the local moments are

\[ +20,000 \times \frac{20^2}{24} = +333,000 \text{ lb.-ft.} \quad \text{and} \quad -20,000 \times \frac{20^2}{12} = -667,000 \text{ lb.-ft.} \]

If we add general hogging or sagging moments of plus or minus 118,000 lb.-ft. to cover the uneven arrangement of the live load, these moments are increased to plus 451,000 and minus 785,000 lb.-ft. as shown in Fig. 138.

The values of \( C_1 \) which we have so far assumed (20 in the first example and 107 in the second) are only suitable for office buildings. If we have a six-story warehouse with an imposed load of 200 lb. per square foot, \( w_L \) is 6 \( \times \) 200 = 1200 lb. per square foot. With raft beams at 20-ft. centres this is 24,000 lb. per foot run of beam. If the raft is 100 ft. long then

\[ \frac{C_1L}{2} = \frac{C_1 \times 100}{2} = 24,000 \text{ and } C_1 = 480. \]
If a loading intensity of 1 ton per square foot causes a sinking of 1\(\frac{1}{4}\) in. and the raft beams are 2 ft. x 6 ft. deep \(C_2=8 \times 2240 \times 20=358,000\) lb. per foot; \(I=\frac{2 \times 6^3}{12}=36\).

\[
L \times \sqrt[4]{\frac{C_2}{EI}} = 100 \times \sqrt[4]{\frac{358,000}{3,000,000 \times 144 \times 36}} = 6.93.
\]

From Fig. 136, the moment at mid-point is 
\(-0.0021C_1L^3 = -0.0021 \times 480 \times 100^3 = -1,010,000\) lb.-ft.

If the columns were spaced at 20-ft. centres along the beam and if the combined live and dead load per foot of beam were 25,000 lb. (with a mesh-panel slab) the local moments would be, say

\[+25,000 \times \frac{20^2}{24} = +417,000\] lb.-ft. and \[-25,000 \times \frac{20^2}{12} = -835,000\] lb.-ft.

If we add the general hogging and sagging moments of \(\pm 1,010,000\) lb.-ft., these become \(+1,427,000\) and \(-1,845,000\) lb.-ft.

We have so far taken results from Figs. 135 and 136, both of which show the value of the moment against the value of \(L \times \sqrt[4]{C_2/EI}\). This again is a difficult value to envisage but it can be made more tangible by substituting actual values. If we start with a standard reinforced concrete beam which is \(B\) ft. wide and \(D\) ft. deep such that \(D=3B\) and if we then assume that \(D\) is some percentage of the overall length \(L\), say 4 per cent., then \(D = \frac{L}{25}\) and \(B = \frac{L}{75}\); \(I = \frac{BD^3}{12} = \frac{L}{75} \times \frac{L^3}{25^3} \times \frac{1}{12}\), giving \(L = 61.3 \sqrt[I]{I}\).
If $E = 3,000,000 \times 144$ lb. per square foot, $\sqrt{E} = 144$, and $L \times \sqrt{(C_2/E)} = 0.425 \sqrt{C_2}$.

If $C_2$ to produce a sinking of 1 ft. is 20 tons per foot run of raft beam is 44,800 lb.; $\sqrt{C_2} = 14.52$, and $L \times \sqrt{(C_2/E)} = 6.18$.

From Fig. 136, the moment at the mid-point under triangular loading is $-0.0031C_1L^3$. Putting $D = 0.06L = 0.08L$, etc., and $C = 40, = 80, = 120$ tons per foot, etc., we get the curves in Fig. 139 which show the moment in terms of $C_1L^3$ directly related to $C_2$.

If the proportions of a beam are not in the ratio of $D = 3B$, we can still use Fig. 139 by finding the equivalent standard beam. If $B = 1.5$ ft. and $D = 6$ ft., $I = \frac{1.5 \times 6^3}{12} = 27 = \frac{1.86 \times 5.58^3}{12}$. The beam is thus equivalent to a standard beam 1.86 ft. wide by 5.58 ft. deep.

Having worked out all these moments it is useful to remember that in calculating the bearing capacity of a granular soil we have ignored seven of its eight physical characteristics, including shearing strength and elastic compressibility, and based the result entirely on its internal friction. In calculating the bearing capacity of a clay we have again ignored seven characteristics, including internal friction and elastic compressibility, and based the result entirely on its shearing strength. In calculating the moments in these raft beams we have ignored both friction and shearing strength and based our figures exclusively on immediate elastic compressibility. On sites where rafts are necessary, for example under a multi-storied building whose total weight exceeds 90 per cent. of the total safe bearing capacity of the site, we must also consider the permissible relative settlements of the columns and choose an appropriate depth of beam. Under an office building a depth of 4 to 5 per cent. of the length of the building may be assumed to start with, and 6 to 7 per cent. under a warehouse. The stiffer the raft beams the higher the moments caused by unequal loading but the smaller their deflections.
If relative settlement of the superstructure does not matter, a plain slab raft might be best although the additional weight would probably rule it out. The question of the deflection of the raft beams is discussed later in this chapter.

**Combined Live and Dead Loading.**—Fig. 140 shows an idealised plan of a raft, although actual rafts are never so simple. Suppose the bearing pressure has been fixed at $\frac{1}{2}$ ton per square foot ($\frac{1}{2}$ ton = 1680 lb. per square foot). We begin by estimating the self-weight of the raft. Guessing a 12-in. slab, the maximum effective upward load on any panel is 1680 - 150 = 1530 lb. per square foot. The total effective load on one panel is $400 \times 1530 = 612,000$ lb. The moment is, say $\frac{W}{36} = \frac{612,000}{36} = 17,000$ lb.-ft. per foot = 204,000 lb.-in. per foot.

With stresses of 1000 and 18,000 lb. per square inch and $m = 15$, the minimum $d_1 = \sqrt{\frac{204,000}{193 \times 12}} = 9.36$ in.

If the raft is 100 ft. x 100 ft. and we assume that the depths of the beams is 6 per cent. of this dimension, that is, 6 ft. deep, and that they are 2 ft. wide, we have a preliminary section as shown in Fig. 141. The beams are 5 ft. x 2 ft. net and weigh 1500 lb. per foot; four half beams each 20 ft. long weigh 60,000 lb. The splays at the corners might add 5000 lb. The panel therefore weighs

- slab $400 \times 150 = 60,000$ lb.
- beams = 60,000 ,,
- splays = 5000 ,,

125,000 lb., say 56 tons, or 0.14 ton per square foot average.

The maximum load a panel of ground will support is

$$20 \times 20 \times \frac{1}{2} = 300 \text{ tons gross}$$

Deduct weight of raft = \(\frac{56}{244}\) tons net.

Assume the loading in tons from three consecutive columns Nos. 31, 32 and 33 in Fig. 140 is as follows.

<table>
<thead>
<tr>
<th></th>
<th>Col. 31</th>
<th>Col. 32</th>
<th>Col. 33</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load</td>
<td>175</td>
<td>205</td>
<td>175</td>
</tr>
<tr>
<td>Probable live load</td>
<td>15</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Improbable live load</td>
<td>40</td>
<td>50</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>230</td>
<td>270</td>
<td>230</td>
</tr>
<tr>
<td>Self-weight of raft</td>
<td>56</td>
<td>56</td>
<td>56</td>
</tr>
<tr>
<td></td>
<td>286</td>
<td>326</td>
<td>286</td>
</tr>
<tr>
<td>Allowable bearing</td>
<td>300</td>
<td>300</td>
<td>300</td>
</tr>
<tr>
<td>Excess load</td>
<td>—</td>
<td>26</td>
<td>—</td>
</tr>
<tr>
<td>Excess bearing capacity</td>
<td>14</td>
<td>—</td>
<td>14</td>
</tr>
</tbody>
</table>

If the beam extending from column No. 31 to column No. 33 in Fig. 140 has sufficient surplus strength to carry a central concentrated load of 26 tons on a span
of 40 ft., the excess load on column No. 32 is easily transferred to the two neighbouring panels.

In practice such simple cases rarely occur. The regular spacing of columns may be impossible owing to irregularity of the site, location of lifts and stairs or the architectural need for large column-free rooms. All large office buildings have internal areas or light wells. If, after all the excess load has been spread, there are still some areas where the load is appreciably less than the average, the loading on these should be increased to equal the loading on adjoining panels, omitting the improbable live load, by adding sand or ballast. The bearing capacity of some silts may be increased by driving consolidation piles (say 10 in. x 10 in. x 25 ft. piles at 5 ft. centres in both directions) and excess loading may be dealt with by driving such piles along the beams under the overloaded panels. This really begs the question because we have assumed a uniform site, that is of course unless the loading is so high that the whole site has to be piled. The whole raft must be drawn in plan and all column loads and bearing areas tabulated. (This is easy on a drawing board but impracticable, for a large raft, on the page of a book.) It is then useful to draw a second plan showing only the excess loads.
and excess bearing capacities. If the building is symmetrical about one or both centre lines and uniform in spacing and construction the work is greatly simplified. Otherwise there is often the choice of spreading excess load either on the beams running north to south, on those running east to west, or on both. A simple idealised raft beam under a building 160 ft. long is shown in Fig. 142. This shows excess loads of 26 tons carried to neighbouring panels causing moments of 260 tons-ft. or 7,000,000 lb.-in. due to these excess loads alone. These must be added to the local moments due to spanning from column to column under the uniform load. This effective load is \((300 - 56)\) tons per panel or \(122\) tons per beam on a local span of 20 ft. and the moment it causes under each column is about \(\frac{122 \times 20}{12}\) or 204 tons-ft. The "excess-load" moment of 260 tons-ft. and the local moment of 204 tons-ft. both occur under full design load and we now have to consider whether we would encounter worse conditions under partial loading. If we take the arrangement of load in Fig. 130 the central part of the

![Diagram of raft beam](image)

RAFT is fully loaded and we should here have local moments and "excess-loading" moments very nearly as great as under fully loaded conditions. We should also have, theoretically, on a yielding soil, a general moment given by Figs. 135, 136 and 139 and this moment depends entirely on the value of the compressibility factor for the soil \(C_2\). Some soils are very springy under applied point loads. Attempts to roll down a layer of hardcore cause an elastic wave under the roller which immediately recovers as the roller passes. This is largely due to the fact that the surface is free and can bulge up freely both in front of and behind the roller. To what extent this phenomenon would repeat itself under a large raft whose underside is several feet below ground level, is a matter of guesswork. Most soils on sites where rafts are required, are subject to long-term consolidation settlement which usually takes years to develop. If the arrangement of imposed load shown in Fig. 130 could not possibly persist for more than a few hours then we need only consider the immediate elastic compressibility of the soil. Assuming this is 10 tons per square foot per foot of sinkage or 200 tons (448,000 lb.) per foot run of beam and keeping to a beam 6 ft. deep (Fig. 141) and 160 ft. long

\[
I = \frac{2 \times 6^3}{12} \quad \text{and} \quad L = 160; \quad L \times \frac{4C_2}{NeI} = 11.7.
\]
From Fig. 136, the moment at mid-point is \(-0.006 C_1 L^3\) (tension on underside). With eight floors and an assumed unbalanced imposed load of 10 lb. per square foot per floor, we have 80 lb. per square foot total and with a 20-ft. spacing of the raft beams, this makes a maximum of 1600 lb. per foot run of beam.

\[
1600 = \frac{4}{3} C_1 L = \frac{4}{3} C_1 \times 160, \text{ from which } C_1 = 20.
\]

\[
-0.0006 C_1 L^3 = -0.006 \times 20 \times 160^3 = -49,300 \text{ lb.-ft.} = -22 \text{ tons-ft. (tension on underside).}
\]

If we take the view that the arrangement in Fig. 130 might persist indefinitely and that the long-term settlement expected is 1 ft. per ton per square foot, then \(C_2 = 44,800 \text{ lb. per foot of beam. And if, in addition, we take the pessimistic view that the unbalanced imposed load might include irregularly disposed partitions and permanent furniture and might reach a value of 20 lb. per square foot per floor, then } C_1 = 40; \text{ and } L \times \frac{C_2}{E} \sqrt{\frac{L}{E}} = 6.6.

From Fig. 136, the moment at mid-span is \(-0.0026 \times 40 \times 160^3 = -425,000 \text{ lb.-ft.} = -190 \text{ tons-ft.} \text{ These moments are in addition to the local moments and excess loading moments.}

Taking the optimistic view we have

local moment \(-204 \text{ tons-ft.}\)
excess-loading moment \(-260 \text{ "} \)
partial imposed loading moment \(-22 \text{ "} \)

-486 \text{ "}

Taking the pessimistic view we have

local moment \(-204 \text{ tons-ft.}\)
excess-loading moment \(-260 \text{ "} \)
partial imposed loading moment \(-190 \text{ "} \)

-654 \text{ "}

Assuming that we should allow our calculated stresses to rise 25 per cent. to cover this pessimistic figure, the moment to be covered at working stresses would be \(-654 \div 1.25 = -522 \text{ tons-ft.} = -14,000,000 \text{ lb.-in.} \)

If \(b = 24 \text{ in. and } d_1 = 67 \text{ in.}, \frac{M}{b d_1^2} = \frac{14,000,000}{24 \times 67^2} = 126. \text{ The corresponding positive moments near the mid-point caused by the loading in Fig. 137 are equal and opposite to those in Fig. 133 but the panels of raft near the mid-point would not be fully loaded. The local moment (dead load only) is } \frac{205}{2} \times \frac{20}{24} = +86 \text{ tons-ft.}

There is no positive excess-loading moment (Fig. 142) in this particular case, although in an irregular raft there would be some.

Taking the optimistic view we have

local moment \(+86 \text{ tons-ft.}\)
excess-loading moment \(-22 \text{ "} \)
partial imposed loading moment \(+108 \text{ "} \)
Taking the pessimistic view we have

<table>
<thead>
<tr>
<th>Component</th>
<th>Moment (tons-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>local moment</td>
<td>+86</td>
</tr>
<tr>
<td>excess-loading moment</td>
<td></td>
</tr>
<tr>
<td>partial imposed loading moment</td>
<td>+190</td>
</tr>
<tr>
<td></td>
<td>+276</td>
</tr>
</tbody>
</table>

Reducing this moment by 25 per cent. gives \( \frac{276}{1.25} = +221 \) tons-ft. = +5,930,000 lb.-in. (tension at top).

These are only approximate figures based on the simplest assumption, but they show that no general rules can be laid down about the allowance that should be made for moments caused by partial loading on flexible rafts. It is generally possible, as in this example, to set some sort of upper and lower limits but the final choice can only be made after a careful study of all the circumstances in each particular case, especially of the value \( C_2 \).

In the year 1920 stresses of 600 and 16,000 lb. per square inch were usual and the imposed load on an office floor was taken at 100 lb. per square foot. Today the opinion would be that the construction was 25 per cent. stronger than calculated and the imposed load assumed was at least 50 per cent. too high. A raft under such an office building designed to act under fully loaded conditions had a large margin of strength that could cover reasonable minimum variations due to partial loading. Now that the tensile stress has been increased to 20,000 lb. per square inch and the imposed load reduced to 50 lb. per square foot, some additional allowance should be made.

**Stepped-back Buildings**

Planning authorities now often insist on the upper stories of a building being set back to give light and air to adjoining property as in Fig. 143. The outer columns may carry only one suspended floor while the inner ones carry six or seven. If the site is so poor that the whole of it is required to carry the building then very heavy excess-loading moments occur on the raft beams. On a slightly better site a raft could be provided under the centre with the outside rows of columns carried on beam-and-slab or plain slab foundations. The "excess-load" moment could be of the type in Fig. 144, the moment due to this being \((48 \times 50) - (22 \times 25)\) tons-ft. = 1,850 tons-ft. (tension at the underside). To this must be
added the local moments. With such a heavy general moment it would probably not be necessary to consider partial loading for a building of the office type. A small allowance might be advisable for a warehouse with heavy imposed loads.

**Deflection of Flexible Rafts**

All calculations of deflection depend on the value $EI$. Little is known of the value of $E$ for concrete in bulk except that it may vary from 2,000,000 at the top of a column to 5,000,000 lb. per square inch at the bottom. With continuously high compressive stresses plastic yield develops amounting sometimes to about 1 in. in 100 ft. in prestressed concrete. An assumed value of 3,000,000 x 1.44 lb. per square foot seems fairly reasonable in calculating deflections. The correct value of $I$ for a T-section which becomes rectangular for reversed bending moments is also difficult to fix. If it is important, then both the $I$ of the full T-section and that of the net rectangle should be calculated and some intermediate value chosen depending on what kind of moment predominates and remembering that

![Fig. 144.](image)

![Fig. 145.](image)

moments in the middle half of the span cause most of the deflection. If the reinforcement exceeds 1 per cent. at the top and bottom of all sections allowance may be made for it.

The $I$ of the section in Fig. 145 may be taken as

$$
\frac{BD^3}{12} + \left(2 \times \frac{\phi BD}{100}\right) \times (15 - 1) \times (0.425D)^2 = BD^3(0.083 + 0.051\phi).
$$

Theoretically the local moments due to the beams spanning from column to column cause local deflection. With normal construction these are negligible. Excess-loading moments of the type in Fig. 142 may cause a little general bending but a closely approximate figure of the general deflections due to these can soon be calculated. We are really only concerned with loading of the type in Fig. 130 where there is a general bending in one direction only throughout the whole length of the raft beam and we are most particularly concerned with the case when this general deflection at the centre (Fig. 130) or ends (Fig. 137) is sufficiently large to affect the distribution of loading on the ground to an appreciable extent. Fig. 146 represents the case of a very stiff beam carrying a triangular load and resting on a very easily compressible soil. In the limit the upward reaction of the ground becomes uniform as shown. The upward deflection $\Delta$ ends above
the mid-point of a uniform beam loaded as in Fig. 146, and is \(0.0583 \frac{W}{2} \left(\frac{L}{2}\right)^3 \frac{1}{EI} = \frac{WL^3}{275EI}\); since \(W = \frac{C_L}{4}\), \(\Delta = 0.00091 \frac{C_L L^5}{EI}\).

The central moment is \(-0.0104C_L L^3\) (tension at underside). The deflection and central moment of the beam in Fig. 147 are equal and of opposite sign. If we

\[\text{Fig. 146.}\]

\[\text{Fig. 147.}\]

\[\text{Fig. 148.}\]

replace the stiff beam in Fig. 146 by the flexible beam in Fig. 133 the loading on the underside ceases to be uniform and we have the moments in Figs. 135, 136 and 139 and the central deflection of a flexible beam under these loading conditions is given in Fig. 148. The values in Fig. 148 are not the total downward sinking of the central point, they are the difference in level between the ends and the centre of the beam. The total sinking of the central point under this loading varies from
\[
\frac{C_1 L}{4C_2} \text{ for an infinitely stiff beam to } \frac{C_1 L}{2C_2} \text{ for a beam without any stiffness. The sinking of the ends under this loading varies from } \frac{C_1 L}{4C_2} \text{ for an infinitely stiff beam to zero for a beam without stiffness, all as shown in Fig. 134.}
\]

The term \( L \times \sqrt[4]{\frac{C_2}{EI}} \) appears again in Fig. 148 and we may again bring this into practical focus by assuming a value for \( E \) of 3,000,000 \times 144 lb. per square foot and a value of \( \frac{BD^3}{12} \) ft.\(^4\) for \( I \) where \( B = 0.33D \) all as in Fig. 139. The resulting curves are shown in Fig. 149. Here again the value of \( C_2 \) varies with the type of uneven loading. If the uneven load is transitory live load as in Fig. 130 then \( C_2 \) is the immediate elastic compressibility of the soil. If the uneven load is permanent dead load as in Figs. 143 or 150, then \( C_2 \) is the total long-term settlement coefficient. If we have a six-story warehouse, with an imposed load of 200 lb. per square foot on each floor, on a raft 100 ft. long with beams at 20-ft. centres and assume that the imposed load may be arranged as shown in Fig. 130, then the intensity of uneven loading at the mid-point of the raft beam is \( 6 \times 200 \times 20 \) or 24,000 lb. per foot of beam; \( C_1 = \frac{24,000}{50} = 480. \)
If the soil compresses 2 in. under a load of 1 ton per square foot during the time that this arrangement of load persists

\[ C_2 = 2240 \times 6 \times 20 \text{ ft.} = 269,000 \text{ lb.} \]

If we wish to restrict the value of \( \Delta \) in Fig. 148 or 149 to \( \frac{1}{4} \) in., \( \Delta = 0.0208 \text{ ft.} \). Now \( C_1 L = 480 \times 100 = 48,000 \); so \( \Delta = 0.000005435C_1L \), and, from Fig. 149, we should require a raft beam about \( 0.102L \) or 10 ft. deep. The moment caused by this arrangement of loading at the mid-point of the beam from Fig. 139 would be

\[ -0.007C_1L^3 = -0.007 \times 480 \times 1,000,000 \text{ lb.-ft.} = -3,360,000 \text{ lb.-ft.} \]

(tension at underside). If we accept a deflection of \( \frac{1}{4} \) in. then \( \Delta = 0.000000087C_1L \), and a raft beam 8 ft. deep would do. This beam would have a moment at mid-point of 

\[ -0.00525C_1L^3 \text{ lb.-ft.} \]

If the raft beams had no stiffness (the curve marked \( D = 0 \) in Fig. 149), then \( \Delta \) (when \( C_2 = 269,000 \)) is \( 0.0000019C_1L = 0.0000019 \times 480 \times 100 = 0.901 \text{ ft.} = 1.09 \text{ in.} \)

\[ \text{Fig. 150.} \]

We can check this reading of the curve directly because the sinking of the centre point of a beam with no stiffness under this loading is \( \frac{C_1L}{2C_2} = \frac{480 \times 100}{2 \times 269,000} \text{ ft.} = 0.0892 \text{ ft.} = 1.07 \text{ in.} \) (within errors of reading from a curve). With a six-story office building 100 ft. long on the same site, \( C_1 \) would scarcely be more than 24 for raft beams spaced at 20 ft. centres, making \( C_1L = 2400 \) and the deflections of and moments in the raft beams (assuming that both buildings stood on identical rafts) would be only one twentieth of those we have just calculated. With \( C_1 = 24 \) and \( C_2 = 269,000 \) no beam would be required to keep the overall deflection below \( \frac{1}{4} \) in. If a beam 6 ft. deep (equal to \( 0.06L \)) were provided to resist local and "excess-load" moments this uneven distribution of live load would produce an additional moment at mid-point of 

\[ -0.0027C_1L^3 \text{ (see Fig. 139) = } -0.0027 \times 24 \times 100^3 = -64,800 \text{ lb.-ft.} \]

(tension at underside).

We have so far discussed moments and deflections caused by uneven distribution of live load. If the general pattern of (dead + full live) loading is fundamentally non-uniform the process of spreading the excess load from the heavily
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loaded to the more lightly loaded areas will cause heavy moments and appreciable deflection of the raft beams. This is not generally a source of danger. The existence of heavy moments in the beams is obvious and cannot be overlooked. The beams needed to resist these large moments and shear are so deep that the deflections are small.

If we take a theoretical case as in Fig. 150 of a building set back as shown, the loading on the raft consists of a uniform load due to the first story plus an approximately triangular load due to the upper stories. If this triangular load varies from zero to 1100 lb. per square foot with raft beams at 20-ft. centres, \( C_1 = \frac{1100 \times 20}{50} = 440 \).

If the long-term settlement of the ground is 1 ft. per 1 ton per square foot \( C_2 = 1 \times 2240 \times 20 = 44,800 \) lb. per foot. Assuming a beam 10 ft. deep \( (= 0.10L) \), \( M \) at mid-point (Fig. 139) = \(-0.0006C_1L^3 = -0.0006 \times 440 \times 100^3 = -4,220,000 \) lb.-ft. \( \Delta \) from Fig. 149 is \( 0.00000075C_1L = 0.00000075 \times 440 \times 100 \text{ ft.} = 0.033 \text{ ft.} = 0.39 \text{ in.} \). Due to this deflection, the intensity of pressure on the ground under the centre of the raft would be more than that under the edges by (theoretically) \( 0.033 \times 2240 = 74 \) lb. per square foot only. With this depth of raft beam we are very close to the condition of an infinitely stiff beam which would give a uniform bearing pressure throughout as shown in Fig. 146.

Checking on these conditions, \( M = -0.0004C_1L^3 = -4,580,000 \) lb.-ft. and \( \Delta = 0.0009 \frac{C_1L^5}{EI} \). Since \( I = \frac{3.33 \times 10^3}{12} \text{ ft.}^4 \), \( \Delta = 0.0334 \text{ ft.} = 0.4 \text{ in.} \).

Raft under Flexible Superstructure on Non-Uniform Ground

Small local soft patches may be dug out and replaced with mass concrete but major variations across the site may render construction impracticable for

![Diagram of Cellular Raft](image)

large structures. Small rafts on patchy ground may be designed on the assumption that they may rest either on the outside quarters or on the middle half of their length and width. A raft carrying a small bunker on a heap of fine flue dust and another carrying a small turbo-generator in an area subject to salt mining subsidence were both designed on these assumptions. The latter could, if necessary, be jacked up at one end and re-levelled.

If we have a building as in Fig. 151 straddling a geological fault we could vary the intensity of loading artificially so that the estimated settlements were the same.
on both sides of the fault. Provided that these settlements developed exactly as estimated and exactly at the same rate we could have one continuous raft and one continuous superstructure. If the difference between the two sides were small it might be attempted but if one side were soft clay with settlement developing slowly and the other side sand with immediate settlement it would not work and separate buildings and rafts would be necessary. Again if the building sits across an old watercourse or over a swallow-hole in chalk country we could have the conditions in Fig. 152. If there were enough existing buildings close by to
give an accurate picture of local settlement it might be successful. The construction necessary to carry the whole weight of the central portion across to the harder parts of the site is only practicable in terms of everyday construction for a relatively small building.

Design Details of Rafts

To cope with the heavy shearing forces, loads should be taken back to the columns as directly as possible. Framing with thin slabs, tertiary beams and secondary beams is usually unworkable, a square panel as in Fig. 153 being an ideal arrangement because there are four main beam sections in shear to share the load from the column.

If any one panel is fully loaded the four adjoining panels must all be heavily, if not quite fully, loaded. The proper moment coefficient to be adopted in such a case has long been a matter for argument. The total positive moment across one diagonal, plus the total negative moment under one beam irrespective of sign must add up to $\frac{WL}{24}$. Assuming the moments are uniformly distributed along the sections and assuming the unit negative moment is twice the unit positive moment we get $+\frac{W}{72}$ and $-\frac{W}{36}$. Grashof-and-Rankine method would give $+\frac{W}{48}$ and $-\frac{W}{24}$. The old French regulations $+\frac{W}{72}$ and $-\frac{W}{36}$. Moments as low as $+\frac{W}{90}$ have been used. The author has for years used $+\frac{W}{36}$ and $-\frac{W}{36}$. The B.S. Code No. 114 now gives values of $+\frac{W}{40}$ and $-\frac{W}{30}$ but these are probably meant for a floor where adjoining panels may be unloaded and where the beams
have neither the breadth nor the stiffness of the normal raft beam. In arranging the steel in the slab remember that general sagging deflection of the raft as a whole may cause unexpected tension in all the slab steel. Reasonable laps and end hooks are required everywhere.

Generally the outside columns carry much more than 50 per cent. of the load on the internal columns and half a panel of raft will not support them. A heavy cantilevered toe as in Fig. 154 is usually needed. This fixes the edge of the exterior floor panels.

The difficulty with the reinforcement in the beams comes generally from the need to cover local moments, "excess-load" moments and, possibly, overall moments. A usual basic section is given in Fig. 155. Since uneven live loading might cause shearing of either kind at any point a continuous stirrup system is
called for. With large rafts these may be $\frac{3}{8}$-in. or $\frac{1}{4}$-in. diameter links and this raises several points concerning bond and anchorage. There must clearly be some relation between the overall height and bar diameter of a link. The author would say that for

- $\frac{1}{4}$-in. bar, link should be not less than 4 ft. overall
- $\frac{3}{8}$-in. bar, 5 ft.
- $\frac{1}{2}$-in. bar, 6 ft.

The overall size of links of $\frac{3}{8}$-in. diameter may normally be kept to a bending tolerance of $+0$ to $-\frac{1}{4}$ in. but with larger sizes, employing the usual methods of bar bending variations of $+0$ to $-\frac{3}{8}$ in. may be expected. A $\frac{1}{2}$-in. bar is normally bent to an inside radius of two diameters and will only fit tightly around a 3-in. diameter rod. The actual "contact" between a link and a main rod under normal bending practice could be as shown in Fig. 156. The situation may be improved by bending to a steel template with smaller mandrels and by sorting out the links after bending, on sizing templates and using all big ones or all small ones in any one beam. All these requirements should be considered and covered by a clause in the specification and items in the bill of quantities. Minor variations in link height may be overcome by leaning the larger ones over at an angle to the vertical but with links 7 ft. high swinging the top of the link over 3 in. horizontally decreases the vertical height by only about $\frac{1}{2}$ in. In some few cases small steel wedges have been provided between the main bars and the links and spot welded in position, an excellent but expensive idea. The links must rely partly for their anchorage on the compaction of the concrete around and between them and the main bars. The main bars should be as large as is practicable and should be welded and run continuously across the raft. Four bars $\frac{1}{2}$ in. diameter have an area of 7 sq. in. and in a beam 6 ft. deep would supply a bending strength of $7 \times 8 \times 5$ or 280 ton-ft. The combined moments in such a beam are unlikely to fall below this at any section. With modern dry concrete there is real danger of air pockets occurring between the links and main bars. Pushing the nose of a vibrator against the main bar will cause vibrations to run back a long way and
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has been known to shake bars loose from concrete already set. A workable
mixture placed by hand is the best solution in a difficult case however unfashion-
able it may be.

Real rafts may, in theory, be built on any type of soil but the geological con-
ditions that make a raft necessary are practically confined to sites whose safe
bearing capacity does not exceed 1 ton per square foot. Where the ground will
safely carry more than 1 ton per square foot at the surface, deeper foundations
are almost certain to be practicable and probably more economical for a building
that would need a raft at a shallow depth.

With spacings of 20 ft. by 20 ft., the column load will not then normally
exceed about 300 tons, the shear on each beam section not exceeding 75 tons

or 168,000 lb. If the arrangement in Fig. 155 shows \( \frac{3}{4} \)-in. links spaced in pairs at
1-ft. centres in a 5-ft. beam; they will resist \( \frac{4 \times 0.3 \times 18,000 \times 50}{12} \) or 90,000 lb.
(40 tons). If they were \( \frac{1}{2} \)-in. diameter in a beam 7 ft. deep they would resist a
shearing force of 80 tons.

The beams will generally require bent-up bars as shown in Fig. 157 to cover
the variations in total moment and total shear due to local moments and local
shearing. The amount of load from each mesh panel of slab to be carried by each
beam may be taken as shown by the hatching in Fig. 158, the panel being divided
by drawing lines at 45 deg. from each corner, but this amount may be taken as
uniformly distributed along the beams in this type of raft. In case some authority
asks for the beams to be calculated for triangular loading the calculated moments
in a continuous beam of uniform section loaded as shown in Fig. 159 are interior
span +0.0626WL; interior support −0.104WL; end span +0.105WL; next-to-end
support −0.132WL.
Local Bending Moments on Beams of Flexible Raft

The student's first encounter with continuous beams is based on a conception of frictionless and incompressible supports. The impact of this early teaching often makes it difficult to realise that a raft may well sink anything up to a foot and only in the smallest or simplest cases is its sinking absolutely uniform throughout the whole raft.

If we have a continuous beam ABCD of uniform section as shown in Fig. 160 carrying a uniform load and carried on frictionless and incompressible supports, the reactions on the supports are \(0.4W\) and \(1.1W\) and the moments at points B and C are \(0.10WL\). If we now take a continuous raft beam abcd as in Fig. 161 carrying a flexible superstructure consisting entirely of simply-supported beams the column loads will be \(0.5W\) and \(1.0W\). If the upward pressure of the ground is uniform then the moments at points b and c must be zero and the moments half way between these points must be \(0.125WL\). If we cut the raft beam at b and c into three separate parts, points a, b, c and d will remain all at the same level but if the beam is continuous it will, in addition to deflecting in each of the three spans, generally deflect upwards until it develops a general moment of \(0.10WL\) (tension on top) at points b and c to cancel the moment of \(0.10WL\) (tension on bottom) that would develop if the column loads were actually \(0.4W\) and \(1.1W\) instead of \(0.5W\) and \(1.0W\). But if a raft beam bends so that its ends sink below the mid-point then we must assume that the ground pressure does not remain uniform but increases at the ends and decreases in the centre. This brings the beam back towards a straight line and brings the moments towards those in Fig. 160. With the depth of construction usually employed it is a fair assumption to calculate the local moments as equal to those in a continuous beam on incompressible supports, particularly if we have made some allowance for irregular partial live loading and added a little to cover minor inequalities in the ground.
Effect of Flexure of Raft on the Superstructure

If the superstructure consists entirely of simply-supported beams, relative vertical movement of the columns is not serious. A steel-framed building using material to B.S. No. 15 (elongation 20 per cent.) is excellent. Any statically-indeterminate frame which relies on exact positioning of the points of support is dangerous on a real raft and any material devoid of elastic or plastic yield is unsuitable. The effect of column settlement on continuous beams and on frames is discussed in the author's book "Displacement Method of Frame Analysis".

Floor-slab Rafts

Reinforced concrete floors which lie directly on the ground, or on a layer of hardcore, are now often called rafts. As they have to carry traffic, any beams required to spread small concentrated loads, such as stanchions for single-story steel sheds, must be placed below the slab as shown in Fig. 162. With a very heavily loaded garage floor provision of beams of this type at intervals in both directions, say 12 ft. centres, with a corresponding beam all round the outside edge, adds greatly to its strength. These beams seem to have some sort of tying-in effect remotely similar to the hoops in Fig. 82 and to some extent they increase the effective depth of the slab to that of the beams.

Pavement Rafts

Any concrete slab resting on the ground is now also referred to as a raft. In general far too little attention is given to the drainage and strength of the foundation on which these floors lie. The Romans were prepared to make their road beds 3 ft. 6 in. thick and it is certain that these roads could have carried the pedestrian traffic for which they were designed for many centuries. Compare this with a modern motor road where one section failed after only a few weeks. The first thing necessary, is to deal adequately with the ground and surface water, raising the general level of the ground if necessary by chalk filling or hardcore. On ground not capable of supporting more than $\frac{1}{4}$ ton per square foot a layer of consolidated hardcore or chalk filling 1 ft. thick is advisable, or 2 ft. for very heavy traffic. The weakest part of a pavement is the edge and this should either be thickened out or supported as shown in Fig. 163. The author would not rely on dowel bars for a joint. On hard well-drained ground such as solid chalk lying near the surface, no reinforcement is necessary and 6 in. of no-fines concrete topped with a $\frac{3}{4}$ in. finish worked well in may be used. The question of a working surface is generally difficult. Provided the sand and shingle do not contain any soft particles the surface will carry light or medium traffic if the floor is properly laid and cured. This is easy for a road or outside pavement, but the
ground floor slab of a multi-story building must often be put down at once or the ground may be churned into a quagmire. It is virtually impossible to prevent heavy traffic ruining the surface as soon as it hardens. There can be no real joint between a well-made reinforced concrete slab and a granolithic finish laid several weeks later. Probably the best solution is to let the contractors knock the surface about as they please and finally put on a granolithic finish 2-in. thick, laid in small panels and reinforced with a stout mesh. The thickness of the reinforced concrete slab and the amount of reinforcement depend more on the soundness of the foundation than on the traffic to be carried. A 6-in. slab with a mesh weighing 4 to 6 lb. per square yard is enough on a solid dry base. An 8-in. slab with 1/4-in. diameter rods at 6-in. centres both ways may be required on a soft or wet base. Some idea of the effect of a yielding base is shown in Fig. 164.

Imagine a strip of 6-in. slab 1 ft. wide and 10 ft. long carrying a knife-edge load of 2700 lb. in the centre and resting on a very soft base so that the pressure is spread uniformly.

\[ EI = 3,000,000 \times 144 \times 1 \cdot 0 \times \frac{0 \cdot 5^3}{12} = 4,500,000 \text{ ft. units.} \]

\[ \Delta = \frac{Wl^3}{8EI} = \frac{1350 \times 5^3}{8 \times 4,500,000} \text{ ft.} = 0 \cdot 0047 \text{ ft.} = 0 \cdot 0563 \text{ in.} \]

The moment is \(-1350 \times 2 \cdot 5 = -3380 \text{ lb.-ft.}\). Referring to Fig. 132 (page 130), this corresponds to \(-0 \cdot 125WL\) which occurs when \(L \times \sqrt[6]{\frac{C_2}{EI}} = 0\). To reduce this moment to \(-0 \cdot 0625WL\) requires a value of \(L \times \sqrt[6]{\frac{C_2}{EI}} = 6\). With \(L = 10\) and \(EI = 4,500,000\), this makes, \(C_2 = 574,000 \text{ lb.} = 256 \text{ tons per sq. ft. of compression.}\) If we used a 6-in. slab reinforced with 1/4-in. bars at 6-in. centres this would have a safe moment of resistance of about 33,000 lb.-in. = 2750 lb.-ft.

With \(W = 2700 \text{ lb. and } L = 10 \text{ ft.}, -2750 = -0 \cdot 102WL\). From Fig. 132 this corresponds to \(L \times \sqrt[6]{\frac{C_2}{EI}} = 3 \cdot 3\), or \(C_2 = 52,000 \text{ lb.} = 23 \cdot 3 \text{ tons per square foot per foot of sinking.}\)

With a slab of indefinite length \(L\) carrying a knife-edge load at its centre as in Fig. 132, if we assume that the spreading effect ends when \(y = 0\) (\(y\) being measured.
as in Fig. 133) and taking the safe tensile stress in plain concrete as 150 lb. per square inch, the largest knife-edge load per ft. width of slab that a plain slab will safely carry is theoretically, as shown below.

<table>
<thead>
<tr>
<th>Tons per square foot per foot</th>
<th>6-in. slab</th>
<th>9-in. slab</th>
<th>12-in. slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_2 = 10$</td>
<td>680 lb.</td>
<td>1130 lb.</td>
<td>1620 lb.</td>
</tr>
<tr>
<td>40</td>
<td>960 &quot;</td>
<td>1600 &quot;</td>
<td>2200 &quot;</td>
</tr>
<tr>
<td>80</td>
<td>1140 &quot;</td>
<td>1900 &quot;</td>
<td>2720 &quot;</td>
</tr>
</tbody>
</table>

All the foregoing figures are theoretical but they emphasise the need for a firm base for any ground floor or pavement to carry heavy rolling loads. Before beginning a major paving project, test areas should be prepared by putting down patches of hardcore and rolling and these patches should be test-loaded by loading cast-in-situ concrete slabs 3 ft. by 3 ft. in plan to establish a value for $C_2$. With a thin pavement carrying a few isolated heavy wheel loads the value of $C_2$ may depend more on the type and thickness of the hardcore layer than on the physical properties of the soil beneath.

**Example of Raft Construction**

Several large rafts were constructed in Shanghai in the years 1920–1921. Unfortunately most of the details have been lost due to war damage. One of them was over 200 ft. long and over 150 ft. wide. The ground consisted of silt to a depth of 400 ft. with a safe bearing capacity of $\frac{1}{2}$ ton per square foot. This could be increased to 1 ton per square foot by driving consolidation piles (in this case the trunks of fir trees) at 5-ft. centres. A total settlement of 12 in. was expected and allowed for. The panels were rectangular and the slab was splayed as shown in Fig. 165 so that the effective depth of the slab at the centre-line of the beam was twice that at mid-span.
Moments of $\frac{w_b b^2}{24}$ and $-\frac{w_b b^2}{12}$ in the short direction and $\frac{w_e b^2}{24}$ and $-\frac{w_e b^2}{12}$ in the long direction were taken, where $w_b$ and $w_e$ are the fractions of the total load according to Grashof and Rankine. The beam steel was all assembled and the beam shutters erected and braced before the slab was concreted, the shutters and screeds being supported on precast blocks of concrete as shown in Fig. 165. The superstructure was to be a steel-framed office building with ruling spans of 18 ft. to 25 ft. The raft beams were generally 7 ft. deep overall except under the main entrance hall where the column spacing was much larger and here they were increased to 9 ft. After the excess loads had been spread or taken up by consolidation piles the panels under the open areas were very much underloaded and these were filled with sand to equal the dead loading intensity elsewhere. These rafts have apparently given satisfaction. Plate VII (facing page 123) is reproduced from one of the few surviving photographs.

**Example of Raft for Gasholder Tank**

A foundation raft for a gasholder tank is considered in the following. The geology is shown in Fig. 166. The surface is alluvium overlying the Woolwich and Reading Beds. Details of five boreholes, four round the periphery of the

![Diagram of geology](image)

**Fig. 166.** *(Based on a Crown Copyright Geological Survey Map by permission of the Controller of H.M.S.O.)*

tank and one in the centre, are given in Fig. 167 (opposite). It was decided to fix the level of the edge of the raft at 13·50 ft. The general arrangement is shown in Fig. 168 (page 158). As the construction consists of 14-in. screed, 9-in. structural slab and 3-in. mass concrete, and the raft rises 6 in. towards the centre-point, the shallowest part of the foundation is at about 12·90 ft. A trial pit showed the gravel and sand to be a fairly compact bed of ballast capable of carrying 3 to 4 tons per square foot. A small watercourse ran through one edge of the site and at this point the top of the ballast dipped to about level 8·00 ft. (Borehole No. 3
SECTION B - B

Fig. 168.
(For Section A - A, see Fig. 169, page 160.)
in Fig. 167.) As the underside of the ring beam foundation is at level 11.42 ft. (see Section B-B in Fig. 168) it was expected that a maximum of about 3 ft. 6 in. of soft material would have to be removed from under this edge of the foundation and replaced by mass concrete. The watercourse was diverted in culvert. The dip in the gravel-and-sand stratum proved more extensive than the boreholes indicated and the amount of soft material was double that expected.

The main raft carries a uniform load of 0.92 tons per square foot from the tank which, with its own weight, makes a total of 1 ton per square foot. A further load of 104 tons at the centre point is spread over an area of 18 ft. x 18 ft. increasing the loading at this point to 1.4 tons per square foot. The ring-beam also carries the wall of the steel tank which is about 0.6 tons per foot run, and 48 rest-blocks each carrying 22 tons spaced at about 12-ft. centres. These increase the loading under the ring beam to 1.4 tons per square foot. There proved to be a fair flow of ground water through the ballast and the pits were constructed inside steel sheet-piling. Pits of this type must be constructed without disturbing the ground under the main raft and the specification must insist on this. If timbering, sheeting or sheet-piling is withdrawn any voids must be filled with mass concrete and any loose ground consolidated. (Items in the list of quantities should cover this.) A longitudinal section of one pit is shown in Fig. 169 which corresponds to Section A-A in Fig. 168. A cross-section is shown in Fig. 170.

Gross area of section = 8 ft. 8 in. x 10 ft. 6 in. = 91 sq. ft.
Deduct area of opening = 6 ft. 8 in. x 6 ft. 8 in. = 44.5

Weight = 46.5 x 150 = 7000 lb. per foot run of pit.
Load on roof 8.67 x 2050 = 17,800 lb.
... from rest-block per foot, say = 5000

22,800

Add self-weight 7000

Total load on foundation per foot = 29,800

Gross load on ground = \frac{29,800}{8.67} = 3440 lb. per square foot

Self-weight = 350

Net loading = 3090

The pressure on the walls of the pit is uncertain. If the steel sheet-piles keep back the ground pressure it may be nil. If not it may be about one third of the loading under the main raft, say 750 lb. per square foot. Since there is no side-way of the corners of this frame it can be solved by the Theorem of Three Moments.

\[ I \text{ of roof section} = 1.0 \times \frac{1.5^3}{12} = 0.282. \]

\[ I \text{ of wall section} = 1.0 \times \frac{1.0^3}{12} = 0.0833. \]

\[ I \text{ of floor section} = 1.0 \times \frac{2.33^3}{12} = 1.06. \]
SECTION A-A

Fig. 169. (See Fig. 168, page 158.)

Fig. 170.
The corresponding values of $\frac{L}{I}$ are $\frac{7.67}{0.282}$, $\frac{8.6}{0.0833}$ and $\frac{7.67}{1.06}$, which are 27.2, 103 and 7.23 respectively.

The fixed-ended moments are

- **Roof.** $\pm \left[ (2050+225) \times \frac{7.67^2}{12} + \frac{5000 \times 7.67}{8} \right] = \pm 15,950$ lb.-ft.
- **Floor.** $\pm 3090 \times \frac{7.67^2}{12} = \pm 15,150$ lb.-ft.
- **Walls.** $\pm \frac{w \times 8.6^2}{12}$, where $w$ may range from 0 to 750.

Therefore

$$27.2M_A + 2M_B(27.2 + 103) + 103M_C = (3 \times 27.2 \times 15,950) + (3 \times 103 \times 6.17w)$$

and

$$103M_B + 2M_C(103 + 7.23) + 7.23M_D = (3 \times 103 \times 6.17w) + (3 \times 7.23 \times 15,150).$$

Also by symmetry $M_A = M_B$, and $M_C = M_D$. Solving the foregoing equations $M_A = 4770 + 4.33w$ and $M_C = -710 + 6.42w$. The theoretical moments on the left-hand side of Fig. 170 are for $w = 0$, and on the right-hand side for $w = 750$.

Considering the thickness of the slabs, it would seem reasonable to design the roof and the floor for moments of 20,000 lb.-ft. and the walls for 10,000 lb.-ft.

- $A_{st}$ in roof, say, $\frac{20,000 \times 12}{18,000 \times 0.86 \times 16} = 0.97$ sq. in.;
  - $\frac{1}{2}$-in. bars at 6-in. centres = 1.065 sq. in.
- $A_{st}$ in floors, say $\frac{20,000 \times 12}{18,000 \times 0.9 \times 27} = 0.55$ sq. in.;
  - $\frac{1}{3}$-in. bars at 10-in. centres = 0.532 sq. in.
- $A_{st}$ in walls, say $\frac{10,000 \times 12}{18,000 \times 0.86 \times 10} = 0.775$ sq. in.;
  - the walls have $\frac{1}{4}$-in. bars every 5 in. outside (1.065 sq. in.)
  - and $\frac{3}{4}$-in. bars every 10 in. inside (0.532 sq. in.).

The reinforcement is shown in Fig. 171. The ring-beam carries loads of 22 tons at about 12-ft. centres. Bending moment, say $22 \text{ tons} \times \frac{12}{12} = 22 \text{ tons-ft.} = 590,000 \text{ lb.-in.}$

$$A_{st} = \frac{590,000}{18,000 \times 0.86 \times 16} = 2.39 \text{ sq. in.}; \text{ nine } \frac{1}{8}\text{-in. bars} = 2.76 \text{ sq. in.}$$

A section through the beam is shown in Fig. 168 and the reinforcement is shown in Fig. 172.

**Example of Tank Raft on Piles**

The circular foundation shown in Figs. 173, 174 and 175 would today be described as a raft although it has none of the characteristics of a true raft. The site lies fairly close to that described in Fig. 188 (page 177) and it was known that
precast reinforced concrete piles 35 ft. to 40 ft. long could be driven to carry 40 tons each with an ample factor of safety.

**Loading.**—The foundation carries a steel tank 60-ft. diameter, weighing 2850 tons.

\[
\frac{2850 \times 2240}{0.785 \times 60^2} = 2250 \text{ lb. per square foot}
\]

Self-weight, say \[
168
\]

\[
\frac{2418 \text{ lb. per square foot}}{168} = 1.45 \text{ tons per square foot}
\]

In addition to the general loading there is an extra 15 cwt. per linear foot around the periphery of the tank.

At 40 tons per pile each pile will carry 37 sq. ft. of slab or an area 6 ft. square.

**Design.**—The natural reaction is to space the piles out on a square grid and adopt a flat-slab design. There are several reasons why this is very seldom done.
Firstly the spans are so small that grip lengths occupy about half the span and nothing is saved by stopping the bars for each span. If the set of the piles is doubtful and they have to be placed nearer together or if they have to be moved to avoid small obstructions, the pattern of support changes. A flat-slab floor really depends on the column cap and the stiffness of the columns above and below it. With no cap and a long pile standing in very soft ground no stiffness is forthcoming and much higher moment coefficients have to be faced. In this particular case this argument applies only to the external panels since the loading must always be uniform, but the moments in these external panels might be $-\frac{WL}{16}$ and $+\frac{WL}{20}$ in strip A, and $-\frac{WL}{50}$ and $+\frac{WL}{50}$ in strip B. The shearing strength of slabs of this type has generally been checked by the resistance to punching shear.
SECTION A-A

For position of Section A-A see Fig. 175.
At 150 lb. per square inch, the minimum \( d = \frac{40 \times 2240}{150 \times 4 \times 15} = 10.7 \) in. It always looks wrong to make the slab thinner than the diameter of the pile and this raft is made 14-in. thick. Check for resistance to beam-shear outside the 45-deg. line drawn to the tensile steel as in Fig. 178. If \( d_1 = 12 \) in., the lever-arm is about 10-5 in. At 100 lb. per square inch, the beam-shear strength is \( 4 \times 38 \times 10.5 \times 100 = 160,000 \) lb. = 71.5 tons.

The bending strength is checked by imagining that the slab first acts as a 14-in. slab spanning 6 ft. and then functions as a beam spanning 6 ft. carrying this slab. It is equivalent to the arrangement shown in Fig. 179 except that the slab thickness \( t \) is increased until it equals the beam depth \( d \). The reinforcement in both the beam and the slab consists of continuous straight bars in as long lengths as practicable, running right across the raft and only lapped where necessary.
QUARTER PLAN
SHOWING BOTTOM STEEL AND LINKS ONLY
NOTE: 162 No. links marked (16) are spaced round the ring beam as shown mostly in pairs. They are arranged as shown to miss the ends of the bars marked (19) and (20).

SECTION A -- A

Fig. 176.
QUARTER PLAN
SHOWING TOP STEEL ONLY

SECTION B - B

FIG. 177.
The loading on the slab is \(2418\) lb. per square foot and is the same on all spans.

\[
M = -2418 \times 6^2 \times \frac{12}{12} = -87,000 \text{ lb.-in.}
\]

and

\[
+2418 \times 6^2 \times \frac{12}{24} = +43,500 \text{ lb.-in.}
\]

\[
A_{st} = \frac{87,000}{18,000 \times 0.86 \times 12.5 \text{ in.}} = 0.45 \text{ sq. in. (at top)},
\]

and

\[
\frac{43,500}{18,000 \times 0.86 \times 11.5 \text{ in.}} = 0.245 \text{ sq. in. (at bottom)}.
\]

The steel provided is \(\frac{3}{4}\)-in. bars at \(1\)-ft. centres (\(0.44 \text{ sq. in.}\)) at the top and \(\frac{1}{2}\)-in. bars at \(1\)-ft. centres (\(0.3 \text{ sq. in.}\)) at the bottom.

Loading on the "beam" is \(2418 \times 6 = 14,500\) lb. per foot.

\[
M = -14,500 \times 6^2 \times \frac{12}{12} = -522,000 \text{ lb.-in.}
\]

and

\[
+14,500 \times 6^2 \times \frac{12}{24} = +261,000 \text{ lb.-in.}
\]

If we accept the ruling that a "beam" of this type may be assumed to be three times as wide as the column then we have a rectangular beam 42 in. wide.

\[
\frac{M}{bd_1^2} = \frac{522,000}{42 \times 11.5^2} = 94 \text{ only}
\]

\[
A_{st} = \frac{522,000}{18,000 \times 0.86 \times 11.5} = 2.93 \text{ sq. in. (at the top)},
\]

and

\[
\frac{261,000}{18,000 \times 0.86 \times 12.5} = 1.35 \text{ sq. in. (at the bottom)}.
\]

The steel provided is two \(1\)-in. bars plus two \(\frac{3}{4}\)-in. bars (\(2.77 \text{ sq. in.}\)) at the top, and four \(\frac{1}{2}\)-in. bars (\(1.76 \text{ sq. in.}\)) at the bottom.

The view in the lower left-hand corner of Fig. 174 shows a section through the "beam". In the years preceding 1914 many slabs of this type with small span-depth ratios were reinforced with bottom steel only. No engineer would now accept such construction. The author has instinctively cut the top steel and increased the bottom steel. If any pile may wander 3 in. out of position this
could increase the span from 6 ft. to 6 ft. 6 in. and thus increase the positive moment by 20 per cent. The loading on the outside ring-beam varies, and is, say, $3.5 \times 2418 = 8450$ lb. per foot.

$$M = -8450 \times 6^2 \times \frac{12}{12} = -305,000 \text{ lb.-in.}$$

$$A_{st} = \frac{305,000}{18,000 \times 0.86 \times 13.5} = 1.48 \text{ sq. in. (at top).}$$

The steel provided is four $\frac{1}{4}$-in. bars (1.76 sq. in.) at the top and four $\frac{3}{8}$-in. bars (1.23 sq. in.) at the bottom.

The main reason for making the ring beam deeper than the rest of the slab is to get the reinforcement below all the other steel. Drawings of the reinforcement become crowded and confusing if attempts are made to show everything on one plan. *Fig. 176* shows the bottom steel and *Fig. 177* shows the top steel. The view in the lower right-hand corner of *Fig. 174* is a spacing diagram showing the relative levels of the four layers of bars in the main slab and the two layers in the ring beam. The links are provided to support the top bars and keep the steel generally in place.

One surprising thing about these circular foundations is the small average load per pile since the outside piles are not fully loaded. This foundation carries

\[
\begin{align*}
144 \text{ cu. yd. concrete} & = 260 \text{ tons} \\
\text{Tank} & = 2850 \text{ ,,} \\
\text{Periphery} 3.14 \times 60 \times 0.75 & = 141 \text{ ,,} \\
& = 3251 \text{ tons}
\end{align*}
\]

There are ninety-seven piles or only an average load of 33.7 tons per pile. The raft has 175 cwt. of main bars and 14 cwt. of links. Decreasing the thickness to 12 in. would save 20 cu. yd. of concrete but would need about 35 cwt. more steel to give the same bending strength. Increasing the thickness to 16 in. would need 20 cu. yd. more concrete but might save about 30 cwt. of steel. We have only $\frac{1}{4}$-in. bars at 12-in. centres in a slab 14 in. thick or only 0.21 per cent. of the base area. To give the same bending strength in a 16-in. slab this would reduce to 0.155 per cent. only, which looks too little.
CHAPTER XII
DRIVEN-PILE FOUNDATIONS

A surprising number of academic and professional experts have spent a great deal of time and effort in attempting to produce an accurate pile-driving formula. Assuming that such a formula could be produced and further assuming that it could be reduced to some practicable form it would still only take us one third of the way towards our goal. A perfect formula could only tell us the exact equivalent static resistance encountered by the pile at the instant of driving. We still should not know to what extent this resistance increases or decreases with time, due to natural or human causes, nor what is the relation between the safe load on a closely spaced group of piles and the safe load on a single pile—and only a small minority of unimportant loads are carried on single piles. The Foundation Code recommends the use of the Hiley driving formula which is perhaps as good as any although the author never uses it. If only a fraction of the effort spent to produce a formula had been employed on designing and building bigger and better pile-driving plant we should now be much better off. The progress made in this country between 1914 and 1959 in heavy piling equipment is depressingly small. The weight of the monkey in general use was, in 1914, 20 to 30 cwt. and is now 2 to 3 tons. Monkeys or steam hammers of 6 tons are rare. Pile frames were then mounted on rails and pinched along with bars giving free movement in one direction only. Most frames are still of this type and slewing frames mounted on undercarriages or tractors are still uncommon.

If a pile is meant to carry a safe load of 50 tons the best way to drive is to apply a load of 125 tons. Even if the rapid and economical handling of 125 tons of kentledge is beyond our mechanical experts a non-automatic monkey of 3 tons, which is less than 2\% per cent. of the ideal load, seems a little outdated.

The depth of site exploration required is discussed in Chapter II and shown in Fig. 10 (page 13). The author has expressed his views on everyday reinforced concrete in his "Reinforced Concrete Design". A review of prestressed concrete piles has been compiled.* It has been claimed that prestressing allows the use of smaller piles. This is exactly what we do not want. Prestressing, when further developed, may help long piles to stand up to handling and driving stresses better than normal reinforced concrete piles. Piles 60 ft. to 80 ft. long in jetty work, particularly raking piles, may be better if prestressed. This point is not yet certain but is certainly worth pursuing. Piles up to 40 ft. in length for land foundations can easily be made strong enough to stand up to any normal driving force without prestressing. One of the drawbacks to prestressed piles is the danger to the workmen when cutting off the pile-heads. One of the highly tensioned wires may fly and inflict quite a nasty injury. A discussion of the column strength of long piles of steel, timber, cast-iron and reinforced concrete is contained in Appendix B. It applies generally only to submerged sites.


170
Consolidation Piles

Certain silts can be consolidated and strengthened by driving small piles at fairly close intervals. An example is quoted in Chapter XI where timber piles driven at 5 ft. centres raised the safe bearing capacity of a silt from 2 to 1 ton per square foot. The success of the method probably depends on the percentage of silica in the silt and it would probably fail in very soft plastic clay. In areas where the method has not previously been exploited it should be checked by loading tests after keeping a very careful record of ground levels whilst driving the test piles. Foundations on beds of silt deeper than 100 ft. are rare in this country and cases where consolidation piles could be used are therefore few.

Risen Piles

Figure 180 shows the actual rise in ground level caused by driving a group of piles into clay. The rise was immediate and equal in volume to the volume of piles driven. On other sites rises up to 1 ft. 6 in. were observed and concrete roads and river walls drifted away laterally from 24 in. to 7 in. Setting-out pegs drifted laterally 5 in. The piles already driven rise with the ground. What happens then when a pile is raised 2 ft. after driving? If it is driven through plastic clay on to tight ballast this lifting could raise the point of the pile clear of the ballast layer. At the beginning of a contract all risen piles should be redriven until the engineer has a clear record of the amount of driving resistance lost due to rising. Redriving is easy with a modern frame mounted on an undercarriage but with old-fashioned plant it is a slow and unsatisfactory job. The central piles in each group should be driven first. This also is easy with modern plant. The engineer must make it clear in his specification and bill of quantities that the piles must be driven in a given order and that considerable redriving is anticipated.

Pile-caps

Pile-caps present the same problems as footings on good ground, namely heavy shearing and short grip-length. Examples of caps for two, three and six piles are shown in Fig. 181. These are for 14 in. x 14 in. piles at 3 ft. centres carrying 40 tons per pile. Caps for four and nine piles follow simply from those for two and six piles. If it is necessary to restrict the depth or width of the pile caps, reinforcement as shown in Fig. 182 may be used. Sometimes circular hoop steel near the base of the cap is used as shown in Fig. 185 (page 173) following the principles of the cone foundation in Figs. 82 and 83. Sometimes this idea is modified into a square “hoop” as shown in plan in Fig. 183 either welded or overlapped as shown. Sometimes “gridiron” steel as in Fig. 184 is useful but, unless carefully bent, gridirons tend to wind into a gauche quadrilateral and do not lie flat. On the other hand they are all in one piece. Generally an ample cross-section with not much steel offers the best solution.
2 - PILE CAP
5'-3" x 2'-3" x 2'-6" DEEP

3 - PILE CAP
5'-3" x 5'-3" x 2'-6" DEEP

6 - PILE CAP
8'-3" x 5'-3" x 3'-0" DEEP

Fig. 181.
Test Piles

It is absolutely essential that the engineer responsible for the foundation should be present or represented at the driving of the test piles. The author has sometimes been presented with the driving record of a reinforced concrete pile and been asked to say what he considers the safe load to be. Never will he express an opinion unless he has been present at and checked the whole of the driving. One of several incidents will show why. The author was responsible for the design of piled foundations on a site where test piles had already been driven by another contractor. The driving records were neat, clear and convincing. On inspecting the site, the heads of these piles were neat, standing a comfortable distance above ground level and looking as if they had just been removed from the moulds. When the author has finished with a test pile the head certainly does not look like that. Even if it is sound when the necessary set is reached the helmet and packing are removed and driving continued on the bare pile head (producing sets which frighten those who have not seen this performance before) as long as it will stand up. The first thing to do was to redrive the test piles which shifted at the first blow. No satisfactory set was obtained anywhere on the site near the levels at which the test piles had been stopped.

Positioning Piles

If the ground is known to be free from obstructions and the test piles all show no abnormal tendency to wander out of position, say within 2 in., and all easily reach the specified set, the engineer may concentrate on designing the most economical substructure. If the ground is patchy with many piles failing to reach a proper set or if there are obstructions too deep down to remove easily, the engineer should concentrate on producing a design for a substructure that will cope with these conditions.

If we have a floor to a large factory which has to carry 2 cwt. per square foot and is supported on piles carrying 40 tons each we can produce the flat-slab design in the upper part of Fig. 186.

\[
\begin{align*}
\text{Imposed load} & = 224 \text{ lb. per square foot} \\
\text{Self-weight, say} & = 110 \quad " \quad " \\
& \quad \frac{334}{"} \quad \text{say, } 0.15 \text{ ton.}
\end{align*}
\]
Maximum floor area per pile = \( \frac{400}{0.15} = 267 \) sq. ft.; say, 16 ft. 3 in. square.

Slab thickness = \( \sqrt{\frac{W}{40}} + 1\frac{1}{2} = \sqrt{\frac{40 \times 2240}{40}} + 1.5 = 9 \) in.

Thickness of drop = \( \sqrt{\frac{W}{28}} + 1\frac{1}{2} = 10.7 + 1.5, \) say, 12\( \frac{1}{2} \) in.

This is quite good provided we are certain of the positions and sets of the piles. If one pile fails to reach a set we must drive another alongside it. Assuming each of these two piles is good enough for 30 tons we are wasting 20 tons of our carrying capacity. The new pile is off centre and upsets the symmetry of the floor arrangement. This may need extra reinforcement to put things right. If

![Diagram of floor arrangement](image)

the ground is patchy it is usually better to accept the situation at the outset and adopt the arrangement shown in the lower part of Fig. 186, which consists of a 9-in. slab spanning 10 ft. carried on beams 2 ft. deep. These beams have continuous reinforcement top and bottom sufficient for a maximum span of 14 ft. The first pile is pitched and driven. If it is good enough for 40 tons the next one along the beam is driven 14 ft. away. If this one only reaches a set worth 30 tons the next pile is pitched only \( \frac{30}{28} \times 14 = 10 \) ft. 6 in. away, the spacing being always proportional to the safe bearing capacity of the previous pile. This utilises the full safe bearing capacities of the piles and saves a lot of time which could be lost by dithering about on borderline cases. In the lower part of Fig. 186 the left-hand beam requires shuttering but has less concrete than the right-hand beam. Which is the cheaper of the two depends on site conditions. The right-hand arrangement has the advantage that slab and beam can be concreted in one operation. Many sites in the valleys in South Wales have variable ground conditions and this type of construction has been used there on several occasions.

**Example of Group of Piles**

Fig. 187 (opposite) shows one unit of a large foundation. The complete foundation measures 226 ft. by 121 ft. The outside consists of a slab-and-girder floor carried on piles (seen at the left-hand of the section in Fig. 189 on page 178).
Fig. 187.
The centre carries a number of heavy installations, arranged in a double row, which rest on reinforced concrete walls 5 ft. thick, 31 ft. 6 in. long and 10 ft. high. Each wall is carried on a group of 35 piles, the total load on each group being 1330 tons. The local geology is shown in Fig. 188 (opposite). This indicates alluvium overlying the Woolwich and Reading Beds with a fault about 400 yards away. A borehole on the site shows 10 ft. of made-up ground, 12 ft. 3 in. of peaty clay, 1 ft. of sandy clay, 10 ft. 3 in. of clean ballast then more clay. The top of this lower clay is soft but it gets harder gradually as the depth increases (see Fig. 189).

Many thousands of piles have been driven on nearby sites in lengths up to 40 ft. but none quite so closely grouped. Five test piles 14 in. x 14 in. x 45 ft. were driven, each encountering heavy resistance in the ballast. The driving record of one of these, showing the number of blows per foot of penetration is given in Fig. 189. It is clear that, although individual piles may be driven through the ballast, the first piles in a group would so consolidate and tighten this stratum that following piles would never get through. If driven piles are to be used in groups they must terminate in the ballast. There is no question about the ability of any single pile to carry safely 40 tons but there are two serious considerations that could jeopardise the collective strength of a group. Details of one loaded wall are shown in Fig. 189. The ground above the ballast is peaty clay and there was the chance that this might cause some rise in ground level similar to that shown in Fig. 180 and possibly some lateral displacement of the first piles to be driven. It was specified that the centre row in each group (see the plan in Fig. 187) had to be driven first and, if necessary, redriven after the two outside rows were completed. Upward movements of the centre piles were observed and measured, and on redriving it was confirmed that these centre piles had pulled up out of their bearing in the ballast during the driving of the outside rows. The driving called for three successive sets not exceeding 1 in. for 20 blows of a 2½-ton monkey falling 3 ft. and most piles finished with sets of 1/2 in. or 1/2 in. for 10 blows. On redriving, the set for the first 10 blows varied from 1/2 in., with no apparent rise of the piles, to 2½ in. maximum (one pile only) and averaged 1-20 in. On the average about 20 blows were necessary before the original hard set was again picked up. All this was foreseen and covered by items in the specification and bill of quantities. As the frame was mounted on an undercarriage the redriving was easy. A much more serious consideration was the stratum of comparatively soft clay immediately below the ballast. As the walls are spaced at 16 ft. 3 in. centres (see Fig. 187) the load per unit cannot spread further than this. Allowing a spread of 1 in 2 as in Fig. 189, the load of 1330 tons is spread over an area of 16 ft. 3 in. by 37 ft. 6 in. at the top of the clay.

This is a net loading intensity of \( \frac{1330}{16.25 \times 37.5} = 2.18 \) tons per square foot. The height of ground above the clay is about 35 ft., equal to about 2½ tons per square foot making a total gross loading intensity of about 4½ tons per square foot. The peaty-clay stratum immediately above the ballast was reached in a small cofferdam adjoining this foundation. At level -9.00 ft., the clay was of a type that would have a safe bearing capacity of about 1 ton per square foot for shallow foundations. Judging by the relative driving resistance encountered (Fig. 189) the clay immediately below the ballast was capable of supporting about twice
FIG. 183.
(Based on a Crown Copyright Geological Survey Map by permission of the Controller of H.M.S.O.)
as much. Clay squeezed out from below the foundations would have to follow some such path as mst marked in Fig. 189 and this tendency would be confined to a shallow layer as the stiffness increased appreciably below level -30·00 ft. If the ground were composed of Rankine's ideal soil it would need a net loading intensity of about 18 tons per square foot to cause shear-friction failure at this depth.

If we assume the type of failure shown in Fig. 4 (page 8) then B in Fig. 189 is $2 \times 37·5 = 75$ ft. and $h = 35$ ft. The net loading intensity at failure $= 6·28f\left[1 + \frac{35}{1·57 \times 75}\right] = 8·15f$. We have a net loading intensity of 2.18 tons per square foot corresponding to a value for $f$ of $\frac{2·18}{8·15} = 0·268$ tons per square foot $= 600$ lb. per square foot. With a factor of $2\frac{1}{4}$ this becomes 1500 lb. per square foot corresponding to a form of stiff clay. Point n in Fig. 4 would lie $(35 + 37·5)$ ft. below the ground surface, and the ground below level -30·00 ft. would certainly raise more than 1500 lb. per square foot although the ground above level -10·00 ft. might not average more than 500.

We are left with the problem of relative long-term consolidation settlement under the two portions of the foundation. As shown in Fig. 189, the two parts are separated by a drainage trench capable of taking up some small relative movement. Whether the clay, which lies 30 ft. below ground-water level and which already carries $2\frac{1}{4}$ tons per square foot, will show measurable consolidation remains to be seen. No visible movement has so far developed.
CHAPTER XIII
BORED-PILE FOUNDATIONS

Bored piles must be protected from necking caused by soft ground squeezing or falling in and seriously reducing the diameter. They must also be protected from loss of cement due to the action of moving ground water. These two necessities probably account for the tendency to use 17-in. diameter piles where formerly 15-in. and 12-in. were fairly common. Provided a reasonable percentage of the piles are adequately test-loaded they provide a reliable foundation. Where the piles go through and fetch up in plastic clay they may be better than driven piles as their use eliminates the ground heaving severely which may be caused by driving. (See Fig. 180, page 171.) To what extent large diameter drilled foundations will replace groups of bored piles remains to be seen. Mobile rigs that can travel over the site and put down small diameter short bored piles are now used for dealing with minor loads.

Example of Bored-pile Foundation

This example is a foundation to support a six-story steel-framed building with brick panel walls.

Site Exploration.—Local geological conditions are shown in Fig. 190 and these indicate alluvium overlying drift (probably ballast) and then a great depth of London Clay. The site is less than 100 yards from and slopes gently towards the River Thames. Standing on the site were the burnt-out remains of a warehouse possibly eighty to one hundred years old. This had heavy brick external walls and was about 40 ft. wide with two rows of cast iron columns running the length of the building. The columns terminated at ground floor level and were carried on brick walls running across the basement. Everything was carried on a thick raft of lime concrete, the top of this raft being about 8 ft. to 10 ft. below pavement level and about 15 ft. above mean water level in the river. The bottom courses of the old walls were reinforced with bands of hoop-iron. The old raft was said to be 7 ft. thick. No signs of settlement were visible in any part of the old walls. The author's first guess was that the old raft was founded on ballast overlying the London Clay and that the new building might well be carried on this old raft as the average total load per square foot of site area for the new and old buildings was about the same. To check this a borehole, shown in Fig. 191, was put down. There was nothing that would now be regarded as a possible foundation within 12 ft. of the bottom of the raft. The next guess was that the old raft was merely a capping and that the old building was carried on timber piles arranged under the bearing walls. This conjecture was strengthened by the provision of the cross walls in the basement and the hoop-iron reinforcement. If this were so these old piles might be reaching the end of their useful life. It was a practical certainty that the new building would require pits and trenches below the basement floor and these would carve away much of the strength of the old raft. It was certain that old buildings on adjoining
The load-time-settlement curves are given in Figs. 194 and 195. The load was applied in increments of 10 tons and reached 80 tons in 40 minutes. This load of 80 tons was left on for 23\(\frac{1}{2}\) hours and then quickly removed. The behaviour of the pile under 80 tons load gave the impression that it might have carried a maximum of 90 or 100 tons but no more. The maximum settlement was 0.301 in. of which 0.131 in. recovered when the load was removed leaving a permanent settlement of 0.17 in. The immediate settlement under the maximum working load of 40 tons was about 0.05 in.

**Pile-caps.**—The pile-caps are designed for stresses of 1000 and 18,000 lb. per square inch. The maximum number of piles per cap is nine (see Fig. 196). Maximum safe load is 9 x 40 = 360 tons. Minimum size of stanchion base at 40 tons per square foot = 9 sq. ft. = 3 ft. x 3 ft. Spacing of piles is 3-ft. 6-in. centres.

Assuming the stanchion bears evenly on its base

\[
M \text{ on centre-line} = 120 \text{ tons} \times 3 \text{ ft. 6 in.} = 420 \text{ tons-ft.}
\]

less 180 tons x 9 in.

\[
= \frac{135}{285} \text{ tons-ft.}
\]

\[
= 7,660,000 \text{ lb.-in.}
\]
Total punching shear = $8 \times 40$ tons  
(one pile being directly under the stanchion) = 320 tons.  
Minimum $d$ for punching at 200 lb. per square inch is  
$$\frac{320 \times 2240}{200 \times 4 \times 36} = 24.9 \text{ in.}$$  
The pile-cap is 9 ft. 3 in. (= 111 in.) wide.  
Minimum $d_1$ (for $h_1d_1^2$) = $\frac{7,660,000}{\sqrt{193 \times 111}} = 18.9 \text{ in.}$  
The pile-cap is 3 ft. 6 in. deep; $d_1$ = say, 39 in. If we accept a "spread line" of 45 deg., as shown by the dotted lines in Fig. 196, little beam shearing can develop if we make the pile-cap 3 ft. 6 in. deep.  

Total $A_{st} = \frac{7,660,000}{18,000 \times 0.85 \times 39} = 12.9 \text{ sq. in.}$;  
steel provided is twenty-four 1-in. bars ($= 18.85 \text{ sq. in.}$).  
Shearing force = 120 tons = 268,000 lb.  
Lever-arm = say, $0.85 \times 39 = 33.1 \text{ in.}$  
Shearing-bond stress = $\frac{268,000}{33.1 \times 24 \times 3.14 \times 1} = 108 \text{ lb. per square inch.}$
The pile-cap was actually designed to carry the gross moment of 420 tons-ft. Since the central row of piles has a definite width we should, theoretically, increase the moment by \(3 \times 20 \times 0.3 \text{ ft.} = 18 \text{ tons-ft.}\). If the stanchion base is more flexible than the pile-cap, the applied load will tend to concentrate under the centre of the base and the full deduction of 135 tons-ft. may not be realised. Details of the reinforcement are shown in Fig. 196.

It was not possible to place a group of piles centrally under stanchion No. 27 and a cantilevered cap was necessary (see Fig. 193). As the combined centroid of pile group Nos. 16 and 27 is not under the centre of gravity of the combined stanchion loads the piles will not be uniformly loaded. The exact answer depends on the relation between the compressibility of the piles and the flexibility of the pile-cap. This was rendered impossible of solution on this site as the pile-heads are cast in holes bored through the old raft. A first approximation is to assume that the beam sinks uniformly then tilts in a straight line as shown in Fig. 197. If the load on the left-hand pile is \(C\) and the load on the right-hand piles is \((C+E)\) each, then the load on intermediate piles is pro rata to their distance from the left-hand pile. The pile 7 ft. from the left-hand pile carries \(C + \frac{7}{22.5} E = C + 0.311E\).

Also the combined moment of all piles about the centre-line of stanchion No. 16 must equal 153.3 tons \(\times\) 19 ft. 6 in.
Pile Loads  
\[C\]
\[C + 0.078E\]
\[C + 0.156E\]
\[C + 0.233E\]
\[C + 0.311E\]
\[C + 0.690E\]
\[2C + 1.690E\]
\[2C + 2.000E\]
\[\frac{10C + 5.158E}{81C + 73.84E}\]

Moments  
\[-3.50C\]
\[-1.75(C + 0.078E)\]

\[81C + 73.84E = 153.3 \times 19.5\]

\[10C + 5.158E = 171.5 + 153.3\]

giving
\[C = 26.68\text{ tons} \quad \text{and} \quad E = 11.25\text{ tons}.

The loads on the piles from left to right are 26.68, 27.56, 28.44, 29.30, 30.18, 34.44, 36.18 each and 37.93 tons each. The shearing force on the beam changes sign at section x in Fig. 197 and the moment here is

\[+2 \times 37.93 \times 3.7 = +531\text{ tons-ft.}\]
\[+2 \times 36.18 \times 3.5 = +253\text{ , }\]
\[153.3 \times 7.5 = -1150\text{ , }\]
\[-366\text{ tons-ft. (tension at top)}\]

\[= 9,850,000\text{ lb.-in.}\]

\[b = 69\text{ in.}; \quad d = 36\text{ in.}; \quad d_1 = \text{say, 34 in.}; \quad bd_1^2 = 69 \times 34^2 = 79,700.\]

\[\frac{M}{bd_1^2} = \frac{9,850,000}{79,700} = 123.5.\]

\[A_{st} = \frac{9,850,000}{18,000 \times 0.85 \times 34} = 19\text{ sq. in.}\]

Steel provided is 19 \(\frac{1}{4}\)-in. bars,
that is four bars marked (104), nine bars marked (105) and six bars marked (106) in Fig. 198.

Lever-arm = 0.85 \times 34 = 28.9 \text{ in.}

Maximum shearing force = 153.3 - 2(37.93) = 77.44 \text{ tons} = 173,500 \text{ lb.}

Shearing stress = \frac{173,500}{69 \times 28.9} = 87 \text{ lb. per square inch.}

Shear-bond stress = \frac{173,500}{28.9 \times 3.14 \times 19 \times 1.125} = 89 \text{ lb. per square inch.}

The reinforcement is shown in Fig. 198.

**Example of Short Bored Piles**

An example of the use of short bored piles is shown in Fig. 199. A boundary fence 20 ft. high was required on a site consisting of a layer of 8 to 12 ft. of fairly recently made ground and topsoil overlying a deep bed of ballast. The made ground was very mixed including some industrial waste and it was possible that patches of material that could not be bored through might be encountered. A few trial piles went down successfully and the remainder followed without much
The estimated wind load per panel is 1300 lb. at a height of 10 ft. 3 in. above ground level and the weight of one panel of fencing is 2000 lb. The stability of the base is shown in Fig. 200.

Weight of central block, when supports are grouted in solid
3 ft. \times 2 ft. 6 in. \times 2 ft. 3 in. \times 150 = 2530 lb.

Weight of base slab
7 ft. 6 in. \times 2 ft. 3 in. \times 6 in. \times 150 = 1260 lb.

Weight of earth on base
2 ft. 3 in. \times 2 ft. 3 in. \times 2 ft. 3 in. \times 100 = 1140 lb. each end.

Taking moments about the head of the leeward pile,

$$1300 \times 13 \text{ ft.} + R_1 \times 6 \text{ ft.} = 1140 \left(4\frac{1}{2} \text{ in.} + 5\text{ ft. 7}\frac{1}{4} \text{ in.}\right) + 5790 \times 3 \text{ ft.}$$

$$R_1 = \frac{6840 + 17,370 - 16,900}{6} = 1219 \text{ lb.}$$

The load on the leeward pile = 4035 + 28 \times 6 = 6851 lb. = 3.06 tons.

The overturning moment is 16,900 lb.-ft. and the restoring moment is 24,210 lb.-ft., giving a factor of stability of \(\frac{24,210}{16,900} = 1.43\). As we have ignored friction from the surrounding ground this is enough.

When no wind is blowing there is a central load of 2000 lb. and other loads totalling 6070 lb. more or less uniformly distributed over a span of 6 ft.

$$M = 2000 \times 6 \times \frac{12}{4} + 6070 \times 6 \times \frac{12}{8} = 90,630 \text{ lb.-in.}$$
With working stresses of 1290 and 18,000 lb. per square inch, the actual force is 203,000 lb. The block is 32 in. wide overall; 26 in. wide; 7 in. thick; 8 in. high; 6 in. thick at base slab. The block is anchored down to the base slab. When the wind blows the upstanding block, the wind moment at base of block = 1300 x 13 lb.-ft. 

\[ M = \frac{90.630}{27 \times 45} \times 26 = 166 \text{ only.} \]

When the wind blows, the upstanding block (Fig. 193) must be anchored down to the base slab. Wind moment at base of block = 1300 x 13 lb.-ft. 

\[ A_t = \frac{18,000 \times 0.83 \times 45}{90.630} = 1.34 \text{ sq. in.} \]

\[ A_t = \frac{22,500 \times 0.83 \times 45}{203,000} = 7.80 \text{ lb.} \]

\[ A_t \text{ (at a stress of 22,500 lb. per square inch) = } 0.34 \text{ sq. in.} \]

\[ A_t \text{ (at a stress of 22,500 lb. per square inch) = } 0.34 \text{ sq. in.} \]

\[ \frac{26}{26} \times 26 = 780 \text{ lb.} \]

\[ \text{SECTION A-A} \]

\[ \text{SECTION C-C} \]

\[ \text{SECTION E-E} \]
have to turn and get a grip in a fairly thin slab. The loading on the bottom slab under maximum wind is shown in Fig. 202. The maximum moment is $6851 \times 21$ in. $- (1518 \times 16.5$ in. $+ 252 \times 1.5$ in.) $= 118,622$ lb.-in.

$$A_{st} = \frac{118,622}{18,000 \times 0.83 \times 4.5} = 1.40 \text{ sq. in. (at bottom)};$$

the steel provided is eight $\frac{1}{2}$-in. bars ($1.57$ sq. in.) marked (121) in Fig. 201.

The moment at the point of uplift is

$$1219 \times 21 \text{ in.} - (1518 \times 16.5 \text{ in.} + 252 \times 1.5 \text{ in.}) = +222 \text{ lb.-in.}$$

Theoretically there is no tensile stress in the top of the slab, the positive and negative moments being almost exactly balanced. To give a factor of safety and help anchor the upstanding block to the slab three $\frac{1}{2}$-in. bars marked (121) in Fig. 201 are provided. These will resist a moment of $3 \times 0.196 \times 22,500 \times 0.83 \times 4.5 = 49,300$ lb.-in. under conditions of maximum wind load. This will take care of two contingencies that may arise in work of this type. Firstly both piles may be a little out of position and secondly the base may bear partly on the ground instead of wholly on the piles as we have assumed. But there is a much more cogent reason for this top steel. This is an interesting example of a "balanced structure". When the live loading consists of some highly indeterminate pressure acting against and tending to balance other loads it is easy to make an unsafe estimate of the factor of safety. If the structure has to have a real factor of safety of 1.5 with stresses of 1562 and 22,500 lb. per square inch then the stresses must not exceed 2350 and 33,800 lb. per square inch if the live load is increased 50 per cent.

If the upward and downward loads of 6750 lb. in Fig. 202 are increased to 10,100 lb. then $R_1$ becomes $-185$ lb. and $R_2$ becomes 8255 lb. The moment under the down-thrust becomes $148,100$ lb.-in. and the moment at the uplift becomes $-29,258$ lb.-in. instead of $+222$ lb.-in., that is one hundred times as large and of opposite sign.

$$A_{st} \text{ under down-thrust} = \frac{148,100}{33,800 \times 0.83 \times 4.5} = 1.17 \text{ sq. in.}$$

$$A_{st} \text{ at uplift} = \frac{29,258}{33,800 \times 0.83 \times 4.5} = 0.232 \text{ sq. in.}$$

Another, but not quite so striking, example is the foundation described in Chapter X, Fig. 113.
CHAPTER XIV
CAISSONS ON LAND SITES

Open Caissons

The sinking of wells has a long history and the sinking of large piers as open caissons in river or sea work is quite usual but the application of their methods of construction to pits and sumps on land sites is not as common as it might be. If the shape is of no importance a circular caisson is cheapest since it requires no reinforcement against external pressure and has less length of wall for any given area than a square caisson. For a net area of 324 sq. ft. with walls 2 ft. thick, a square caisson requires 80 ft. circumference while a circular caisson requires only 70 ft. Fig. 203 shows a circular caisson sunk to form a clinker pit in a cement works. Fig. 204 shows a small square caisson sunk to form a pumping sump lying about 300 yards W.S.W. from the point marked srrw in Fig. 188 (page 177). The general nature of the ground, as shown by a fairly close borehole, is indicated in Fig. 204. Fig. 205 shows a more complicated shape that was sunk for a tippler pit in Oxford clay. The broken lines show temporary members put in to prevent racking during the sinking operation. The simpler shapes are easier to design and manage. The skin friction is practically independent of the type of ground as the stiffer types of ground can be more widely undercut. The walls may be only 1 ft. 6 in. thick in easy ground or 2 ft. on a more difficult site, equal to a skin friction of 225 to 300 lb. per square foot. If it so happened that there was plenty of cast-iron kentledge and a crane handy, thinner walls could be used but in water-bearing ground the finished job must be heavy enough not to float. Many early caissons were provided with a steel plate to form a cutting edge but this is not necessary except perhaps in coarse ballast where it protects the concrete. The caisson in Fig. 205 weighed 500 tons and had a skin area of

![Diagram](image)

Fig. 203.
3450 sq. ft. equal to 2-9 cwt. per square foot. This was not too much and it had to be watered down.

The method of sinking is normally as follows. The ground is excavated in open cut to a depth of about 4 ft. or 7 ft. above ground-water level, whichever is less. The shuttering for the walls is then suspended from scaffolding or joists spanning across the hole, clamped into position and the walls concreted to a height of, say, 6 ft. for a small caisson or 9 ft. for a larger one. The shuttering is unclamped and swung clear. When this first section is strong enough, excavation to a depth of about 2 ft. is begun systematically and symmetrically on the lines a-a-a-a shown in plan in Fig. 206 starting in the middle of the sides and working towards the corners until the caisson sinks. In stiff ground it is possible to dig 2 or 3 inches outside the wall. The next 2 ft. of excavation is taken out on the lines b-b-b-b shown in plan in Fig. 207 starting at the corners and working inwards. It is important that the excavation is always symmetrical, as once a caisson gets out of plumb it is very difficult to get it back and stop it drifting laterally. If it sticks, water is pumped round the outside. When it has dropped about 4 ft. the shuttering is swung back and another lift is concreted. A difficulty always arises in water-bearing ground. There is no space outside the caisson to accommodate a pumping sump while the floor is being concreted. One method is to use a short length of cast-iron flanged-plain pipe 2 ft. to 3 ft. diameter as a sump and cast this in the floor. When all the rest of the floor is complete and strong enough, a stiffened steel plate (or circular casting) is bolted down on to a gasket to seal the opening and covered with concrete (see Fig. 208).

Difficulties encountered during sinking are usually due to insufficient weight but it is possible to have a case where a caisson finishes in ground that turns out to be very much softer than expected and the caisson tends to keep on sinking after it has reached its correct depth as in Fig. 209. In such a case a series of narrow trenches could be cut across the site, one at a time, and strips of plain or reinforced concrete floor cast in them. If the ground would accept cement grout, tubes could be driven where shown. Failing this, steel sheet-piling could
be driven down outside the walls. Most caissons, after they are sunk, have intermediate floors, cross walls, hoppers, stairs or roofs added to them. This increases their weight against flotation but until these additions are actually built the caisson must be heavy enough not to float. The friction of the ground will tend to resist upward movement but this should only be regarded as a part of the factor of safety. If the caisson as first sunk has not sufficient weight to equal the maximum upward water pressure on the bottom, a ring of relief holes should be left in the wall at the height where danger develops. In simple cases tapered holes may suffice; for example, if the ground water is tidal they could be made good at low water. Otherwise steel tubes or pipes may be built in as shown in Fig. 210.

Caissons may always wander a few inches laterally and, if the exact position is important allowance must be made for this.

**Compressed-air Caissons**

Compressed-air caissons are rarely required except in the immediate vicinity of open water and the remarks in Chapter XVI apply generally to such caissons.
CHAPTER XV

FOUNDATIONS SUBJECTED TO LATERAL LOADS

As with vertically loaded foundations, the first thing to check is the strength of the site against shear-friction failure. A foundation cast on a very smooth rock stratum or on a very smooth layer of mass concrete might slide due to insufficient friction although the coefficient of friction between two faces of concrete could scarcely be less than 1·0. Shear-friction failure is much more likely to occur in the ground immediately below or adjoining the foundation. If we have a shallow block of concrete cast on the ground as in Fig. 211, and weighing \( W \), the force \( P \) necessary to pull it along on a reasonably dry site is not less than \( W \). On water-logged sand it may fall to 0·33\( W \) and on wet clay to the shearing strength of the clay on the plane abc. If we have a deeper block such as the anchor block in Fig. 212 the maximum force \( P \) in Rankine's soil would be \( \frac{1}{2} \left( \frac{1 + \sin \theta}{1 - \sin \theta} \right) w(H^2 - h^2) \).

In a cohesive soil the ground might fail as shown in Fig. 213, the maximum permissible \( P \) depending on the shearing strength \( f \).

**Design of an Anchor Block**

A design for an anchor block to resist both horizontal and vertical components is shown in Fig. 214. This was intended to anchor the cables for a high wireless mast. The topsoil was ignored and various radii were tried for the cylinder of failure, the radius giving approximately the highest value of \( f \) being found by trial and error. This proved to be 12·6 ft. as shown in Fig. 214. The
total tension of 110 tons is resolved into 87 tons horizontally and 67 tons uplift. The block is 9 ft. deep overall of which 1 ft. is above ground and 1 ft. in the topsoil whose strength is ignored. This leaves 7-ft. depth effective to resist lateral pressure and the dimension of 13 ft. 9 in. is chosen so that all the forces run through one point. The block is made 12 ft. 6 in. wide to give a weight of 1.5 times the vertical component.

The intensity of lateral pressure against the chalk face is

\[
\frac{87}{7 \times 12.5} = 0.994
\]

ton per square foot. This pressure is resisted partly by the weight of chalk above the line of shearing failure and partly by shearing on a cylindrical surface 12 ft. 6 in. long and about 14 ft. wide (measured along the curve). Since the block is not infinitely long there is also some friction on the cheeks of the chalk above the shear line and on the two sides of the block which is cast tight against the chalk face. Neglecting this and taking moments about point X

\[
87 \times 9.1 = 31.3 \times 3.75 + 14.2 \times 6 + 12.5 \times 14 \times f \times 12.6
\]

\[f = 0.267\text{ tons per square foot} = 600\text{ lb. per square foot.}
\]

With Rankine’s ideal soil weighing 150 lb. per cubic foot and with an instantaneous angle of repose of \(\theta = 47\) deg., a face 7 ft. high and 12 ft. 6 in. long would resist a horizontal pressure of

\[
\frac{1}{2} \times 150 \times 7^2 \times 12.5 \times \frac{(1 + 0.731)}{(1 - 0.731)} \text{ lb.} = 132 \text{ tons} = 1.52 \times 87 \text{ tons.}
\]

Since the weight of the block exceeds the upward component of the tension it might be thought that we could include the friction between the base of the block and the ground in the forces resisting the horizontal component. This is not so. If we increase the working tension of 110 tons to the ultimate tension of 1.5 \times 110 tons the block is on the point of lifting. All stability problems of this type should be checked under both working and ultimate conditions.
General Remarks

Provided the surface is really level, all but the softest ground will develop a large amount of counterpressure although there may be appreciable horizontal yielding. Screwed-up tie-rods encased in concrete after the load comes on them are better than dead reinforced concrete members. If the effective surface is sloping as in Fig. 215 the effective counterpressure is much reduced or even non-existent although the visible surface is level. This is probably the reason why so many lines of sheet-piling lean forward. In the kind of soft soil usually found forming the banks of estuaries, the author makes the length \( L \) in Fig. 215 equal to a minimum of \( 4H \). Provided that we can accept appreciable lateral movement, sands and clays may be used to resist lateral thrust, but for a structure highly susceptible to horizontal movement of its foundation, such as an arch of small rise-span ratio, these soils are usually dangerous. Even for foundations for a shed frame (see Fig. 43, page 57, and Plates II and III facing pages 58 and 59) where the horizontal thrust is small and the rise very high it is advisable to tie the foundations together on poor soils.

Jacking open the crown of an arch is a process best avoided but on a ballast foundation it may take up dangerous movement. On a clay site the process is far too short to give any indication of the final lateral yielding to be expected. The design of arched abutments and the effect of their lateral movement is discussed in the author's "Reinforced Concrete Arch Design". This includes abutments on piles.

Earthquake Surge

Foundations which carry structures in areas prone to earthquakes are designed to carry a lateral force varying usually between one tenth and one forty-ninth of the weight of the building. In simple cases this sideways lurching can be taken on reinforced concrete walls which extend right down to the foundations. If this is not practicable, stiff framing of columns and beams is necessary. This brings heavy moments on the lowest flight of columns and heavy twisting on the
foundations. This may be dealt with by tying the column feet together by running foundation beams in both directions.

The top part of Fig. 216 shows a simplified version of such a beam joining three columns. Analysis of the superstructure gives a lateral force of 42.9 tons per bay distributed among the three columns as shown, the points of contraflex lying 9 ft. and 10 ft. above the centre-line of the beam. The beam is an inverted T-section 4 ft. wide. The effective load on the ground is 2 tons per square foot or 8 tons per foot run of beam which is increased to a maximum of 12 tons by the overturning effect of the earthquake.

![Diagram of beam and forces](image-url)

**Horizontal Loading**

![Diagram of horizontal loading](image-url)

- A to B: 110 tons ft.
- B to C: 386 tons ft.

**Vertical Loading**

![Diagram of vertical loading](image-url)

- A to B: 243 tons ft.
- B to C: 459 tons ft.

**Combined Loading**

![Diagram of combined loading](image-url)

Fig. 216.
FOUNDATIONS SUBJECTED TO LATERAL LOADS

The moment at the foot of each internal column is shared by the beams fixing the foot of that column, the ratio in which it is split depending on the relative stiffness of the various spans of the beam. In unsymmetrical cases the distribution may be calculated by the displacement method. Assuming that the intersection points, such as A, B and C in Fig. 216, remain in a straight line, each joint has one angular displacement and one equation of equilibrium. In a symmetrical case half the moment is taken on each side.

If in Fig. 216 the values $M_1, M_3$ and $M_6$ are the applied moments and $K_1$ and $K_2$ are the stiffness factors $I/L$ for the two spans

$$M_5 = \frac{K_1}{K_1+K_2} \left( -M_3 + 0.5M_6 - \frac{K_2}{K_1} M_1 \right)$$

Since this particular problem is symmetrical, $K_1 = K_2$ and $M_1 = M_6$ giving $M_5 = -0.5M_3$

$$M_3 = 16.3 \times 9 = 146 \text{ tons-ft.}$$

Therefore

$$M_5 = -73 \text{ tons-ft. (tension at top).}$$

The moments in the beam due to the horizontal loading are shown in the third diagram in Fig. 216. The fourth diagram shows the moments due to vertical loading and the bottom diagram shows the combined loading. The moment $M_B$ due to vertical loading is 386 tons-ft. and the maximum combined moment at B is 386 + 73 = 459 tons-ft. Similarly $M_A$ is 110 + 133 = 243 tons-ft.

It is, of course, usual practice to allow the calculated stresses under combined dead and earthquake loads to exceed normal working stresses by some 50 per cent. If the building is designed for stresses of 1000 and 18,000 lb. per square inch under combined live and dead loading the calculated stresses may be allowed to rise to 1500 and 27,000 lb. per square inch when earthquake loads are added.

Turbo-alternator Foundations

Many items of industrial plant are now carried on elevated platforms or plinths of reinforced concrete and these are referred to by the plant makers as foundations since they represent the underside and support for the various washers, coolers, purifiers, condensers, heaters, etc., that sit on them. From the viewpoint of the structural engineer, the platforms and columns above ground level are superstructures—or at any rate substructures. A simple system of heavy beams in one direction with a thick slab spanning between, usually offers the best solution, designed with an eye to future alterations and additions. A steep fall is required—never less than one in a hundred—to throw off rain water and spillings of industrial and commercial fluids and dusts.

An interesting example of such platforms are those provided for turbo- alternators. The student may be puzzled by the sight of a turbo-alternator, obviously weighing less than 100 tons supported on half a dozen columns capable of carrying more than 1000 tons. He is scarcely to be blamed if he deduces that the design is dictated by questions of vibration or critical speed but such machines do not vibrate and their running speed is nowhere near the critical speed of the platform. Reinforced concrete too, tends to absorb and dampen vibration. If Fig. 217 shows a mass $W$ supported on two similar weightless columns at a height
$H$ above a fixed foundation the natural period of horizontal vibration is $\frac{2\pi}{\sqrt{\mu}}$ seconds, where $\mu = \frac{24gEI}{WH^3}$. If $I$ (in foot units) = 1.33, $E = 3,000,000 \times 144$ lb. per square foot, $W = 44,800$ lb., $H = 20$ ft., and $g = 32$ ft. per second per second,

$$\mu = \frac{24 \times 32 \times 3,000,000 \times 144 \times 1.33}{44,800 \times 8000} = 1230.$$ 

Therefore $\frac{2\pi}{\sqrt{\mu}} = 0.18$ seconds or 333 vibrations per minute.

We have assumed that the columns are weightless but this may be adjusted by increasing $W$ by adding a third of the weight of both columns. This increases $W$ to 52,800 lb. and increases the period to 0.195 seconds (308 per minute). The design of turbo-alternator platforms is purely a psychological process and has no relation to the theory of structures. The value and speed of the machine, its very small interior tolerances and the devastating effect of an encroachment on these produce a mental complex calling for an appearance of great strength and stiffness in its supports. The fatal seizing-up of a turbine makes a deep impression, even on those only indirectly concerned. Many problems in practical engineering are ruled by personal or political reasons. This is an outstanding example. The beginner is advised to copy existing work and refrain from asking tactless questions.

**Foundations for Reciprocating Loads**

The pressure on the ground under the entire base supporting a reciprocating machine should be positive under all loading conditions, particularly on wet sites. Fluctuating pressures seem to pump ground water up under the foundation.

If a rotating weight $W$ in Fig. 218 spins at an angular velocity $\omega$ around a horizontal shaft at a radius $r$ the radial acceleration is $\omega^2r$. The unbalanced force is $\frac{W}{g} \omega^2r$. If $W$ is in pounds, $r$ in feet and $\omega$ in radians per second, the result is in pounds, the symbol $g$ being 32 ft. per second per second. A speed of 100
revolutions per minute is equal to \( \frac{2\pi \times 100}{60} \) or 10.48 radians per second. If \( W = 100 \text{ lb.}, \ r = 3 \text{ ft.}, \) and \( w = 12.56 \text{ radians per second} \) (120 r.p.m.) the unbalanced force is \( \frac{100 \times 12.56 \times 3}{32} = 1485 \text{ lb.} \)

The direction of the unbalanced force is always away from the centre of rotation so it changes direction constantly. The maximum unbalanced force from a reciprocating piston as in Fig. 219 occurs at each end of its stroke and is also equal to \( \frac{Ww^2r}{g} \), where \( r \) is half the stroke. If the piston makes 100 complete backward and forward strokes per minute then \( w = 10.48 \).

A reciprocating machine should not be placed on an elevated platform, such as that in Fig. 217 without checking the natural period. A single-cylinder piston engine rated to run at 300 r.p.m. could wreck this simple structure which is calculated to have a natural frequency of 308 per minute.
CHAPTER XVI

FOUNDATIONS ON SUBMERGED SITES

This chapter contains only a short general survey. Particular methods of construction and design are dealt with in detail in other works.

General Considerations

The primary approach to any civil engineering contract depends on its size and value. A small contract cannot bear the expense of heavy specialist plant nor the cost of continuous technical supervision. In thinly populated areas a small contract is at a great disadvantage if labour has to be imported, housed, fed and amused. These considerations become more important on submerged sites where conditions are more difficult and where the most effective plant may be something not usually owned by small contractors. Even if such plant may be hired the small contractor may have no foreman and no leading hands familiar with its use. On a very large contract where there will be a small army of resident engineers and agents, complicated and expensive plant may be designed and built exclusively for this one contract and can be written off on completion. Such work is outside the scope of a general text-book. On medium-sized projects having at least one general foreman and at least part-time supervision from a resident engineer and agent, good standard equipment is available. On small contracts it is usually a case of managing with such small items of plant as are not required on bigger work and searching round the contractor's yard for odd scraps of structural steelwork that may be cut and welded into something useful. Some engineers are excellent on large works where everything is available on a large scale but are unable to cope with small works where they are thrown back on their personal ingenuity. Some others do well on small contracts but seem bewildered by the size and pace of a large contract. The first and most important thing then is to get the size of the contract clearly in focus and adjust one's mental horizon and method of approach accordingly.

All site investigation should include the collection of data that may affect constructional operations. On submerged sites this becomes relatively more important and should include records of water levels, currents, water temperatures and amount of silt carried.

Methods of Construction

There are at least four general methods of dealing with submerged sites, namely, (1) Avoiding the issue; (2) Excluding the water; (3) Working above water level; and (4) Working below water level by divers.

The problem may be avoided by placing filling to bring the site above water level and then putting down foundations through this filling. This substitutes two simpler operations for one more difficult one. In very deep bridge piers
the deepest part of a river may be partly filled so that the caissons reach bottom at a manageable depth. With a small stream a diversion channel may be dug to bypass the site, or the water may be piped above it as in Fig. 220. In extreme cases the water in an artificial island may be frozen but this is specialist work and outside everyday practice. Water may be excluded from a site by cofferdam or by compressed air. Driving piles from barges or from staging is the most usual form of working above water level but sinking open-topped caissons by grabbing or jetting without using divers is sometimes possible.

**Cofferdams**

The most elementary form of cofferdam is an earthen bank with clay hearting or clay facing sometimes strengthened with boulders or stone pitching. Another elementary form is the double skin of timber filled with puddle clay. On a soft bottom the timber skins were carried on timber piles. On a rock bottom steel mandrels were jumped into the rock. The modern engineer's mind turns so easily to steel sheet-piling that he may forget that there is still the odd occasion when an earth or timber dam might be cheaper or more effective. Whether or not a cofferdam in steel sheet-piling will prove successful depends on two things—firstly a suitable bottom and secondly proper falsework and guides to get the piles pitched dead vertical in both directions. On a clay bottom the problem is easy. With sand or fine ballast longer piles are required (see Figs. 20, 23 and 24 on pages 34 and 38) but driving can be controlled. With coarse ballast (and possibly boulder clay) a large stone may deflect a pile and spring it out of its clutch. One or two sprung clutches may be coped with but any widespread springing may necessitate a second line of piling driven outside the original line with clay puddle dumped between. A hard chalk or shale bottom may split and shatter over a wide area making dewatering impossible. Dumping bargeloads of clay outside the dam might or might not cure this. Before specifying that work should be done inside steel sheet-piling, or any other type of dam, the engineer should make certain that this is practicable. Minor leaks through pile clutches may be stopped by the time-honoured expedient of throwing ashes or other light flocculent material into the water during the process of dewatering. The general remarks about the size of cofferdam in relation to the size of the permanent work in Chapter VI will apply. If a lot of water is coming through the clutches, as may happen with old piling, additional clearance may be necessary all round the permanent work. The strength and safe spans of steel sheet-piling are given in handbooks issued by the firms that market them. One point is not always made clear. Suppose Fig. 221 shows 40 ft. sheet-piling on a non-tidal site, assumed to have an effective ground support 30 ft. from the top. The top waling is put in at water level. The second waling is to go in at a depth of 17 ft. After this second waling is
fixed the external water pressure between the first and second walings is \(\frac{1}{4} \times 17^2 \times 62.5 = 9000\) lb. per linear foot of wall, and the approximate moment on the sheet piling is \(9000 \times 17 \times \frac{4}{3} = 230,000\) lb.-in. per foot. At a stress of 17,000 lb. per square inch this needs a section modulus of 13.5 in.\(^3\) per foot. But if the water level inside the dam has to be lowered 18 ft. before this waling is fixed a much larger moment is produced, the difference between external and internal pressure increasing from zero to 1225 lb. per square foot at 18-ft. depth and remaining at this figure, the effective loading being as shown in Fig. 221 on a span of 30 ft. This loading produces a maximum moment of approximately 1,180,000 lb.-in. per foot requiring a section modulus of 70 in.\(^3\).

![Fig. 221.](image)

If the walings cannot be lowered and fixed under water, although this is now generally feasible, then two sets of walings could be temporarily fixed at A\(_1\) and B\(_1\). Then A\(_1\) could be leap-frogged down to A\(_2\), and B\(_1\) then down to B\(_2\), A\(_2\) being finally brought down to A\(_3\). This means an additional set of walings and struts, and four fixings and three unfixings above water level instead of one fixing below. On a clay bottom excavation inside the dam may be complicated by rising of the ground. The excavation could be taken out in narrow trenches similar to Fig. 209. Concrete, say 2 ft. thick, laid in the bottom of these trenches could be reinforced with rods whose ends are welded to the steel sheet-piling, as the lower ends of the piles are usually flame-cut through and left in position. Perhaps a better solution would be to drill holes, say 4 ft. diameter, well down into the clay and form a series of mass concrete piers inside the dam. In a small dam bored piles could be used. Driving piles into clay inside a dam might cause the ground to heave (Fig. 180) and split the dam.

**Stilling Dams**

Most types of underwater construction are very difficult to control in fast moving water. Currents exceeding 1 knot are likely to cause difficulty. Where the bottom will not accept piles, a rubble wall to break the current or possibly a
timber screen tied back to land anchorages may be necessary before divers can work or caissons be sunk. This is another contingency that the engineer should foresee and provide for.

**Compressed-air Caissons**

Compressed air up to 75 lb. per square inch has been used but pressures of 35 to 40 lb. per square inch are about the limits of normal working. Pressures of 20 to 25 lb. per square inch, corresponding to 46 to 57 feet head of water are common. *Fig. 222* shows a section through a caisson. It should be heavy enough to overcome 2 cwt. per square foot of skin friction (3 cwt. per square foot on a difficult site) on the area in contact with the ground after allowing for the upward pressure of the air in the working chamber. The height $H$ of this chamber should be enough to leave 7 ft. clear during sinking after allowing for the depth to which the cutting edge may sink into the softest stratum encountered. If the caisson is to penetrate hard ground the angle $\theta$ in *Fig. 222* should be steep enough to give easy access for pneumatic tools. The caisson may be pitched by floating out. This may not be possible with a reinforced concrete caisson and the lower part may have to be built in steel plate and filled with concrete after the caisson is pitched. Caissons may be constructed and lowered from overhead framing supported on falsework or on barges (*Fig. 224*). If the ground is open in character loss of air round the cutting edge may be excessive and clay must be dumped round the outside.

The employment of men who will work in compressed air is strictly controlled by Special Regulations of the Ministry of Labour. The special equipment needed makes the cost very high if only a few caissons are required. If many are required the cost of sinking under compressed air on clay or ballast sites is roughly the same as sinking open-topped caissons by grabbing or jetting. In general compressed-air caissons are more certain in action against unknown obstacles such as boulders in clays of doubtful origin, more effective in penetrating harder ground or very coarse ballast and the actual foundation stratum is visible.
Open Caissons

These have something in common with open-topped caissons sunk on land sites (see Chapter XIV). The differences are in the weight, the method of excavation and method of closing the bottom. There is also the difficulty of pitching the caisson in position on a submerged site. If the bottom is very irregular in level the higher parts could be removed by divers but it is more usual to level up by dumping clay into the low spots.

The caisson may be built as shown in Fig. 224, being suspended by large screwed rods. The standard type of Thames barge carries 100 tons. A caisson 20 ft. x 20 ft. with walls 2 ft. thick has a net area of \((20^2 - 16^2)\) or 144 sq. ft. and weighs 5½ tons per foot height submerged. A caisson of this size 30 ft. high could be hung from two such vessels. If the walls were only 1 ft. thick a caisson 60 ft. high could be carried. The normal contractor's cranes are not capable of carrying weights of this order of magnitude but a few port authorities have very heavy floating cranes that may be hired to lift and pitch weights of several hundred tons.

The weight of concrete below water level is only about 85 lb. per cubic foot so a caisson on a submerged site has only about 57 per cent. of the effective sinking weight of a land caisson where the inside is pumped dry. On the other hand the skin friction acts only on the area below bed level. If the excavation is grabbed or drilled out there is less opportunity of undercutting the leading edge and higher skin friction can be expected. Soft soil or sand may be jetted or pumped out. Only in very favourable conditions can the caisson be pumped dry after sinking and the bottom put in as in a land caisson. This is seldom a drawback as most caissons on submerged sites are filled solid while most of those on land sites are needed for pits or sumps. The concrete filling is placed under water by tremie (see Chapter VI) by bottom-opening skips or special bags. (See later in connection with Figs. 230, 231 and 232.) Small obstructions encountered during sinking can be removed by divers. On some variable sites the caissons are sunk through the soft upper strata with open tops but are made with pockets in the walls to receive a heavy diaphragm which is put in as soon as the harder lower stratum is reached and the last stage of sinking is done under compressed air.
Working Over Water

In the early part of the twentieth century it was standard practice to commence water work by erecting timber staging. This staging was usually of timber piles, driven from a barge with timber capping and bracing with steel joists spanning from cap to cap. These joists carried the various cranes, pile frames, etc., required to put down the permanent work. Occasionally complete trestles as in Fig. 223 were made up and launched as a unit. With round timber the caps and sills were dapped, stub-tenoned and drift bolted to the legs as in Fig. 225. With square timber all joints in falsework that has to be dropped, hauled or heavily handled into position should be bolted—not spiked, dogged or hitched. With decreasing usage of timber and increasing shortage of experienced timbermen, the practice of building out from permanent work is increasing. Permanent piles already driven, caissons or cylinders already sunk are used to support heavy cantilevered girders that reach out to the position of the next permanent support and carry the necessary plant at their outermost end. With heavy modern plant and a tendency to longer spans, very heavy construction may be needed. The girders are usually mounted on bogies when being run forward and are then jacked up and packed solid when in their working position. If successive supports are at 30-ft. centres as in Fig. 226 and a total of 60 tons equivalent dead load has to be carried on the end of two girders, the moment in each is 900 tons-ft. or 10,800 tons-in. At 10 tons per square inch a $Z$ of 1080 in.$^3$ is required for each. A pair of plate girders with $48$-in.$\times \frac{1}{2}$-in. webs and $14$-in.$\times 1$-in. flanges would carry this.

Any method of working that keeps all personnel above water level is attractive when the water is very cold. Where the foundations are small and widely spaced as in Fig. 226 the cost of cofferdams would be proportionately very high and falsework would be required to pitch and drive them. Apart from filling in with earth, rubble or cut stone blocks, the timber pile, which was exploited by the Romans, is probably the oldest method of above-water work. Piles of steel or reinforced concrete or possibly aluminium alloy are its natural successors. The author has seen too much trouble with long reinforced concrete piles, particularly rakers, to be enthusiastic about the use of long and slender piles in jetty work. He would restrict their use to easy sites and lengths not exceeding 40 diameters. This
again emphasises the need for bigger and more versatile piling plant to deal with bigger piles. Slender rakers driving into soft ground tend to curl downwards due to their natural deflection as in Fig. 227. When meeting the surface of a harder layer the shoe may tend to skid outwards as in Fig. 228. Prestressed piles, when further developed, may help to solve this problem. Steel box-piles, if these can be protected by adequate concrete casing, have great advantages over concrete piles in handling and driving. A group of three piles, covered above bed level by a concrete cylinder is shown in Fig. 229. On a reasonably soft bottom the steel piles could be driven, the cylinder lowered, or concreted down, and forced several feet into the bottom either by clearing the site by jetting, loading with keelledge or by jacking down against the underside of joists attached to the pile heads.

![Fig. 227](image)
![Fig. 228](image)

Many of the older seaside piers in Great Britain stand on cast-iron screw piles, many of them still in better condition than some reinforced concrete piles less than half their age. Their great drawback is the need for bracing at frequent intervals. This bracing, being of steel sections, has only a short effective life and is a danger to boats and swimmers. A combination of cast-iron piles and aluminium alloy bracing with synthetic insulation to keep the metals out of actual contact seems to offer a possible solution. Reinforced concrete screw piles have been used and the Foundation Code includes some notes on them. They could be very useful for jetties on deep mud banks. The engineer would be well advised to carry out full-scale screwing tests on lengths of pile cylinder and on the helicoidal shoes before completing his design.

**Work by Divers**

In calm, clear water not colder than 60 deg. F. a diver can drill, blast and remove several cubic yards of rock per day at depths up to 60 feet, and a pair of divers will cut out a level seating for a cylinder in a few days under these
conditions. Of all structural operations diving is perhaps the most susceptible to small changes in local conditions. A current of one knot may bother an unprotected diver particularly if it drifts silt across the site and muddy or cold water greatly reduces his output. Divers can descend to depths of well over 100 feet but their output at great depths is very small. In any but the clearest water conditions it is impossible to supervise divers working at any appreciable depth and this may be a great drawback to employing them.

![Diagram of cylinder guide frame](image)

The use of modern versions of the old diving bell is not common in this country possibly because of our large tidal range and consequent tidal currents. A large steel compressed-air caisson has been used abroad as a large diving bell. This was floated out and settled on the sea bed which was excavated or filled in to receive blockwork, cylinders or possibly floating caissons. The steel caisson was then refloated and removed for re-use. Exploration of the ocean by means of a closed sphere has been carried to depths of several miles. This idea could be adapted for visual inspection of underwater sites and could be extended to the use of mechanical plant remotely controlled from inside the sphere or cylinder.

**Example of Divers' Work.**—An example of work carried out by divers is shown in Fig. 231 entailing the seating of reinforced concrete cylinders on a difficult sloping rock site. The site was commanded by a 5-ton derrick crane mounted
on the existing quay wall. Foundations were marked out on the bottom using a 60-ft. rail checked for distance at quay level and plumbed by theodolite. All foundation work was carried out by divers working in pairs from a raft with their compressor, air receiver and jetting pump on shore. The silt overburden was jetted away. The rubble was man-handled into flat steel tipping trays and hoisted by crane. With the site reasonably clear, drilling and blasting was carried out by the divers to provide a seating for the piers, the small debris being cleared by jet and large rubble hand loaded and hoisted as before. The average quantity of rock excavated per foundation was 21 cu. yd. Polar Ammon gelignite was used and averaged 1\frac{1}{2} lb. per cubic yard of rock. Diver-hours per foundation averaged 745. On completion of the seating area the cylinder guide-frame (Fig. 230 on page 211) with the base frame (Fig. 232) attached was lowered by crane, centred, plumbed and secured at quay level with long rails. The guide frame had a maximum height of 66 ft. and the assembly weighed 3\frac{1}{2} tons (see Plate VIII facing page 123). At the open side of the recessed foundation the divers built a "retaining wall" of lean concrete in sandbags to act as shuttering for the foundation concrete and the foundation was then concreted to the top of the base frame (Fig. 232). The first length of concrete cylinder was then placed and concrete raised about a foot inside to complete this part of the operation. The concrete cylinders were 7 ft. outside diameter with 7-in. walls in heights of
5 ft. 4 in., each length weighing 4½ tons. For lifting purposes three lugs were cast at the top inside each length of cylinder, spaced at 120 deg. A three-sling 3-legged lifting frame, made out of steel channel, picked up the lugs and fell clear on placing without having to undo any bolts, chains or shackles. Concreting of the foundations by divers working inside and around the cylinder guide-frame was necessarily slow. A "snorter" bag was used for this operation and consisted of a strong canvas bag suspended from the crane at the open end and having the bottom slit across and temporarily secured with a chain and quick release pin. The bag was filled with concrete and lowered to the diver who released the pin and guided the bag into position. At his signal the crane hoisted the bag slowly and the concrete was placed without disturbance.

![Diagram of cylinder guide-frame and base frame](image)

**Fig. 232.**

**Floating Caissons**

For civilian use floating caissons are usually substructures more than foundations, the foundation work proper consisting of preparing the sea-bed or river-bed to receive them. This is usually done by divers or from a diving bell. If there is a dry dock or slipway available the problems of getting the caissons afloat is simple. Otherwise they can be built and lowered as in Fig. 234. It is not essential to construct a large caisson in one operation. The reinforced concrete unit shown in Fig. 233 is 10 ft. x 10 ft. with walls 7 ft. high, weighs 20 tons and will float with 1 ft. freeboard. If this is the largest unit that can be handled and floated, sixteen of them could be combined to form the bottom of a caisson 40 ft. x 40 ft. Some floating caissons on a soft sea- or river-bed have a number of open bottomless compartments. When the caisson has been settled in position, piles or cylinders
are bored, driven or sunk through the open compartments to reach a firm bearing stratum, the deck of the caisson serving as a working platform. All floating caissons must be checked for stability. The metacentre for the unit in Fig. 233 is 4.69 ft. above the underside of the bottom slab while the centre of gravity is only 3.15 ft. above. The metacentre then is 1.54 ft. above the centre of gravity. In this case the centre of buoyancy is actually above the centre of gravity.
APPENDIX A

BENDING RESISTANCE OF RECTANGULAR MEMBERS OF REINFORCED CONCRETE

Charts of Resistance Moments and Percentage of Reinforcement.—Curves giving the bending strength of reinforced concrete members of rectangular cross-section are given in Charts 1 to 8 on pages 213 to 222. The charts apply to the straight-line (modular-ratio) theory and to the load-factor method in accordance with the recommendations of the B.S. Code No. 114 (1957).

It is not possible to cover all variations of the factors involved, particularly when compression steel is used, as the depth of embedment \( d_2 \) varies. The author has always been very strongly of the opinion that designers should compile their own

PERCENTAGE OF TENSILE STEEL ON AREA \( \frac{bd_1}{M} \) (tensile stress not exceeding 18,000 lb. per sq. in.)

Chart 1.
tables, which are made quite easily by assuming a depth to the neutral axis and working backwards.

With the exception of some of the curves for sections with equal steel at the top and bottom, all the graphs have the same characteristic, that is a rapid increase in bending strength up to a “critical” point and a smaller, or no, increase beyond. The two parts of the curves are, of course, parts of two different curves which intersect at the “critical” point.

Considerations of shearing strength usually determine the minimum thickness of foundations. For example with a square footing slab carrying a fully loaded square column, the effective depth required for resistance to bending is less than 0.75 times the column-diameter while the minimum overall depth required for resistance to shearing is about 1.1 times the column-diameter. It follows that compression steel is usually not required. Since the author does not recommend a stress of 20,000 lb. per square inch in foundations, we are reduced to Chart 1 or 3 for most cases, and usually only interested in the “critical” points on these curves as given in the table on page 218.
Cross-sectional Areas of Reinforcement Bars.—In the table on page 223 are given the cross-sectional areas of round reinforcement bars when spaced at various distances. The weight of 1-ft. length of each size of bar is also given.

Comments on B.S. Code for Reinforced Concrete (No. 114, 1957).—Not only does the 1957 edition of the B.S. Code of Practice for Reinforced Concrete in Buildings (C.P. No. 114, 1957) give a range of permissible stresses but it also gives alternative methods of design. The new load-factor method is in effect the revival of ideas put forward by Professor Talbot and others more than fifty years previously. At that time also the "steel-beam" theory was much in use. If a section had the same area of steel near the top and bottom, then the concrete could be completely ignored in tension and compression, the steel being stressed to ±16,000 lb. per square inch. The London County Council gave official sanction to this idea in their Regulations of 1915. Generally, however, the idea was frowned on by academic experts and became discredited. Using allowable stresses of ±18,000 lb. per square inch and applying the load-factor method to a rectangular section having the same amount of steel near the
PERCENTAGE OF TENSILE STEEL ON AREA $bd_1$
(tensile stress not exceeding 20,000 lb. per sq. in.)

**Chart 4.**

<table>
<thead>
<tr>
<th>Tensile stress in steel not exceeding 18,000 lb. per square inch</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Chart 1 ($m = 15$)</strong></td>
</tr>
<tr>
<td>$f_{eb}$</td>
</tr>
<tr>
<td>---------</td>
</tr>
<tr>
<td>750</td>
</tr>
<tr>
<td>1000</td>
</tr>
<tr>
<td>1250</td>
</tr>
<tr>
<td>1500</td>
</tr>
</tbody>
</table>
PERCENTAGE OF TENSILE STEEL ON AREA bd₁

Stress in steel not exceeding 18,000 lb. per sq. in.
Stress in concrete not exceeding 1,000 lb. per sq. in.

CHART 5.

top and bottom we arrive at results practically indistinguishable from this older discredited idea—an amusing reflection on codes of practice and regulations generally. (The curves marked $A_{sc} = A_{st}$ in Charts 7 and 8 on pages 221 and 222 show these values.)

The strength of a reinforced concrete structure depends on the strength of the concrete and steel and the adhesion between the concrete and steel. The B.S. Code devotes too much space to, and lays far too much emphasis on, test-cube strengths and insists that the reinforcement must conform to a relative British Standard, but fails to insist that, for a sound structure, the concrete must be of such a nature as to ensure high adhesion to the steel when the concrete is placed in bulk around and between a reasonably compact group of bars. In foundation work, adhesion can be more important than compressive strength.

If a man spends ten minutes making a 6-in. test-cube this is 0.75 cu.ft. of concrete per hour. If a 14/10-mixer turns out 200 cu. ft. of concrete per hour, it would require 267 workmen to pack this concrete to the same extent as the test-cube, even in a mass-concrete block. To cope with a heavily reinforced member (say at the joints
of a large Vierendeel girder) might require 500 workmen per mixer. Compared with practice this is about seventy times out of scale.

When a reinforced concrete structure fails or deteriorates in service, the concrete industry is somewhere to blame. It is fashionable to blame the contractor but it is very seldom indeed that he is entirely responsible. Our teaching and research establishments, our Codes and Regulations and our drawing offices must, in fairness, share this blame.

**Bar Spacing and Size in Simple Sections.**—The ratio between the allowable shearing stress and the allowable local shear-bond stress varies a little but is practically constant at a value of 1.75.

The total allowable shearing force $Q$ on a beam of rectangular cross-section is the product of the allowable shearing stress $g$, the breadth $b$, and the lever-arm $l_a$, or $Q = gb l_a$. The allowable local shearing-bond stress is $\frac{Q}{\ell_a^2}$, where $\ell$ is the total perimeter.
of the bars in the breadth \( b \). If \( \frac{Q}{lb} = 1.75g \) and \( Q = qbl_a \), then \( o = \frac{b}{1.75} \); and if \( b = 12 \) in., then \( o = 6.85 \) in. This means that if a rectangular beam is fully stressed in shearing and in local shear-bond stress at the same time, the total perimeter of all bars in a width of 12 in. is 6.85 in.

If the number of bars in 12 in. of width of the cross-section is \( n \), then \( o = \frac{n \times (\text{bar diameter}) \times 3.14}{6.85} = 3.14n \) (bar diameter); that is \( n = \frac{2.18}{(\text{bar diameter})} \). If we have a wide slab with bars spaced evenly, the spacing should be

\[
\frac{12}{n} = \frac{12 \text{ (bar diameter)}}{2.18} = 5.5 \times \text{(bar diameter)}.
\]

Thus to produce these fully stressed shearing conditions \( \frac{1}{2} \)-in. bars would be spaced at 2.75 in., \( \frac{3}{8} \)-in. bars at 4.13-in. centres, and 1-in. bars at 5.5-in. centres.
PERCENTAGE OF TENSILE STEEL ON AREA bd₁

(Stress in steel not exceeding 18,000 lb. per sq. in.)
(Stress in concrete not exceeding 1,250 lb. per sq. in.)

Chart 8.

It is not suggested for a moment that these are values that must be worked to as very few sections are pushed to their limit in both beam shearing and local shear-bond.

The theoretical argument may be taken a step further. If we work to stresses of 1000 and 18,000 lb. per square inch with \( m = 15 \), the critical value of \( M_r \) is \( 193b_d₁^{2} \), with 1-26 per cent. of tensile steel and a lever-arm \( l_a \) of 0-848\( d₁ \). The total steel in a width of 12 in. is

\[
A_{st} = \frac{1.26}{100} \times 12 \times d₁ = 0.151d₁.
\]

But \( A_{st} = n \) (bar diameter)\(^2\) \times \frac{\pi}{4} \), and if \( n = \frac{2.18}{(\text{bar diameter})} \), then the bar diameter = \( \frac{d₁}{11.35} \).

This again is only a theoretical value but it is clear that rectangular sections heavily stressed in shearing require small bars closely spaced to restrict the bond stresses.
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BENDING RESISTANCE OF RECTANGULAR MEMBERS
APPENDIX B

COLUMN STRENGTH OF LONG PILES OF STEEL, TIMBER, CAST IRON AND REINFORCED CONCRETE*

In land foundations where the pile has continuous lateral support from top to toe from the surrounding soil, the resistance encountered during driving is a criterion of the safe load the pile will carry. In jetty work, where the pile has a long unsupported length, the fact that it stands up to a sharp dynamic blow during driving is no proof that it will support a large steady dead load as a long column.

The fundamental reason for the failure of a column is the same as the fundamental reason for the failure of a beam—excessive fibre stress. The difference between them is that the deflection of the beam does not increase its fibre stress, while the deflection of the column causes increased eccentricity of loading with increased fibre stresses and in long columns is the dominant factor in causing failure.

Piles as Columns.—No practical strut is ever quite straight or quite uniform and any load, however small, causes deflection. As the load is increased the deflection increases until the combined effect of load and deflection sends the fibre stress over its ultimate value and the column fails. In addition to the considerations that cause flexure of normal building stanchions, long piles in jetty work have a further cause of flexure. It is extremely difficult (sometimes impossible) to drive piles exactly in their correct position and the ganger or foreman often “humours” them into line with block and tackle to make them register with their bracing connections. Long jetties expand and contract with changes of temperature, and this also flexes the piles. Rakers must flex slightly under their own weight and most jetties have more or less tide drag causing further flexure.

It is clear that in estimating the carrying capacity of long piles, we must start with the conception of a bent column. The method which follows is based on such a conception.

* Reprinted by permission from Civil Engineering, April 1946.
The L.U.D. Theory Applied in Mild Steel Piles (B.S. No. 15).—Let ACB in Fig. 1 be a strut of constant cross-section and overall width a carrying its ultimate load \( P_{\text{ult}} \) the equivalent round-ended length being \( L \). Let \( \varepsilon_0 \) be the accidental eccentricity (due to inequalities in material, inaccuracies of workmanship, etc.), and \( \Delta \) the central deflection

\[
\Delta = \varepsilon_0 \left( \sec \frac{mL}{2} - 1 \right) \quad \text{where} \quad m = \sqrt[3]{\frac{P}{EJ}}
\]

This is no use to us unless we know the value of \( \varepsilon_0 \) and we can only guess this in any given case. We can, however, show that \( \Delta \) cannot exceed a certain limit. The column will fail when the fibre stress reaches the yield point, say 40,000 lb. per square inch.
The worst condition we can conceive at failure, under any system of loading or any amount of accidental flexure, is that the fibre stress along the outside of the curve from A to B is everywhere -40,000 lb. per square inch and the fibre stress along the inside of the curve is everywhere +40,000 lb. per square inch. As elastic conditions are practically constant up to these stresses the value of $\Delta$ under these worst conditions would be

$$\Delta = \frac{1}{2} \times \frac{L^2}{4} \left( \frac{40,000}{30,000,000} + \frac{40,000}{30,000,000} \right) \times \frac{1}{a} = \frac{L^2}{3000a}$$

In a short column which fails before tension develops the value of $\Delta$ must be less than $\frac{L^2}{3000a}$ but the effect of $\varepsilon_a$ is proportionately great since $\Delta$ is small. In a very long column high tension develops and $\Delta$ more nearly approaches $\frac{L^2}{3000a}$ but the added effect of $\varepsilon_a$ is negligible since $\Delta$ is so large. The writer therefore proposes as a practical solution to write

L.U.D. (Limiting Ultimate Deflection) = $(\Delta + \varepsilon_a) = \frac{L^2}{3000a}$ in all cases.

With no transverse loads the maximum fibre stress at C is

$$\frac{P_{ult}}{\text{area} \times \text{section modulus}} = 40,000 \text{ lb. per square inch.}$$

Putting $(\Delta + \varepsilon_a) = \frac{L^2}{3000a}$ and $h = \text{radius of gyration}$

$$\frac{P_{ult}}{\text{(area of cross-section)}} = \frac{40,000}{1 \pm \frac{1}{6000} \left( \frac{L}{h} \right)^2}$$

It is remarkable that this method of design based entirely on a deflection calculated on the overall width of the section results in an expression similar to some semi-empirical Eulerian formula.

As mild steel is equally strong in tension and compression and as the compression stress is always greater in a constant-section column (such as a pile must be) than the tension stress, we can ignore the negative sign in equation (1).
The writer has a serious quarrel with present methods of teaching column design and tabulating column strengths. The student is told that the value $\frac{I}{I + \frac{I}{6000} \left( \frac{L}{k} \right)^2}$ is a reduction factor, inevitably conveying the false impression (fostered by Euler) that long columns fail at small stresses. He should be taught that the value of the denominator of this expression is a multiplying factor which increases the applied axial load $\frac{P_{ult}}{\text{area of cross-section}}$ to the fatal value of 40,000 lb. per square inch and
it is quite clear that transverse loads are completely immune from this multiplying factor. Using a factor of safety of 2.5 we can divide both sides by this amount and write

\[
\frac{\text{(safe value of } P)}{\text{(area of cross-section)}} \times \left[ 1 + \frac{1}{6000} \left( \frac{L}{k} \right)^2 \right] = 16,000 \text{ lb. per square inch}
\]

Ultimate strengths for axial loads are given in Fig. 4.

**High Tensile Mild Steel Piles (B.S. No. 548).**—Assuming yield point and U.L.P. = 49,000 lb. per square inch with \( E = 30,000,000 \) lb. per square inch,

\[
\text{L.U.D.} = (D + \varepsilon_a) = \frac{L^2}{2450a}
\]

Multiplying factor = \[ 1 + \frac{1}{4900} \left( \frac{L}{k} \right)^2 \]

Ultimate strengths for axial loads are given in Fig. 4.

**Example.**—A steel box pile (B.S. No. 548) has the following properties: equivalent round-ended length = 40 ft.; area of section = 36 sq. in.; section modulus = 104 in.3; radius of gyration \( k = 4.65 \) in.

It forms part of a jetty as sketched in Fig. 2. In addition to its vertical load of 70 tons it resists an estimated tide drag of 2 cwt.s. per foot height over an exposed height of 50 ft.

\[
L = 40 \text{ ft.} = 480 \text{ in.}; \quad \frac{L}{k} = \frac{480}{4.65} = 103.
\]

Multiplying factor = \[ 1 + \frac{1}{4900} \times 103^2 \] = 3.17.

Tide drag moment, say \[ 224 \times 50^2 \times \frac{12}{12} = 560,000 \text{ lb.-in.} \]

Stress due to this is \[ \frac{560,000}{104} = 5400 \text{ lb. per square inch.} \]

Applied axial stress = \[ \frac{70 \times 2240}{36} = 4350 \text{ lb. per square inch.} \]

Total effective stress = \[ 5400 \times 1 = 5400 \text{ lb. per square inch} \]

\[
\text{plus} \quad 4350 \times 3.17 = 13,800
\]

Total = 19,200 lb. per square inch.

As the failure stress is 49,000 lb. per square inch, this pile has a factor of safety of \[ \frac{49,000}{19,200} = 2.55. \]

**Strength of Oregon-Pine Pile.**—A pile would scarcely drive unless it had an ultimate crushing strength of at least 1 ton per square inch along the grain, as defective and cross-grained timber goes to pieces under the driving. We may take \( \pm 2240 \) lb. per square inch as a "safe low" failure stress and assume \( E = 1,200,000 \) lb. per square inch up to this stress. Failure of a timber pile is therefore quite similar to that of a mild steel pile except that L.U.D. = \[ \frac{L^2}{2150a}. \]

Multiplying factor = \[ 1 + \frac{1}{4300} \left( \frac{L}{k} \right)^2 \]

Ultimate strength for axial loaded timber piles are given in Fig. 5.
COLUMN STRENGTH OF LONG PILES

EXAMPLE.—A 14 in. x 14 in. Oregon pile stands near the end of a long jetty. It is estimated that temperature expansion of the decking moves the top of this pile a maximum amount of \( \frac{1}{4} \) in. An outline sketch of the pile is shown in Fig. 3. As far as axial loading is concerned its equivalent round-ended length is 20 ft. The temperature expansion will flex it as in Fig. 3. What vertical load will it carry?

\[
M \text{ due to expansion} = \frac{0.25 \times 3 \times 1,200,000 \times 0.083 \times 14^2}{240^2} = 50,000 \text{ lb.-in.}
\]

\[
Z \text{ of 14 in. x 14 in.} = \frac{14 \times 14^2}{6} = 455 \text{ in.}^3
\]

Stress due to expansion \( = \frac{50,000}{455} = 110 \text{ lb. per square inch} \)

Assuming an ultimate fibre stress at 2240 lb. per square inch with a factor of safety of 3, safe stress \( = \frac{2240}{3} \) say, 750 lb. per square inch.

\[
k = \frac{14 \text{ in.}}{\sqrt{12}} = 4.05 \text{ in.}
\]

Multiplying factor \( = \left[ 1 + \frac{1}{4300} \left( \frac{240}{4.05} \right)^2 \right] = 1.82.\)

If \( P \) = safe axial load this pile will carry,

\[
P = \frac{640 \times 14^2}{1.82} = 69,000 \text{ lb.}, \text{ say 30 tons.}
\]

Strength of Cast Iron Piles.—With steel and timber the material has the same effective strength in tension and compression and a uniform value of \( E \) up to column failure. Cast iron has a much higher strength in compression than in tension, while the value of \( E \) drops continuously from an initial value of 15,000,000 lb. per square inch to somewhere about 3,000,000 lb. per square inch at 50 tons per square inch. This means:

(1) That the strength curve for cast iron columns must consist of two distinct intersecting lines, one showing compression failure for shorter columns and one showing tension failure for longer columns. Any curve or set of tabulated values which does not show this distinct break is obviously and fundamentally wrong.

(2) The equation \( f = M/Z \) does not hold exactly for high stresses as stress and strain are proportional, and a solid column has a higher safe stress per square inch than a hollow column with the same value of \( L/k \).

We shall assume the following:

- Compressive failure stress 40 tons per square inch
- Effective value of \( E \) at this stress 5,000,000 lb. per square inch
- Tension failure stress 12 tons per square inch (calculated)
- Effective value of \( E \) at this stress 10,000,000 lb. per square inch

This gives

\[
L.U.D. = \frac{1}{2} \times \frac{L^2}{4} \left[ \frac{26,800}{10,000,000} + \frac{90,000}{5,000,000} \right] \times \frac{1}{a} = \frac{L^2}{3854}
\]

We shall assume that \( f = M/Z \) is sufficiently accurate for hollow piles and arrive at the calculated ultimate strength for axially loaded piles shown in Fig. 6. Strength
tables for cast iron building stanchions often show higher strengths but, as already pointed out, most tabulated values are obviously in error and the writer would not exceed the values in Fig. 6. We must use two multiplying factors.

For compressive stress: \[ 1 + \frac{1}{770} \left( \frac{L}{k} \right)^2 \].

For tensile stress: \[ \frac{1}{770} \left( \frac{L}{k} \right)^2 - 1 \].

Using a factor of safety of 4 on the values already given, the compressive stress due to any transverse loading, plus \[ 1 + \frac{1}{770} \left( \frac{L}{k} \right)^2 \] times the applied axial stress must not exceed 22,400 lb. per square inch. The tensile stress due to transverse loading, plus \[ \frac{1}{770} \left( \frac{L}{k} \right)^2 - 1 \] times the applied axial stress must not exceed 6720 lb. per square inch.

**Example.**—A hollow cast iron cylinder is 3 ft. outside diameter and 1\(\frac{1}{2}\) in. thick. Its equivalent round-ended length is estimated at 50 ft. and it is subjected to tide drag of 300 lb. per foot height over a height of 50 ft. as shown in Fig. 7. It is proposed to put a test load of 200 tons on this cylinder before it is filled with concrete. Is this safe?

Area of section = \(3\times14 \times 34.5 \times 1.5 = 163\) sq. in.

\[ k = \frac{34.5}{2\sqrt{2}} = 12.2 \text{ in.}; \quad I = 163 \times 12.2^2 = 24,300 \text{ in.}^4 \]

\[ Z = \frac{24,300}{18} = 1,350 \text{ in.}^3; \quad \frac{L}{k} = \frac{600}{12.2} = 49. \]

\[ M \text{ due to tide drag} = 300 \times 50^2 \times \frac{12}{12} = 750,000 \text{ lb.-in.} \]

Multiplying factors \[ 1 + \frac{1}{770} \times 49^2 \] = 4.8 compression

and \[ \frac{1}{770} \times 49^2 - 1 \] = 2.1 tension.

Stresses due to tide drag = \(\pm \frac{750,000}{1350} = \pm 550\) lb. per square inch.

Applied axial stress = \(\frac{200 \times 2240}{163} = 2750\) lb. per square inch.

Total compression stress = \(550 \times 1.0 = 550\) lb. per square inch

\[+2750 \times 4.1 = 11,300\]

Total = 11,850 lb. per square inch

This is very safe as we could go to 22,400 lb. per square inch.

Total tensile stress = \(550 \times 1.0 = 550\) lb. per square inch

\[+2750 \times 2.1 = 5800\]

Total = 6350 lb. per square inch

This is just as safe as we could go to 6720 lb. per square inch.

We could just put on a load of 213 tons.

This cylinder would be made in short lengths and would need very strong machined flanges with enough bolts to develop the tensile strength of the cast-iron section.
Reinforced Concrete Piles.—With shrinkage, plastic yield, tensile stresses in the concrete and a varying elastic modulus, the conditions at failure are not easy to assess and preclude the use of easy multiplying factors as used for steel. Assuming no tension in the concrete, a parabolic stress-strain curve in compression with \( m = 15 \) initially, the failure conditions at the critical section will be as in Fig. 10. It is assumed that the stress on the compression steel will be 15 times the apparent stress in the surrounding concrete or 40,000 lb. per square inch whichever is less. Fig. 8 shows the relation between the load per square inch of area at failure and the total eccentricity. The eccentricity due to transverse loads (if any) is simple as it is the moment \( M \) divided by the axial load \( P \). The eccentricity due to column flexure [that is \( (d + e_a) \) in Fig. 1] may be taken as

\[
\text{L.U.D.} = \frac{L^2}{8a} \left[ \frac{40,000}{30,000,000} + \frac{4500}{2,000,000} \right] = \frac{L^2}{2240a}
\]

Adding these two eccentricities, Fig. 8 gives the ultimate load \( P \) the column will carry. If we have no transverse loading we have an axially loaded column and the ultimate carrying capacity is shown in Fig. 10. This also shows a series of values put forward by the writer in the March, 1935, issue of Civil Engineering based on a more academic treatment outlined in his “Reinforced Concrete Design”. The newer values assume that the concrete reaches a higher unit strain but at a lower ultimate value of \( E \). The maximum variation between the curves is about 20 per cent. It is impossible to assess the strength of concrete to closer limits. It is often necessary or prudent to limit the total load on a concrete pile owing to the danger of damaging the pile by hard driving to a small set. Most designers would not load any 14 in. \( \times \) 14 in. pile beyond 60 tons. Building Regulations usually regard the safe stress on a centrally loaded column when \( L = 15a \) (say \( L = 50h \)) as 80 per cent. of the safe stress on a beam of similar concrete.

EXAMPLE.—An 18 in. \( \times \) 18 in. pile is reinforced with 4 rods 1 1/2 in. diameter. It is driven on a rake of 1 in 4 as shown in Fig. 9. It carries 40 tons and is estimated to have an equivalent round-ended length of 40 ft. with a tide drag of 100 lb. per foot run over a height of 55 ft. Is this safe?

Area of steel = 7 sq. in. = 2.17 per cent.
Weight per foot = 324 lb.
Buoyancy = 140 lb.
Total = 184 lb.

Component at right angles to pile = \( \frac{184}{4 \times 12} = 45 \) lb. per foot.

Moment due to transverse loads, say \( (100 + 45) \times 55^2 \times \frac{12}{12} = 440,000 \) lb.-in.

Eccentricity caused by this moment = \( \frac{440,000}{40 \times 2240} = 4.9 \) in.

Eccentricity due to column action = \( \frac{L^2}{2240a} = \frac{480^2}{2240 \times 18} = 5.72 \) in.

\[
\frac{\varepsilon_T}{a} = \frac{4.9 + 5.72}{18} = 0.59.
\]

From Fig. 8, ultimate strength = 860 lb. per square inch. Allowing a factor of safety of three, safe load = \( \frac{860}{3} \times 18^2 = 93,000 \) lb. = 40 tons.
ACKNOWLEDGEMENTS

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