FOUNDATION FAILURES

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LONDON
CONCRETE PUBLICATIONS LIMITED
14 DARTMOUTH STREET, LONDON, S.W.1
First published . . . 1961
PREFACE

Although foundations are concealed below the ground, if they are wrongly designed or badly constructed damage of the superstructure will sooner or later remind the engineer or the contractor (or both) of his mistakes. The properties of structural materials, the statical behaviour of engineering structures, the theories of the calculation of stresses, and the forces that structures must resist are now so well known, and embodied in regulations and codes of practice, that the design of structures is almost standardised, and when a failure occurs it is generally the result of carelessness or mishap rather than lack of knowledge. Foundations, however, are subjected to changes in the properties of structural materials and of the soil, which cannot always be foreseen, and these may occur while the foundation is being constructed or many years later. The consequences of such changes, of unsuitable methods of construction or of dewatering or of bad workmanship, and of our still incomplete knowledge of the real bearing capacity of soils and the stresses that can occur in them, can result in serious damage to, or the total collapse of, the structure supported by the foundation.

Because so much is to be learnt from failures the writer has for many years taken every opportunity of investigating the causes of failures of foundations that have occurred in Hungary and has described many of them in his lectures at the Department of Civil Engineering of the University of Budapest. The writer himself had a share of responsibility for some of these failures, and was pleased to find that other Hungarian engineers were willing to co-operate in permitting him to describe failures with which they have been connected; in this respect he is particularly indebted to Professor-Dr. Á. Kézdi, Professor-Dr. P. Csonka, Dr. H. Lampl, and Mr. R. Ocsvár.

It is natural that engineers should not wish to draw attention to their mistakes, but failures are sometimes due to causes of which there has been no previous experience or of which no information is available. An engineer cannot be blamed for not foreseeing the unknown, and in such cases his reputation would not be harmed if full details of the design and of the phenomena that caused the failure were published for the guidance of others. There can be few important foundation works in which no unknown factors are present.

In the year 1957 the writer published in book form a selection of accounts of failures of foundations in Hungary, their causes, and the remedial measures adopted. This work, which was printed in the Hungarian language and published in Hungary, was so well received in other countries that the writer translated it into the English language, and added more examples of failures in other countries. The present book is the result, and it is hoped that engineers throughout the world who read English will find in it some useful guidance on the causes of failures and the methods by which they can be prevented or remedied. To be forewarned is to be forearmed. The causes of the failure of foundations are so diverse that it would not be practicable to deal with all of them in one book—even if they were all known. But it is thought that the examples here given will direct attention to all the major possibilities of failure, in both the design and the
construction. General reference may be made to *Alapozási Hibák*, by K. Széchy. (Műszaki Kiáldó, Budapest, 1958.)

The need for strong foundations has been known for thousands of years. The principles of constructing them have changed but little; the improvements have been only in new materials, in better equipment, and new methods as a result of our greater knowledge of the properties and of the structural behaviour of soils and the effects of changes in their condition. For example, writing nearly two thousand years ago, the Roman Vitruvius recommended that a foundation in water be constructed within a 'cofferdam formed of a double row of piles the space between which must be filled with clay packed in hampers made of rushes, and well pressed down, and the enclosure then emptied of water by means of water-screws and water-wheels with buckets. This is the method in common use, the difference being in the size of the works, that the piles are of steel instead of tree trunks, and pumps are used instead of screws or water-wheels. During the centuries we have learnt a great deal about the effects of erosion and scouring of the soil beneath a foundation in water, of the effects of seepage of underground water, of changes in the water content of soil, and other phenomena that affect the stability of soils and foundations. Most of this information has been obtained by investigating failures and it is hoped that other engineers will publish accounts of failures in order that our store of knowledge may be increased.

BUDAPEST, April 1960. C. S.
# CONTENTS

<table>
<thead>
<tr>
<th>Introductory Note</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
</tbody>
</table>

## PART I.—INVESTIGATION OF SITE

<table>
<thead>
<tr>
<th>Section</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Absence of Preliminary Investigation</td>
<td>5</td>
</tr>
<tr>
<td>11.1.—Underpinning</td>
<td>5</td>
</tr>
<tr>
<td>11.2.—Costly Construction</td>
<td>10</td>
</tr>
<tr>
<td>Unsatisfactory Preliminary Investigation</td>
<td>10</td>
</tr>
<tr>
<td>12.1.—Variation of Soil</td>
<td>10</td>
</tr>
<tr>
<td>12.2.—Neglecting Ground-water</td>
<td>13</td>
</tr>
<tr>
<td>Incomplete Co-operation</td>
<td>16</td>
</tr>
<tr>
<td>13.1.—The Effects of Surface-water on Soil</td>
<td>16</td>
</tr>
<tr>
<td>Neglect of Possibility of Soil Sliding</td>
<td>19</td>
</tr>
<tr>
<td>14.1.—Sliding Soil</td>
<td>19</td>
</tr>
</tbody>
</table>

## PART II.—UNSUITE TYPES OF STRUCTURES AND FOUNDATIONS

<table>
<thead>
<tr>
<th>Section</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unsuitable Superstructures</td>
<td>21</td>
</tr>
<tr>
<td>21.1.—Excessive Bearing Pressure</td>
<td>21</td>
</tr>
<tr>
<td>21.2.—Horizontal Forces on Foundations</td>
<td>23</td>
</tr>
<tr>
<td>Unsuitable Foundations</td>
<td>26</td>
</tr>
<tr>
<td>22.1.—Unbalanced Horizontal Forces</td>
<td>26</td>
</tr>
<tr>
<td>22.2.—Excessive Bearing Pressure</td>
<td>27</td>
</tr>
<tr>
<td>22.3.—Excessive Safety Measures</td>
<td>35</td>
</tr>
<tr>
<td>22.4.—Foundations at Different Levels</td>
<td>42</td>
</tr>
<tr>
<td>22.5.—Excavations Deeper than Adjacent Foundations</td>
<td>43</td>
</tr>
<tr>
<td>Foundations of Different Types Under the Same Building</td>
<td>45</td>
</tr>
<tr>
<td>23.1.—Different Lengths of Piles</td>
<td>45</td>
</tr>
<tr>
<td>23.2.—Bearing Strata of Variable Thicknesses</td>
<td>47</td>
</tr>
<tr>
<td>23.3.—Structures of Non-uniform Weight</td>
<td>51</td>
</tr>
<tr>
<td>Excessively Rigid Foundations</td>
<td>54</td>
</tr>
<tr>
<td>24.1.—Unreasonable Requirements for a Foundation</td>
<td>54</td>
</tr>
<tr>
<td>Incomplete Assessment of Effects of Loads</td>
<td>56</td>
</tr>
<tr>
<td>25.1.—Dynamic Effects</td>
<td>56</td>
</tr>
<tr>
<td>25.2.—Stress Superposition</td>
<td>59</td>
</tr>
<tr>
<td>25.3.—Neglecting Future Loads</td>
<td>67</td>
</tr>
</tbody>
</table>

## PART III.—DEFECTS AND FAILURES DUE TO DEFECTIVE EXECUTION

<table>
<thead>
<tr>
<th>Section</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unsuitable Methods of Dewatering</td>
<td>69</td>
</tr>
<tr>
<td>31.1.—Land-drainage Structure</td>
<td>70</td>
</tr>
<tr>
<td>31.2.—Bridges</td>
<td>72</td>
</tr>
<tr>
<td>31.3.—Sinking a Caisson</td>
<td>74</td>
</tr>
<tr>
<td>31.4.—Differential Settlement</td>
<td>75</td>
</tr>
<tr>
<td>Faulty Excavation</td>
<td>78</td>
</tr>
<tr>
<td>32.1.—Cofferdams</td>
<td>78</td>
</tr>
<tr>
<td>32.2.—Earth Banks</td>
<td>85</td>
</tr>
<tr>
<td>32.3.—Ineffective Bracing</td>
<td>87</td>
</tr>
<tr>
<td>Faulty Construction</td>
<td>91</td>
</tr>
<tr>
<td>33.1.—Changing the Load on Tapered Piles</td>
<td>91</td>
</tr>
<tr>
<td>33.2.—Unsuitable Spacing of Piles</td>
<td>92</td>
</tr>
<tr>
<td>33.3.—Piles in Wrong Positions</td>
<td>93</td>
</tr>
</tbody>
</table>
33.4.—Piles Spaced too Closely .......................... 95
33.5.—Piles of Unsuitable Length ......................... 96
33.6.—Sliding of a Quay Wall ........................... 96
33.7.—Upward Pressure in Silty Clay ..................... 99
33.8.—Subsidence During Sinking of a Caisson .......... 100
33.9.—Non-uniform Compaction of Loose Soil .......... 102
Defective Workmanship and Materials .................. 106
34.1.—Defective Concrete ................................ 106
34.2.—Defective Sheet-piling ............................ 112
34.3.—Faulty Waterproofing .............................. 113

PART IV.—FAILURES DUE TO EXTERNAL INFLUENCES

Defects Due to Ground-water ............................ 116
41.1.—Seeping and Fluctuation .......................... 116
41.2.—Absence of Weep-holes ............................ 119
41.3.—Oscillation of Water-level ......................... 120
41.4.—Change of Use of Adjacent Land .................. 121
41.5.—Unexpected Saturation ............................. 121
Damage Due to Floods .................................... 121
42.1.—Scouring Due to Floods ............................. 121
42.2.—Combined Flood and Seepage ....................... 124
Changes of Water-content of Soil and Application of Additional Loads 126
43.1.—Increased Water-content of Soil .................. 126
43.2.—Water-content of Stored Material ................ 128
43.3.—Precautions to be taken in Excavating Pits .... 130
Landslides .................................................. 131
44.1.—Bridge Abutment ................................... 131
44.2.—Deterioration of the Strength of Clay .......... 131
Effects of Frost, Changes of Temperature, Drought, and Vegetation 132
45.1.—Frost Under Basement Floor .................... 132
45.2.—Higher Temperatures after Construction .......... 132
45.3.—Effects of Drought ................................ 134
45.4.—Effects of Roots of Trees ........................ 136
Swelling of Clay ........................................... 137

BIBLIOGRAPHY .............................................. 140

INDEX ....................................................... 141
INTRODUCTORY NOTE

Most failures of structures are probably caused by the failure of their foundations. Although their number has been considerably reduced due to the rapid development in recent years of the science of foundation engineering, they are still frequent. Their form and origin are, however, often different. The cost of a foundation seldom exceeds one-tenth of the total cost of a structure, but on the foundation depends the security of the superstructure and an attempt to save on this part of the work either in preliminary investigation, in design, or in materials or workmanship will endanger the superstructure, however well it may be designed and built. A disturbing feature of an unsatisfactory foundation is that the defects and faults seldom appear immediately; most of them are not obvious until the building is in use, when it is very expensive to remedy them.

The design of foundations is far from being an exact science. The same theories of foundations cannot be applied to all the problems that arise, because of the different nature of soils at different sites and even at the same site, different methods of doing the work, and different effects of climatic conditions and groundwater on different soils. Accounts of failures and a discussion of their causes and consequences therefore provide a very instructive guide in the design and construction of future work.

The special circumstances and requirements of any foundation on any site cannot be exactly known until the work is started; therefore experience is the engineer's most valuable asset in deciding on the most suitable foundation.

A foundation differs from the other parts of a structure in that it is supported by soil which is a material of uncertain and non-uniform properties; also the loads acting on this uncertain material cannot be calculated exactly, their magnitude and action often depending on the method of doing the work and the weather conditions at the time the work is done.

The Cost of Failures.

The failure of a foundation may be of such a nature that it would be impossible to remedy it at an economical cost, in which case the loss may exceed the total cost of the foundation and the superstructure.

In other cases to the cost of the remedial measures must be added the loss due to the disturbance or interruption in the use of the building. In many cases the failure of a foundation may result in loss of life. When a failure is the result of an attempt to economise in the cost of its foundation, the loss in recorded cases has been many times the saving that the designer or the contractor tried to make.

The use of an apparently cheaper and simpler foundation, or the use of an
unsuitable method of removing water from the soil, may lead, after an unsuccessful and expensive struggle against various unexpected difficulties, to the resort to a more expensive deep foundation which should have been designed at the beginning. In other cases the difficulties encountered in constructing an apparently cheaper foundation, together with the consequent delay, may result in an inferior foundation costing more than a better but apparently dearer one.

Causes of Failures.

There are so many factors that affect the stability of foundations that it is not easy to classify them into groups. Generally, however, the failure of a foundation is due to (1) The absence of a proper investigation of the site or a wrong interpretation of the results of such an investigation, (2) Faulty design of the foundation, (3) Bad workmanship in the construction of a foundation, and (4) Insufficient provision in the design for exceptional natural phenomena such as thermal and biological conditions, rainfall, and floods greater than those hitherto recorded at a site. These are discussed in the following:

(1) Nowadays it is the exception to design a foundation without an investigation of the site. The investigator's report is generally made in a satisfactory manner, but it often happens that the results of the investigation and tests are not properly evaluated or that their significance is not properly understood by the contractor or even by the engineer. Indeed in some cases collaboration between those who make the preliminary investigation and those responsible for the work has been almost non-existent. It is important that the engineer and the contractor should be aware of all the results of the tests of the soil, and particularly that they should take into account the variations of the properties of any different strata below the surface. It is also important that those undertaking the investigation and tests should know the nature of the structure and the requirements of the foundation. There should be close collaboration between the engineer and the contractor and those making the investigation and tests, not only before the foundation is designed but also during construction if any unexpected changes should be found in the soil or if the load on the soil is altered.

It must be borne in mind that all the important properties of a soil cannot be accurately judged from the results of preliminary borings and laboratory tests, as unexpected changes of importance may be detected only when the excavation is made or when the foundation is being built. For this reason there must be intimate collaboration between the site investigator, the engineer, and the contractor when the design is made and throughout the construction, and when the work is completed the site investigator should measure any settlements and compare them with the preliminary calculations and expectations. Wherever possible it is also desirable that those responsible for the foundations of neighbouring structures be consulted.

Among the sources of failures are the defective extraction, handling, and delivery to the laboratory of samples of the soil; this is particularly likely to happen in the case of cohesionless soil and stratified soils. A frequent source of defective data is the faulty investigation of the ground-water, particularly changes in the run-off and infiltration due to possible changes in the vegetation on the surface. An examination of a site cannot be satisfactory unless it takes into account possible changes of the run-off of surface water due to the removal
of vegetation, the permeability of the surface of the ground adjacent to the foundation, and the effect of the weight of the structure, all of which may cause movement or sliding of the surface. Such an elaborate investigation is, of course, necessary only in the case of important structures; it may be modified in the case of smaller or less important structures or if some settlement would not seriously affect the safety or use of the structure.

(2) In some cases an unsatisfactory design is due to an unreliable investigation of the site or to lack of full co-operation between the engineer, the contractor and those responsible for the investigation. In the case of large hydraulic works a faulty design may be the result of failure to make tests of models to determine the effects of uplift, seepage, scour, the flow of the water, and so on. Another important cause of faulty design is failure to consider the interaction, or reciprocity, between the soil and the foundation structure or the deformation of the subsoil when it is subjected to loading, with the result that there may be uneven settlements on soils of non-uniform compressibility. On the other hand much money can be wasted in providing absolutely rigid foundations for structures which would not be seriously affected by differential settlement. Another source of failure is the use of a type of foundation that is unsuitable for the type of structure it is to support. The loads may not be correctly taken into account, or proper consideration may not be given to possible changes in the subsoil due, for example, to vibration, scour, changes in the level of the ground-water, the erection of other structures near by, and so on. An example is a pumping station designed for the installation of steam engines which were later replaced by internal-combustion engines, the frequency of vibration of which coincided with the natural frequency of vibration of the subsoil with the consequent failure of the foundations of the machines. The use of separate foundations of different load-carrying capacities to carry the same load at different parts of a building, or the use of similar separate foundations to carry different loads, have also been responsible for failures, particularly when there are no gaps between parts of a structure to allow for differential settlement.

(3) Defects in the actual construction of a foundation may be the result of defects in the design or of defective site-exploration, or they may be due to poor workmanship or the use of inferior materials. One of the commonest dangers arises in the removal of water from an excavation or a caisson. For example, the use of pumps for removing water from an excavation in quicksand or similar soil may result in such a serious deterioration of the surrounding soil as to endanger adjacent buildings. Cofferdams that are not sufficiently braced, or of insufficient depth, or that are not sufficiently watertight, have also been the cause of serious failures. Among other causes of unsatisfactory foundations may be mentioned unsuitable constructional procedure, inadequate or inefficient equipment, and poor workmanship, including concrete of poor quality placed under water, leaking sheet piling, the imperfect placing of layers of insulating material, the unsatisfactory operation of pumps, compressed-air plants, pile drivers and so on.

(4) In many cases it is impossible to foresee the possibility of damage due to causes such as scour, seepage, floods, extremes of temperature, and biological and chemical effects, and combinations of these and perhaps other phenomena. For this reason, in Part IV accounts are given of actual failures that have resulted from causes that were unexpected or that could not have been foreseen.
Although foundation failures are never due to a single cause there is always one major cause. In the following an attempt is made to explain the causes of the failures. The examples are grouped under the general headings according to the major cause of the failure, and an attempt is made to indicate how in some cases they might be anticipated in future.
PART I

INVESTIGATION OF THE SITE

In the case of small buildings it is still the practice to take out the soil to the depth of the foundation and to assess its suitability by visual inspection and experience gained on similar soils. Some fifty to sixty years ago this was the general practice for all buildings, and even in the case of abutments and piers for bridges investigations were made to a small depth only below the surface of a stratum that appeared to be suitable for carrying the load. This can, however, be dangerous in some cases, because the exposed soil may be a thin layer only, overlying a very weak stratum, in which case a fairly small load would affect the lower stratum and result in settlement.

This has happened in an area of the city of Budapest which is built on the site of a former side-branch of the River Danube. Many centuries ago this branch became separated from the main river, its bed was slowly covered by silt brought down by the Danube when it was in flood, and it became a marsh which was later filled with artificial material. Consequently there are layers of peat and other materials of small bearing capacity in thicknesses of 1 ft. to 5 ft. at depths of 10 ft. to 15 ft. below the surface of the filling. Failure to ascertain the nature of the subsoil led to a number of failures in this area.

Absence of Preliminary Investigation.

(II.I). UNDERPINNING A BUILDING WITH PIERS.—In Fig. 1 is shown a cross-section of a large hospital and its foundation built in the aforementioned area in 1902 on a site that was a marsh until the beginning of this century. The strip footings were built at ground-water level on a layer of sand from 7 ft. to 9 ft. below the surface. The building has been subjected to differential settlement ever since it was built; the rate of settlement increased when the level of the ground-water was raised in the early 1930's and serious cracks appeared in the façades over doors and windows (Plate I, facing page 26) and in the staircase (Plate II, facing page 26).

An investigation of the site revealed the subsoils indicated in Fig. 1, where it is seen that the upper layer of sand has a greatest thickness of 2 ft. 6 in. and overlies a layer of very compressible peat from 4 ft. to 5 ft. thick; the water-content of this layer was increased when the level of the ground-water rose in the early 1930's.

In the year 1935 it was decided to underpin the structure by constructing shallow piers extending from under the basement to the gravel as shown in the illustration, and to support the main walls on reinforced concrete beams carried on these piers. In undertaking this work more mistakes were made which led to further unnecessary expense. For the construction of the piers shallow wells were formed from which the earth was excavated by hand and the water was pumped out. An attempt was made to collect the water from several wells into a central well, in order to keep down the number of pumps required. This resulted in an upsurge of the layer of sand in the well and the subsidence of the
adjacent area of ground. Finally the ground-water was allowed to find its former level and the wells were sunk by dredging and placing concrete in the water.

There were two mistakes in the design of the original foundation, namely the differential settlement of the layer of peat of varying thickness was ignored, and similar foundations were used for the outer walls and the main interior walls in spite of the fact that the interior walls carried heavier loads than the outer walls, with the result that the interior of the building settled farther than the exterior (see also 23.22).

(ii.12). UNDERPINNING A BUILDING WITH A SLAB.—The building shown in Fig. 2 is a hospital that was built in the same district in the year 1904 on strip foundations under the walls without an investigation of the soil below the sand, which extended to a depth of only 5 ft. 6 in. from ground level and under which were layers of peat and silt. During the period of forty years after it was built differential settlements of up to 1 ft. occurred and large cracks appeared. The settlement was continuous but it was greater each spring; this was due to the higher level of the water in a neighbouring creek in the spring, resulting in an increase in the water-content of the soft peat underlying the sand.

In order to prevent further differential settlement a reinforced concrete raft was constructed at ground level under the whole of the building, as is shown in Fig. 2. First the reinforcement was placed in the areas between the strip foundations, and half the bars in the transverse direction were passed through holes made at intervals in the footing. The slabs were concreted and the holes filled with concrete, and when this had hardened the remainder of the footing was
broken through, bars left projecting from the slab were passed through from both sides and wired together, and these holes were filled with concrete. The result is that there is a uniform pressure on the whole of the site covered by the building and the "pressure bulb" due to the load penetrates deeper than the load of the original strip foundations. In this case, in spite of the extension of the area of stress, further settlements were reduced because the compression of the soft upper strata was reduced by an amount that exceeded the settlement due to the extension of the stresses to the deeper but less compressible strata.

The settlement due to loading a homogeneous material is always proportional to the area of the "pressure bulb", but in the case of a stratified soil the ordinates on a stress-penetration diagram must be divided by the respective moduli of compression in accordance with the basic formula of settlement \( \Delta s_i = \frac{p_i \Delta h_i}{M_i} \), in which \( p_i \) is the intensity of the stress at mid-depth of the layer, \( \Delta h_i \) is the thickness and \( M_i \) is the modulus of compression of the layer.

(11.13). UNDERPINNING A BUILDING WITH PILES.—Another method of underpinning a building that was settling excessively was used in the same area. This is a large apartment house and was built about the year 1910. No attempt was made to ascertain the nature of the subsoil. Differential settlement occurred soon after it was built and cracks occurred mainly in the central part containing the staircase. The settlement and cracks were not observed to increase until about the year 1951, when they quickly increased to such an extent that the structure was unsafe.

In the year 1954 an investigation of the site was made. The top layer, in
FOUNDATION FAILURES

depths varying from 17 ft. to 20 ft., consisted of a mixture of rubble, slag, and house-refuse with a fairly high content of organic matter (the loss on ignition was from 1 per cent. to 10 per cent.; the density was 93 lb. per cubic foot, and the angle of internal friction of the consolidated material was 26 deg.). Under this was a dense mixture of sand and gravel varying in depth from 29 ft. at the middle of the site to 16 ft. 7 in. at one side and 21 ft. 8 in. at the other side, thus conforming to the trough-like shape of the bed of the ancient branch of the Danube. This layer provided a good bearing (its void content was 38 per cent., the effective grain diameter was 0.6 in., the angle of internal friction was 45 deg., the index of uniformity was 110, and the density was 110 lb. per cubic foot).

The strip foundations extended to a depth of 16 ft. 8 in., so that they were on good bearing ground at the sides of the building but not at the middle where the settlement occurred. The cellar had a floor 2 ft. 8 in. thick which helped to

![Diagram showing soil profile and settlement graph](image)
INVESTIGATION OF SITE

distribute the load of the building towards the sides, and it was the cracking of this slab at the middle that caused the further damage in 1954. This slab was also reinforced, but the main bars were parallel to the main walls instead of extending transversely towards the strip foundations under the interior walls of the building where the foundations did not extend to firm soil; it is assumed that this reinforcement was intended only to distribute the greater load where heavy goods were stored in the cellar rather than to distribute the weight of the building over the site. Fig. 3 shows that the settlement at the middle was as much as 11 in. at the fifth floor and 10 1/2 in. at the ground floor, and that it followed the trough shape of the old river-bed at the middle of which the foundations rested in compressible material.

As a result of this excessive settlement the central part of the building had to be demolished and the foundations strengthened. Bored piles were formed at each side of the main internal walls and penetrated 4 ft. to 5 ft. into the gravel. The walls were therefore underpinned so that they rested on a continuous reinforced concrete beam cast on top of the pile-caps. The piles, each of which can carry a load of 65 tons, were cast in place with grouted concrete (that is the aggregate was placed first followed by colloidal cement grout); this had the advantage that the grout, when slowly rising from the bottom, not only filled the voids in the aggregate but was forced into the soil surrounding the piles and so increased the bearing area. (The amount of material used for each pile was from three to six times as much as was required to fill the shaft only.) The part of the

FIG. 4.
FOUN DATION FAIL URES

building that was restored had a volume of about 80,000 cu. ft., and the cost of the work was about £25,000.

(11.2). COSTLY CONSTRUCTION.—The need for at least a cursory examination of the subsoil in the case of even unimportant or small structures is shown by the failure of a bridge with a span of only 8 ft. 4 in. crossing a seasonal watercourse. Before the work was started it was noted in the dry season that the creek was filled with old building rubble and leaves. It was proposed to use two foundation blocks each 6 ft. 4 in. wide by 3 ft. 8 in. deep as shown in Fig. 4. During the excavation, rubble and fallen leaves were still present at this depth, so test piles were driven from which it was concluded that a suitable bearing existed at a depth of a further 5 ft. and it was decided to extend the foundation to this depth. As there would have been a distance of only 3 ft. 9 in. between the two foundation blocks a solid foundation was provided comprising 170 cu. yd. of concrete stressed to only 1-2 tons per square foot. A preliminary examination of the subsoil would probably have resulted in the design of a bridge having the form of a light hollow rectangular closed frame at much less cost and with the advantage of less weight.

Unsatisfactory Preliminary Investigation.

Because a careless examination of a site, or a false interpretation of the results, may have consequences as serious as if no attempt were made to ascertain the properties of the subsoil, the following examples are given in order to indicate the information that must be obtained.

(12.1). VARIATION OF SOIL.—A bridge to carry a road over a railway in a deep cutting was urgently wanted in the year 1950 and was built after only a cursory investigation of the soil with the aid of bores of small diameter and an assessment of the experience gained in providing foundations for other structures in the neighbourhood. The three bores indicated that there was a layer of stiff brown clay which was fairly near the surface and appeared, by inspection only, to be uniform for a considerable depth. Based on experience a safe pressure on this material would be 3-3 tons per square foot and a raft foundation would be suitable without dewatering the subsoil. When the excavations were made the clay was seen over the whole area and the concrete foundation was built, followed by the plain concrete abutments and wing walls. Almost as soon as this work was completed one of the abutments settled a distance of 4 in., and as a consequence the design for a reinforced concrete bridge was abandoned in favour of a light steel superstructure which would impose less weight on the soil.

Regular observations were then made, and showed that settlement continued at an increasing rate which varied at different parts of the structure. At the same time the subsoil was properly examined. Bores of ample depth were made under the foundation and samples taken in an undisturbed state. Tests showed that the water content, and therefore the condition of the apparently uniform clay, varied considerably. For example, at one bore the relative consistency was 0.75 at a depth of 3 ft. 4 in., 0.60 at 5 ft., 0.40 at 10 ft., 0.20 at 20 ft., and 0.50 at 21 ft. 8 in. The void-ratios were 0.83 at a depth of 5 ft., 0.94 at 10 ft., 0.80 at 13 ft. 4 in., and 0.74 at 18 ft. 4 in. However, the plastic limit was about 0.16 throughout the entire depth, and the liquid limit varied only from 0.30 to 0.44 (Fig. 5). These variations of the condition of the clay were the cause of the trouble and should, of course, have been known before the foundations were designed.
As is seen in Fig. 7 the plan of the foundation is irregular because the axis of the bridge is not at right-angles to the railway.

The curve in Fig. 6 shows the settlement during a period of years (the numbers in circles relate to the positions numbered on the plan in Fig. 7). It is seen that the greatest settlement when the bridge was first loaded was at (2) where the subsoil was saturated, and was much less at (3) and (4). The settlement at (3) is greater than would be expected by comparing the pressures on the ground and taking into account the unsymmetrical shape in plan of the foundation and the eccentricity of the load. It may be that the lack of symmetry of the foundation and the non-homogeneity of the soil caused the foundation to tend to rotate about an axis other than that indicated by theory.

The subsoil under the other abutment (3' and 4') was drier, and the settlement was less in consequence; the settlement at these places has, however, since increased, probably as a result of a transfer of load or of later increase of moisture content. At (2) the settlement had exceeded 2 ft. by the year 1958, and was still
INVESTIGATION OF SITE

increasing at the rate of \( \frac{3}{8} \) in. a year. At the other places the settlement was less than 1 ft. and was increasing at the rate of only \( \frac{1}{2} \) in. a year. The other abutment had settled twice as much as that to which Fig. 6 relates. Because the settlement was continuing, no remedial measures had been started at the time of writing (1959).

(2.2). NEGLECTING GROUND-WATER.—More serious consequences resulted from an incomplete investigation of the site of a turbine-house at a hydro-electric works at Kesznyéten, Hungary.

The works (Figs. 8 and 9) comprise three main parts differing greatly in weight and purpose. The water-level in the lightly-loaded intake channel is 40 ft. higher than in the outlet. The soil impermeable between them is of moderate thickness and had to be cut through in places; this soil is a loam with a plastic limit of 28 per cent., a liquid limit of 40 per cent., and a coefficient of permeability of \( 10^{-5} \) in. per second. The outlet channel is also lightly loaded but is subjected to upward pressure with a considerable hydraulic head and to erosion by the water from the turbines. The turbine-house is at the centre of the works and transmits a heavy load to the subsoil, and requires a watertight wall to prevent the percolation of ground-water; its lowest operational level is 3 ft. 4 in. below the bottom of the outlet channel.

Preliminary bores indicated sufficiently well the physical characteristics of the subsoil, but no examination was made of the ground-water conditions. With no knowledge even of the levels of the ground-water it was decided that dewatering could be effected within a sheet-piled cofferdam by direct pumping. The deepest foundation of the heavy turbine house was designed to be on a layer of sandy gravel with good bearing properties at the level +284 ft. above sea level while the foundation of the outlet channel was 6 ft. 8 in. higher on fine sand with smaller bearing properties. A cofferdam of steel sheet piles was provided for the construction of the foundation of the turbine-house and including the first part of the outlet channel, and the excavation for the foundation of the inlet channel was done in an open pit. In all cases the water was removed by direct pumping.

Sheet piles 33 ft. 4 in. long were to be used and to penetrate the water-bearing silty gravel to a depth 8 ft. 4 in. below the level of the bottom of the foundation, which was thought to be sufficient to prevent upsurge of the soil within the cofferdam. It is seen from Fig. 8 that the layer of silty gravel suitable for carrying the load is below a layer of fine sand varying in thickness from 6 ft. 8 in. to 8 ft. 4 in. and loam from 27 ft. to 34 ft. thick. Because sufficient sheet piles 33 ft. 4 in. long were not available, pairs of piles 26 ft. 8 in. long were used between each pair of the longer piles. It was desired that the work be done as cheaply as possible. This design was sound with regard to stability, but from the practical point of view of construction it had several defects that caused trouble during construction and afterwards.

When the excavation within the cofferdam reached a depth of 5 ft., and temporary wooden piles had been driven to support some trestles, ground-water surged up around one of the wooden piles and the inflow increased to such an extent that more pumps had to be used. As the excavation proceeded the layer of loam (which it had been assumed was impervious) became wetter and softer as more ground-water surged upwards and formed small "volcanoes" (Plate III, facing page 26), until the loam had the consistency of soft mud (Plate IV,
facing page 26) and the work was stopped. Investigation showed that the underlying layers of fine sand and silty gravel were entirely waterlogged and the existing artesian pressure would cause the water to rise to the top of the cofferdam. It was estimated that some 690 cu. ft. of water per second would have to be removed to keep the cofferdam reasonably free from water, and this would certainly have affected the stability of the subsoil and also of the surrounding soil. The site of the structure is a natural embankment 30 ft. high formed by an ancient watercourse, so that it straddles the natural flow of the ground-water. Also, the sheet piles were not long enough to prevent the inflow of water.

A different method of construction was then adopted. Two pneumatic caissons (Plate V, facing page 27), one measuring 30 ft. by 72 ft. in plan and the other 40 ft. by 72 ft., were sunk about 2 ft. apart; the bases of the caissons were watertight, the sides were built up as sinking proceeded, steel sheet piles and grouting were used to exclude water from the gap between the two caissons, and the foundations were successfully built.

In the case of the foundation for the inlet channel, a temporary cofferdam formed of steel sheet piles was used and the water was removed by pumping. The piling, which should have been left in position as a protection against scour, was, however, removed. As a result of this lack of protection, and also due to the possibility of differential settlement of the different parts of the works, further difficulties arose. The foundation of the pipe carrying water under pressure from the intake channel was, for some distance, similar to the foundation of the channel, whereas the deeper foundation for the remainder of the pipe was built in the foundation for the turbine-house. The differences in the rigidity of the two parts of the foundation inevitably resulted in different settlements, and cracks occurred in the pipes, which were lined with welded steel plates to make them watertight, and some 750 cu. ft. of grout were used to remedy the scour caused by the leaks in the pipes.

Still another result of the insufficient protection of the foundation of the outlet channel was the collapse of parts of a stone revetment. Some parts of the revetment adjoining the outfall were eroded to a depth of 15 ft. at the bottom and 5 ft. on the slope (see Fig. 9), and these were repaired with broken stone. This type of damage frequently occurs in hydraulic works of this kind.

Incomplete Co-operation.

(I3.1). The Effects of Surface-water on Soil.—The foundation for a crane gantry (Fig. 10) in the stock-yard at a steelworks is an example of a failure which shows that the most thorough examination of a site is of no avail unless the results are acted upon by the designer. The soil investigators were asked to report on the properties of the soil and also to advise on the best type of foundation. Bores were made and samples of the soil tested in a laboratory. The bearing capacities were accurately given, together with suitable depths and dimensions for the foundations under each masonry pier. The report suggested the need for making exact calculations of the settlements in relation to the sensitivity of the structure, and mentioned the need to provide drainage to divert the surface-water, but did not indicate a means of dealing with the problem.

As is seen in Fig. 10, the foundations of all the piers are in silty sand, which has an angle of internal friction of 22 deg. to 32 deg., a void-ratio of 0.61 to 0.68,
FIG. 10.
and continued after they were built to such an extent that the construction of the superstructure (a steel gantry) was suspended.

A record of the settlement of the piers is given in Fig. 11; these settlements were continuing a year after the piers were built. The differential settlements of the piers according to their positions on the sloping ground (indicated by numbers at the top of Fig. 10 and against the curves in Fig. 11) have the following points of interest. Under piers Nos. 1 and 2 the layer of silty sand is thinner and the settlement was sudden due to the underlying cohesionless soil, whereas the greater the thickness of the intermediate layer of silty sand the slower the rate of settlement. At the end of five months the settlement of the piers higher up the slope was continuing. The reason for the differences in the settlement of the piers was that the greater weight of the increasing thickness of the filling towards the bottom of the original sloping ground level compressed the underlying soil to a greater extent than the thinner layer of filling higher up the slope. Also, the intensively compacted filling, as a result of friction developed against the sides of the piers, added to the load on the piers in proportion to the depth of the filling.

On the other hand, due to the omission of drainage, the compressible layers of topsoil at the higher part of the original ground level were continuously more saturated and their compressibility increased than was the case lower down where the slope of the original surface was increasingly protected against surface-water as the depth of the filling increased. The fault in this case lay chiefly with the soil investigators, who were not sufficiently emphatic in drawing attention to the unusual conditions that existed at this site and the consequent need for exact calculations of settlements and the provision of drainage, the importance of which was not realised by the designers.

**Neglect of Possibility of Soil Sliding.**

(14.1). Failure Due to Sliding Soil. The result of disregarding the possibility of the sliding of soil as a result of excavation is shown in Fig. 12. This
is a coal store in soil which, due to its geological formation, was susceptible to sliding. The design was, however, based on an investigation of the site, which indicated that the clay subsoil had considerable strength in shearing. The properties of the soil were a cohesion of 0.3 to 1.2 tons per square foot, a plastic limit of 20 to 25 per cent., a liquid limit of 63 to 75 per cent., and consequently a bearing capacity of 1 to 2.5 tons per square foot.

The conclusion was drawn that it would be satisfactory to have vertical earthen walls to a height of 13 ft. 4 in. and a slope of 1 in 2 above, and the excavated material was deposited at the edge of the pit. The plan of the pit was a curve following the contour of the land, but during construction this was changed and the alignment straightened without consulting the investigators who reported on the site, and as a result the depth of the pit was extended a further 15 ft. to 20 ft.

The sides of the excavation remained stable during a dry summer, but the autumn rains caused them to slide before the staging for unloading the coal was built. The drawing shows the original design and the results of the sliding of the saturated soil. The site was originally forest land and the trees prevented much of the rainfall from percolating into the ground; also their roots acted as a natural reinforcement of the top layer. When the rainy season came after the trees had been removed the colloidal clay had lost this natural protection, became saturated, and so lost its resistance to sliding. The pit was finally constructed as shown in Fig. 12, with flatter slopes and with the insertion of a small retaining wall; in addition surface drainage was provided over the whole area of the sloping ground. Much of this additional cost could have been avoided if those who examined the site had taken fully into account the geological conditions and the effect of the existing vegetation, and if the shape of the plan and the depth of the pit had not been altered without consulting the site investigators.

A similar example is given in (32.2) on page 86.
PART II

UNSUITABLE TYPES OF STRUCTURES AND FOUNDATIONS

In addition to imperfect knowledge of the ground, dealt with in Chapter I, failures may be due to faults of design such as the provision of unsuitable types of superstructure or foundations and incorrect assumptions regarding the imposed loads or other effects.

Unsuitable Superstructures.

In the following some examples are given of structures liable to be seriously affected by settlement and which were erected on compressible ground of low bearing capacity.

(21.1). EXCESSIVE BEARING PRESSURE.—The old bridge over the River Tisza at Szeged, Hungary, was constructed between 1880 and 1883 and comprised four clear spans of fixed (hingeless) arches having lengths of 362 ft., 320 ft., 284 ft., and 217 ft. (Fig. 13). Each arch comprised four wrought-iron trussed girders. At that time only multiple-truss girders, arches, or suspension bridges were used for spans of similar magnitude. A multiple-truss bridge was rejected on æsthetical grounds, and a suspension bridge was excluded because of its cost.
The only other reasonable type was an arch bridge, although it was acknowledged that in this case considerable horizontal thrusts would be transmitted to the ground because of the eccentric loads on the foundations. A hingeless arch is sensitive to horizontal and vertical displacements, but because of the great depth to the foundations it was assumed that the displacements would be negligible. The great depth was required because of the adverse stratification of the ground and because scour and erosion were expected. The upper strata of silty sand and silt could not withstand the pressures, which could be resisted only by a stratum of stiff grey silty clay at 56 ft. below O.D.

Consequently the foundations of the first and second piers, which were on either side of the river-bed at the time of construction, took the form of pneumatic caissons sunk to a depth of 58 ft. 6 in., whereas the other piers in the flood area or on the bank were supported on piles whose points were at about the same level as the bottom of the caissons. Borings made just prior to the reconstruction of the bridge in 1942 and again in 1947 disclosed that the water and colloid content of the silty clay diminishes from the right bank towards the left bank. The consistency index was from 55 to 85 and the plasticity index from 40 to 20, whereas the magnitude of the pressures is greater on the right bank. The pressure due to the most unfavourable loading condition under the edge of the first pier might have been 9 tons per square foot and under the edge of the second pier 13·3 tons per square foot. Other physical characteristics of the ground forming this bearing stratum were: void ratio, 0·5; cohesion, 0·8 ton per square foot; angle of internal friction, 3 deg.; moisture content, 29 per cent.

To comply with navigational, æsthetical, and town-planning requirements flat arches had to be provided, and the ratio of rise to span adopted was between 1:11·5 and 1:14; such arches transmitted considerable horizontal thrust to their supports. As adjacent spans were unequal, the unbalanced horizontal thrust was considerable. The horizontal thrust at the first pier amounted to 1000 tons due to dead load and might be increased by the moving load by 750 tons. At the second pier the corresponding thrusts were 450 tons and 590 tons respectively. Although the average pressures were moderate and varied from 3·7 tons to 5·2 tons per square foot, the pressures under the edges of these piers were actually as much as 16 tons and 8·1 tons per square foot as shown in Fig. 13.

Naturally this condition could not exist for ever without serious consequences. No defect was observed for decades, probably because the consolidation of the clay progressed very slowly, and because the passive resistance developed on the sides of the deeply-embedded piers prevented appreciable inclination or displacement of the supports. In course of time, however, the river became wider and deeper because erosion of the loose sand and silt of the river-bed resulted in dangerous scouring around the second pier. This change in the flow of the river was started by the construction, just upstream of the bridge, of a quay which later had to be protected by some groynes which in turn changed the direction of the flow of the river and increased its velocity. The cross-sectional area of the watercourse at low water had increased by about 50 per cent. by the year 1942. The scour around the second pier caused it to move, and also caused some deformation of the arch of the longest span, which was noticed in the year 1922. The vertical displacement of the crown was found to be about 8 in. and the increase of the span 3½ in.
The first remedial measures included decreasing the dead weight by replacing
the stone-block paving by wooden blocks, and cessation of tramway traffic. The
steelwork was strengthened and stiffened so as to be better able to resist further
displacement of the supports. These measures did not, however, prevent further
displacement. With the progressive deepening of the effect of scour around the
second pier the displacements increased and extended the adjoining span and, to a
lesser extent, the other two spans. By 1942 the vertical displacement of the crown
of the first span was 16 in. and of the second span 19 in. The first span had
increased in length by 6 in., the second span decreased by 4 in., the third span
increased by 3 in., and the fourth span increased by 6½ in. The embedment of
the second pier, which was originally 56 ft., had decreased as a consequence of
scour to 6 ft. 7 in., so that the passive resistance which reduced the overturning
effect was practically non-existent. The arches had therefore to support each
other elastically and transmit the unbalanced horizontal forces to the piers
unaffected by scour and which were safely supported on the unimpaired piled
foundations and by the considerable lateral support afforded by the passive
resistance on their sides.

Before remedial works and the stabilisation of the second pier could be
carried out, further misfortune befell this bridge, for it became a victim of the
war. The longest span was destroyed, whereupon the second pier was subjected
to an unbalanced horizontal force of 1050 tons from the second span. The
eroded foundation was unable to resist this large overturning moment and, after
a slow rotational movement, it fell on its unsupported side and caused the collapse
of the steel superstructure of the second span. The movement lasted for two
days, thereby demonstrating the slow propagation of plastic zones in the stiff
grey clay. The third pier was now subjected to an unbalanced horizontal force
of 1050 tons and also started to move, but after a horizontal displacement of 6½ in.
sufficient passive resistance of the earth developed on the deeply embedded faces
to counteract the overturning effect. The masonry pier in this condition acted
as a vertical cantilever, thus demonstrating the good quality of its construction,
and, by resisting the bending effect, prevented the collapse of the third span.
The fourth and fifth piers were displaced but to a lesser degree.

Because of these experiences, the bridge was not reconstructed as an arch
but as a bow-string girder over the larger span and a continuous beam over the
other three spans (Fig. 13), imposing vertical forces only on the ground. The
second pier was reconstructed on a pneumatic caisson sunk to 66 ft. below O.D.
and nearer the new bank of the river, thus disturbing the flow less and providing
a considerably greater cross-sectional area for the river. The pier is also less
exposed to erosion and scour. It is interesting to note that the abutments, which
are now relieved of the horizontal thrust of the arches, moved in a different
direction, namely outwardly, due to the elastic rebound of the ground. This
displacement is about 1½ in., which agrees with the amount calculated and which
was taken into account when placing the roller-bearings of the new girders.

(21.2). HORIZONTAL FORCES ON FOUNDATIONS.—Horizontal forces have led
to trouble at the foundations of some suspension bridges. An apparently small
mistake in the design of the Elisabeth bridge over the Danube at Budapest
necessitated expensive repairs to the foundations and an unnecessary increase in
cost of about 20 per cent. Here again the fault was that the structure was not of a
type suitable to the ground conditions. The bridge comprised three spans of 145 ft., 955 ft., and 145 ft., and had at the time (1903) the largest span of any suspension-chain in the world. The suspension-chains, which must be anchored in the ground on the banks of the river, subject the ground to considerable tensile forces. Rock and other ground having high resistance to tension and shearing are suitable for this purpose, but in this case the anchorages were in relatively hard marl on the right bank and in dense gravel on the left bank. The anchorages, which were large masonry abutment blocks (Fig. 14), were obviously uneconomical because they did not utilise their structural strength. The horizontal forces should be resisted by friction on the faces in contact with the ground. Therefore neither the strength of the ground nor that of the anchor-blocks was economically utilised, because the angle of sliding friction is always less than the angle of internal friction, which is a component of the shearing resistance, and the pressures due to its own weight are much less than the strength of any masonry or concrete. This form of anchorage necessitated two anchor-blocks with a volume of 33,000 cu. yd. on each bank to provide sufficient weight to resist by friction the horizontal pull of the chains. The frictional resistance of the ground was increased by the formation of ridges on the underside of the masonry.

After the completion of the substructure and the erection of the stiffening girders and suspension chains, and while the deck was being constructed, it was observed that the right-hand anchor-block was slowly moving. Construction was immediately stopped. From vertical shafts bored through the block down to the marl, it was found that the sliding was taking place along a bituminous layer placed 3 ft. 4 in. above the bottom of the anchorage-chamber. The layer of bitumen was inserted to ensure that the anchorage-chamber would be perfectly watertight. This was a practical object, although of very minor importance compared with the unexpected consequences, for it reduced the frictional resistance at the joint so formed and consequently the resistance of the entire anchorage. Asphalt is deformed slowly under continuous pressure but the deformation is accelerated if it is subjected to high temperature, and this condition was produced by the existence of local thermal springs. The anchor-block moved 1 3/8 in. in a year, and movement did not cease until the horizontal force was considerably decreased by dismantling the deck.

Very elaborate measures were then taken to strengthen the anchor-blocks and to secure an ample margin of safety against future sliding. A caisson was first sunk pneumatically to a depth of 13 ft. 4 in. close to and in front of the anchor-block (Fig. 14). The caisson contained nearly 15,000 cu. yd. of concrete (partly reinforced). In addition the gap of 16 ft. 6 in. which existed between the two adjacent anchor-blocks was also filled with masonry. Furthermore, each block was loaded with a large decorative feature in the form of an obelisk having a pig-iron core weighing 550 tons. The total additional weight on the two anchor-blocks was 5600 tons. Three horizontal drifts (Fig. 14) were constructed with their centre-lines in the insulation layer, and were used at first for the purpose of investigation. They were subsequently filled with ashlar masonry and acted as dowels between the original surface of the foundation and the upper part of the anchor-blocks which had been separated by the insulation. It is seen that, in order to secure the resistance to sliding required, a very large volume of extra material had to be used.
Similar strengthening works were carried out on the other bank to prevent failure, although there were no hot-springs on that side. These remedial and precautionary measures, which were admittedly more extensive than necessary, cost about £80,000, and caused a delay of 1½ years in the completion of the bridge.

It is interesting to note that in the year 1936 similar trouble occurred during
the construction of the Reichsbrücke in Vienna. This is also a chain-suspension bridge, having spans of 220 ft., 805 ft., and 220 ft. The construction of this large structure was also started on the basis that the chains would be anchored to blocks sunk pneumatically into stiff brown clay. When the pneumatic caissons had been sunk and the erection of stiffening girders had been commenced, more detailed investigation of the ground, and test loadings in the working chamber, showed that the resistance to horizontal shearing of the varved clay subsoil was insufficient. In this case, instead of increasing the volume and contact surface of the anchor-blocks, the steel superstructure was altered by anchoring the chains in the ends of the hollow stiffening girders, thereby relieving the substructure of horizontal pull. The stiffening girders now became compression members subjected to an eccentric thrust of 7115 tons. The weight of the additional steelwork was 5250 tons, which shows clearly how much material and time can be wasted by neglecting to design a structure to suit the soil at the site; it also emphasises the need for a thorough examination of the soil before a structure is designed rather than after the work has been started.

**Unsuitable Foundations.**

It often happens that structures become defective because the foundation is unsuitable. In the present state of knowledge of the bearing capacity of soils, it is rare for a foundation to fail because of failure of the ground in shearing, but the foundation may become unserviceable owing to excessive settlements. The more frequent causes of failure are unsuitable designs and constructional methods, which are due mainly to deficient knowledge of the ground, particularly the condition of the ground-water, and to insufficient knowledge of the applicability of various methods of foundation construction. The result is that foundations have been designed which cannot be constructed economically or safely or which are impracticable. When dealing with an uncommon foundation problem there may be a temptation to design a deep foundation, such as piling, on the assumption that depth alone will solve all the problems and provide perfect safety. This assumption is, however, not always true, because in some conditions piling may even cause defects.

(22.1). **Unbalanced Horizontal Forces.**—A failure of the open-cylinder

*Fig. 15.*

Dense sandy gravel \( w = 156 \%, n = 27 \%, e = 0.317 \), \( \phi = 47^\circ 30' \)
unsuitable types of structures and foundations

The shallow-well foundation of the boiler-house of the Technical University, Budapest, in 1906 was the result of the partial unsuitability of the design of the foundation. The main walls of the building were carried on brick arches supported on shallow brick cylindrical wells of 6 ft. 8 in. diameter spaced at 13 ft. to 16 ft. centres (Fig. 15). The brick cylinders were built on the sites they were to occupy and lowered to a layer of dense sandy gravel by removing the loose material within and beneath them. Fifty years later, when the boiler-house was to be enlarged and an adjoining temporary building was demolished, cracks were seen in the walls at the corners of the boiler-house. The cracks were extending upwards towards the corner, thereby indicating differential settlement. Because the shallow cylinders were suitable foundations for the type of soil, and because other nearby buildings have shown no signs of undue settlements, it will be useful to investigate this case.

The flat arches imposed considerable horizontal as well as vertical forces on the cylinders, which, however, offer little resistance to horizontal forces. The soil near the arches has little bearing value since it was loosened and disturbed when the cylinders were built and sunk. The horizontal forces at intermediate cylinders were self-balancing, but they were unbalanced at the corner cylinders. Consequently the corner cylinders were subjected to an outward displacement as a result of which the arches bearing on the cylinders at the corners, and also the corners of the walls, were subjected to differential vertical settlement. As shown in Fig. 15, the crack due to failure in shear extended to the centre-line of the arch, that is to the position of greatest settlement. If reinforced concrete beams had been provided instead of arches, all loads would have been vertical only and no horizontal displacement would have occurred. It is surprising that the designer overlooked the fact that the lateral forces on the corner cylinders would not be countered by the forces from the adjacent arches as in the case of the other cylinders in the rows—but what has happened once can happen again. Additional cracking (Plate VI, see facing page) was caused when the cylinders were built later for the foundation of an adjoining extension of the boiler-house. This operation further loosened the ground.

It is a general experience that the construction of cylinders and the sinking of caissons cause a loosening of the ground around them and a larger area is affected when the excavation is by means of underwater dredging. This may be very serious when the level of the foundations of adjacent structures is higher than the cylinder or caisson, as the loaded ground may be displaced laterally towards the cylinder and thereby cause vertical settlement.

Excessive Bearing Pressure.

Failures due to excessive pressure on the ground are becoming increasingly unusual, mainly because of the investigation of the ground at the site and the testing of samples of soil which are now undertaken in most cases of large structures. Some examples of earlier failures of this type are described in the following.

(22.21). Designing for Average Values.—A well-known failure of a foundation is that of the grain silo at Transcona, Canada.\(^{1,2}\) * It has been ascertained recently that failure occurred when the pressure on the ground was about equal to the calculated ultimate bearing resistance of an underlying layer

* References thus \(^{(1)}\) are to the bibliography on page 140.
of plastic clay, and was essentially a shearing failure due to the design not being theoretically safe enough. The silo is 77 ft. by 195 ft. in plan and has a capacity of 1,000,000 bushels. It comprises 65 circular bins and 48 inter-bins. The foundation was a reinforced concrete raft 2 ft. thick, the underside being 12 ft. below the surface of the ground. The calculated bearing pressure of 3.27 tons per square foot was based on tests on the ground at the bottom of the excavation, and as this pressure had been used in the calculations for neighbouring structures it was used for these silos. No preliminary investigation of the subsoil was made. The site is in the basin of a glacial lake and there are glacial deposits of clay about 40 ft. thick under a 10-ft. layer of deposits of more recent origin. Below the clay there is a well-consolidated layer of sub-glacial drift about 10 ft. thick on which many of the heavier structures in the district are supported.

Construction started in 1911 and was completed in the autumn of 1913. The first sign that trouble might occur was that in the spring of 1913, when the thaw set in after heavy winter snow, the clay under an adjacent railway embankment, 30 ft. high and composed of ballast, subsided several feet and forced up waves of ground at the sides of the embankment. The trouble was remedied by driving hundreds of 60-ft. timber piles through the ballast to form a staging on which the rail-tracks were carried.

The weight of the silo was 20,000 tons, which was 42.5 per cent. of the total weight when it was filled. Filling the silo with grain started in September 1913, care being taken to distribute it uniformly. In October, when the silo contained
875,000 bushels and the pressure on the ground was 94 per cent. of the design pressure, a vertical settlement of 1 ft. occurred within an hour of movement having been detected. The structure began to tilt to the west and within twenty-four hours was at an angle of 26 deg. 53 min. from the vertical, the west side being 24 ft. below and the east side 5 ft. above the original level (Fig. 16). The uniformly-distributed load was 3.06 tons per square foot, but, allowing for a reduction of 0.72 ton per square foot because of the depth of the excavation, the net increase of pressure was 2.34 tons per square foot. The structure tilted as a monolith and there were only a few superficial cracks. Plate VII (facing page 58) shows the structure after it came to rest, which actually happened soon after its cupola had fallen off. The excellent quality of the reinforced concrete structure is shown by the fact that later it was underpinned and jacked up on new piers founded on rock. The level of the new foundation is 34 ft. below ground. The remedial works were executed in drifts below the basement of the tilted structure. The silo has been in use since 1916.

In 1952 the Division of Building Research of the Canadian National Research Council, the University of Manitoba, and Professor R. B. Peck made some borings at a distance of about 60 ft. from the silo. From examination of undisturbed samples of the clay it was determined that the average water content of successive layers of varved clay increased with their depth from 40 per cent. to about 60 per cent. and the compressive strength $q_u$ of unconfined specimens decreased from 1.1 tons to 0.65 ton per square foot, the average being 0.93 ton per square foot. The average liquid limit was found to be 105 per cent., and the plastic limit 35 per cent.; therefore the plasticity index was 70 per cent., which indicates that the clay was highly colloidal and plastic (Fig. 17). This condition

![Diagram](image-url)

**Fig. 17.**

is also demonstrated by the fact that the ratio of compressive strength in natural and in remoulded states is two. The bearing capacity $q_u$ can be determined from Professor Terzaghi’s formula, $q_u = cN_1 = \frac{1}{2}q_uN_c$, in which $N_c$, according to Professor A. W. Skempton, is $5\left(1 + \frac{B}{5L}\right)\left(1 + \frac{D}{5B}\right)$, in which $B$ and $L$ are the
breadth and length of the foundation and $D$ is the depth. Substituting the data in the foregoing,

$$N_e = 5 \left( 1 + \frac{77}{5 \times 195} \right) \left( 1 + \frac{12}{5 \times 77} \right) = 5.56,$$

and, with the average value of $q_u$, the theoretical ultimate bearing capacity should be $\frac{1}{3}(0.93 \times 5.56) = 2.57$ tons per square foot, which is slightly greater than the pressure at which failure occurred. If the least value of $q_u$ that is 0.65 ton per square foot for the lowest layer, is taken into account $q_n$ is 1.8 tons per square foot. Although Professor R. B. Peck does not assume that the lower bearing capacity of the lowest layer would be decisive in the case of a failure in shear, it is likely to have some effect upon the bearing capacity of the overlying layers. It is not good practice to base a design on average values, for a weak layer will produce concentrations of stress in a stronger layer above it. It is safer to adopt minimum values, by the application of which a lower ultimate bearing capacity is calculated and, in this case, one which is less than the actual pressure at failure. The numerical values in the foregoing provide proof of a failure in shear.

A calculation made by Mr. Karafiát (3) indicates that the rate of loading the ground might have contributed to the failure, since the ultimate bearing capacity of a soil is smaller when a load is suddenly applied than when the loading and consolidation are gradual. This phenomenon applies more particularly in the case of cohesive soils, which require a very long period before they are fully consolidated. Based on Professor Peck’s data, it has been computed (3) that the consolidated shearing strength would develop in about a year, whereas the load of grain was applied in 45 days, which is almost equivalent to a suddenly-applied load.

According to the “critical edge-stress” theory of Fröhlich-Maag, the ultimate bearing pressure at the edge of the foundation is given by $\pi D \gamma \sin \phi$ for a load applied suddenly compared with $\frac{\pi D \gamma}{\cot \phi - \left( \frac{\pi}{2} - \phi \right)}$ for progressive consolidated loading, in which $\phi$ is the angle of internal friction and $\gamma$ is the unit weight of the clay. The calculated limiting pressure at the edge is 1.86 tons per square foot for sudden loading compared with 2.25 tons per square foot for progressive consolidated loading, the difference being about 20 per cent.

(22.22). RAPID APPLICATION OF LOAD.—The effect of consolidation with the passage of time on the bearing capacity of highly-colloidal plastic clays is demonstrated by the failure of the base of an oil-tank at Frederikstad, Norway. (4) The site was on the sea-shore, and the ground comprised soft clay overlying rock at various depths. The problem was to carry large loads on a soft soil. There was an increasing demand for more tanks, and after the first tank was constructed a different method of construction was developed. The tanks were erected in place, the plates being welded from a platform floating on water inside the tank. As the height of the tank increased the platform was raised by pumping more water into the tank. In this way better welding was secured and the strength of the clay was increased because it was slowly consolidated, as the weight of the tank increased as more water was admitted. In addition some equalisation of
the differential settlements was obtained. As a consequence no failure occurred among the large number of tanks constructed, although settlements of 1 ft. 8 in. to 3 ft. 4 in. occurred.

In 1952 another tank had to be built quickly and the gradual filling with water was omitted. The diameter of the tank was 83 ft. 4 in., its capacity about 220,000 cu. ft., and its weight when empty was 550 tons. It was founded on a layer of gravel 1 ft. 2 in. thick, which was covered with a 6-in. reinforced concrete slab connected to a circumferential wall. The tank was tested by pumping about 180,000 cu. ft. of water into it during a period of thirty-five hours. Two hours later the tank tilted towards its eastern side and an upheaval of the ground occurred on the same side. The tank was emptied immediately and the greatest residual differential settlement was then 1 ft. 8 in. The greatest upheaval was about 1 ft. 4 in., and displacement extended over a semicircular area about 34 ft. wide. The total load at the time of failure was 5500 tons, which was equivalent to a uniformly-distributed pressure on the ground of 1.03 tons per square foot.

After the failure the tank was filled with water in stages with an interval of time between each stage to enable consolidation to take place slowly. The tank was put in use in 1954 and no defects had been detected five years later. The total settlement of the tank by 1957 was between 1 ft. 8 in. and 3 ft. 4 in.

<table>
<thead>
<tr>
<th>Description of soil</th>
<th>Water content in %</th>
<th>Shearing strength in tons/sq.ft.</th>
<th>Average values</th>
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<tbody>
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<td>10 20 30 40 50 60 70</td>
<td>1 2 3 4 5 6 7 8 9</td>
<td></td>
</tr>
<tr>
<td>Fine sand</td>
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<tr>
<td>Organic matter</td>
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<tr>
<td>Marine silty clay</td>
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<tr>
<td>Marine clay</td>
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</tr>
<tr>
<td>Rock</td>
<td></td>
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</tr>
</tbody>
</table>

![Diagram](image)

Fig. 18.
In 1955 investigations were made at this site by the Norwegian Geotechnical Institute. In 1949, that is three years before the construction of the tank, borings and investigations had been made to determine the undrained shearing strength of the clay by cone tests on undisturbed samples, the results of which were used as a basis for comparison with the later investigation. A cross section of the site showing the various layers of soil and the results of the laboratory investigations are shown in Fig. 18. The ground consists of an upper layer of silty clay at a depth of 3 ft. to 23 ft. below the surface of the ground, and of a lower layer of soft marine clay at depths of 23 ft. to 58 ft. 4 in., below which is rock. The water contents of the two clays indicate their very plastic condition. The plastic limits were from 0·21 to 0·28 and liquid limits from 0·40 to 0·70 and indicate the fairly colloidal character of the marine clay. The shearing strength had been determined by vane tests in 1949, before the tank was built, and in 1955 after the tank had been in use about two years. As shown by the data in Fig. 18, the original shearing strength of the upper clay decreased with the depth, whereas the shearing strength of the marine clay increased uniformly with depth. This difference may be due to variations of the water content.

However, due to the consolidating effect of the tank, by 1955 the shearing strengths were considerably increased, to a depth of about 40 ft., that is to a depth equivalent to half the width of the tank. The greatest increase was 0·185 ton per square foot, that is 100 per cent., but it is shown in Fig. 19 that the shearing strengths at the eastern side were less than those at the western side, the minimum differences being 0·257 ton and 0·25 ton per square foot. Similar variations have been observed elsewhere and, according to observations made in Norway, are due to the variable depth to the underlying rock, that is to the variable thickness of the compressible clay. The softest clay occurs where the depth to the rock is least.

The theoretical bearing capacity \( q_u \) of the ground was calculated from Professor Skempton's formula, as quoted on page 29, with the following data:

\[
B = L = 83 \text{ ft.} 4 \text{ in.; } D = 1 \text{ ft.} 8 \text{ in. for which } N_s = 6.2.
\]

With the average shearing strength at a depth \( \frac{2}{3}B \) of 0·285 ton per square foot, \( q_u = 6·2 \times 0·285 = 1·77 \) tons per square foot. Since failure occurred at a pressure of 1·03 tons per square foot, there should be theoretically a factor of safety of 1·77 ÷ 1·03 = 1·72. It is again clear that, when calculating a limiting state of equilibrium, average values must not be taken into account; only minimum values should be considered, since failure takes place along the planes of least resistance.

As is shown in Fig. 19, the shearing strength of the clay diminished towards the eastern side of the tank, and by plotting " contour lines " of strength it is shown that the softest core, having a strength of 1·90 tons per square foot, extended to a distance of about half the diameter of the tank, which coincides approximately with the area of the upheaval outside the tank at failure. This indicates the probability of a local failure. Applying Professor Skempton's formula to a width of 33 ft. 4 in. only, then \( N_s = 5·6 \). For a minimum shearing strength of 0·19 ton per square foot, the local maximum bearing capacity is 1·12 tons per square foot and the factor of safety is about 1·12 ÷ 1·07 = 1·05, which is about the theoretical condition for failure, especially when the diminishing-resistance effect of sudden loading is considered. That there was a local failure is indicated also by the fact that tilting occurred towards the eastern
side of the tank. When the shearing strength was subsequently doubled, ample safety was obtained in the least favourable condition. This example demonstrates the importance of preliminary consolidation, and also shows that when the shearing strength of the ground is not uniform there is a possibility of local failure.

(22.23). **Temporary Supports.**—It is nowadays rare for a foundation to fail as a result of the bearing capacity of the soil not being properly calculated, but it occasionally occurs in the case of foundations of temporary supports. Failure of the ground below temporary supports or other construction may occur due to excessive pressure and, even when the bearing capacity is not exceeded, to excessive settlements. The supports of the shuttering for concrete structures are liable to such a danger from such causes as the concrete not being placed

![Diagram 1](image1.png)

![Diagram 2](image2.png)

**Fig. 20.**

at a uniform rate (with consequent temporary overloading), one or more of the supports being temporarily removed, and the moisture from the concrete percolating into the ground to such an extent that the compressibility of the soil is increased, resulting in large local settlements.

An example of such a failure is that of a four-story flat-slab structure which collapsed during the placing of concrete for the first floor.\(^5\) This happened because the bases of the outer posts of the scaffold supporting the shuttering were not founded on the ground, and the weight of concrete caused excessive pressure on the ground and differential settlement of the bases (*Fig. 20* and *Plate VIII*, facing page 50). Consequently adjacent posts were overloaded by about 300 per cent., with the result that they broke and the concrete slabs, not being able to span across the two bays, also collapsed. In this case the settlement of the bases of the exterior posts was increased by the fact that an adjacent timbered excavation was deepened and the lowering of the ground-water which ensued led
to additional horizontal and vertical displacement of the bases. Both of these causes of the accident should have been foreseen.

**Excessive Safety Measures.**

Failures may also result when an excessive degree of safety is sought, as is illustrated by the following examples:

(22.37). **SUPERFLUOUS PILES.**—The ground below the abutments of a small reinforced concrete road bridge over Tápió Creek, Hungary, was washed away during a flood and the bridge collapsed. The failure was due partly to the foundations being at insufficient depth and partly to the distance between the abutments being too small to allow adequate flow of water without excessive velocity. The bridge was a reinforced concrete slab structure having a clear span of 16 ft. 8 in. The ground was silt and saturated sand, which was eroded to a depth of several feet, upstream and downstream from the bridge, due to the restriction of the water-course at the bridge.

A new bridge was designed having a clear span of 40 ft. To ensure that the new abutments would be safe from erosion, their foundations were to be constructed within a sheet-pile cofferdam to a depth of 20 ft. below the water level. In addition they were to be built on timber piles 16 ft. 8 in. long spaced at 3-ft. centres and driven from the bottom of the excavation. Work started with the driving of 24-in. timber sheet piles 12 ft. long for the cofferdams, which was achieved without difficulty, but the excavation within the cofferdam, for which pumping was necessary, was less easy in the lower waterlogged sand. The liquefied soil seeped through every defective joint in the piling and traces of upward surging appeared at the bottom of the excavation. These difficulties were overcome by constant supervision, by caulking of the joints, and by installing drains in the excavation. The high lateral pressure of the liquefied sand, which had a density of 125 lb. per cubic foot, caused serious deformations of the sheet piling, which resulted in gaps opening between the piles. The flowing soil formed cavities behind the sheet piling and led to uneven horizontal displacement of some of the piles, which in turn resulted in further opening of the joints. At this stage it was obviously impossible to proceed.

The driving of the foundation piles was, however, started within the small foundation pit (Fig. 21), with the result that the vibration caused the subsoil to liquefy entirely and opened further the joints in the sheet piling and loosened the struts. In addition the driving of a pile caused piles already driven to rise, no doubt because the liquefied and practically incompressible soil was squeezed by the pile being driven, and this lateral pressure could not be relieved by lateral expansion because of the presence of the sheet piling. Therefore the soil moved upwards with every blow of the hammer and the piles were also carried upwards by as much as 1 ft. to 2 ft. It was eventually admitted that the work could not be accomplished in this way, because not only was there danger of the collapse of the cofferdam, but the continued liquefaction of the soil to greater depths would deprive it of its bearing capacity. Consequently pumping was suspended, the ground-water was allowed to resume its proper level, and longer piles were driven in the inundated excavation. The piles were then cut off under water at the level of the foundation. The elimination of upward seepage and the weight of the static ground-water effected a certain consolidation of the disturbed ground
in the bottom of the excavation, on which a layer of concrete was deposited, whereupon it was possible to place the concrete in the foundation.

The main reason for the trouble, expense, and loss of time was the provision
of the piles, which were entirely superfluous, for the sake of "safety". It should have been known that saturated sand, when enclosed in a sufficiently deep cofferdam to prevent lateral yielding and displacement, would afford sufficient bearing capacity, and that the sheet piling, if left in position, would provide ample protection against scour. The excavation within the cofferdam should have been done by dredging and the concrete for the foundation should have been deposited under water. The attempt to obtain increased safety by means of the piles did more harm than good.

(22.32). Unnecessary Piling.—An unnecessary piled foundation was also provided for a grain silo at Szolnok, Hungary. A similar silo was constructed some years earlier on a solid concrete raft foundation on the alluvial deposits of the River Tisza. Because of the compressible nature of the plastic and comparatively thick layer of green clay, it was natural that the structure settled to the extent of 12 in. to 16 in. after a long period of consolidation. Owing to the rigidity of the raft the settlement was uniform, and no cracks or other defects were experienced in the building and its operation was not disturbed.

In the case of the second and neighbouring silo an attempt was made to reduce the settlement considerably by providing piles under the raft. Cast-in-place rammed piles 16 in. in diameter and 40 ft. long at 4 ft. 6 in. centres were specified. The piles were to be arranged in ten parallel rows (Fig. 22), and driving was started in the excavation from a level 15 ft. below ground and from 4 ft. to 5 ft. above the level of the ground-water.

Here, also, it was found that as more piles were driven those driven previously moved upwards. This would have been expected if the geology of the site (Fig. 23) had been considered. The piles were to be driven through the various layers of brown, green, and grey clays and transmit the load to the underlying brown silt. The penetration of the intermediate layers of clay caused a considerable squeezing of these rather plastic soils, the natural water content of which was mostly above the plastic limit. The plastic behaviour was due not only to the relatively high water content but also to the fairly high plasticity index and liquid limit, which indicated a highly colloidal state. The vibrations due to the driving, combined with the liquefying effect of the pore-water which was squeezed out, produced liquefaction of the clay similar to that experienced in quicksand in the previous example. The relatively low permeability of the clay was an important factor in the liquefaction, because the high pressures set up in the clay by the pile tended to squeeze out the water from the pores, but owing to the low permeability this water could not percolate through the particles and was therefore trapped close to the pile, thereby increasing the water content to a degree causing liquefaction. The lateral pressure resulted in upheaval of the surface, and this displacement caused fracturing of the concrete piles already driven. The spacing of the rammed piles was increased to 8 ft. 4 in., and bored piles of 13 in. diameter were inserted between them. A hole was bored down the centre of each of the fractured piles into which grout was injected to seal the cracks. This arrangement proved successful and the work proceeded without difficulty.

The settlement of the second silo was reduced by 6 in. to 8 in. by the provision of the piles, but at an additional and unnecessary cost because the greater but uniform settlement had done no harm to the first silo.

(22.33). Unsafe Piled Foundation.—Another example of the fallacy of
the belief that a piled foundation is necessarily safe is the case of a reinforced concrete silo at Portland City, U.S.A. The structure (Fig. 24, page 40) comprises storage silos, a two-story track-shed, and an operating house which contains 79 bins and is 78 ft. high. The silos cover an area of 136 ft. by 105 ft., are 100 ft. high, and comprise 99 bins, having a total capacity of 1,053,000 bushels. The structure is carried on a concrete raft 3 ft. thick supported on about 4300 timber piles, each of which was assumed to carry 25 tons when fully loaded. The site was the low-lying bank of a river and the general level was about 10 ft. above low water. The structure was at a distance of about 300 ft. from the river, and adjacent to a new wharf. The level of the ground had been raised to 32 ft. above low water by filling pumped from the river. When construction started the original bank was excavated to 4 ft. 6 in. above low water and the piles, which were 40 ft. to 45 ft. in length, were driven at 2 ft. 6 in. centres over the entire area. The tops of the piles were embedded about 6 in. in the overlying concrete raft.
Pedestals of various heights were erected on the raft to carry the structure. Under the storage silos and most of the operating house the height of the pedestals is about 16 ft. and the space so formed between the raft and the floors of the structures was filled with sand placed hydraulically. (In the construction of multiple-story buildings on sand in North Africa, a sub-basement was provided in which sand could be stored, and moved as necessary to the high side to counteract settlements by putting extra weight on the high side.)

When the storage silos had been constructed to about two-thirds of their height and the operating house had been erected to a height of 12 ft. to 15 ft. above the ground, a settlement was observed. This settlement began when the sand filling was pumped on to the raft, but the placing of concrete for the superstructure was not suspended until the greatest settlement was nearly 2 ft. The greatest settlement was under the storage silos, where it was from 9 in. to 2 ft. Under the operating house the settlement was from 7\(\frac{1}{2}\) in. to 1 ft. 9 in. and was least under the track-shed where it was from 2\(\frac{1}{4}\) in. to 3\(\frac{1}{2}\) in.

Fourteen test piles 50 ft. to 100 ft. long had been driven at various parts of the site and loaded before the work was started, and some of them were tested with loads up to 40 tons. At the time of the subsidence the load on any pile did not exceed 15 tons under the storage silos and 9 tons under the operating house. Because the final set of many of the piles was excessive, more piles were driven on the assumption that if the bearing capacity of some piles was deficient the addition of more piles would provide the total bearing capacity required. This concept was incorrect in this case because of the close spacing of the piles and because the length of the piles was far less than the width of the structure. Also, since the piles supported by friction could not transmit the loads to a lower stratum of higher bearing capacity, the foundation acted as a raft, the effective foundation plane being at the level of the points of the piles where the bearing capacity of the semi-fluid silty sand was for practical purposes not much better than at a higher level. The sand was quite incapable of carrying the design pressure of 3-8 tons per square foot, and in addition the closeness of the piles increased the liquefaction of the soft soil and further reduced its bearing capacity. The deduction from the test loading was also erroneous, because in the circumstances the group of piles could not carry safely even \(\frac{5}{8}\) of the safe load on an isolated pile, because in addition to the well-known action of a group of piles supported by friction, the short piles at close spacing could not act as piles at all.

The first remedial measure was to construct a form of cofferdam (Figs. 24 and 25, pages 40 and 41) comprising 2684 piles in several rows around the foundation with a view to their acting as a sheet-piling to prevent lateral movement of the liquefied sand under load. The first row was driven about 25 ft. from the structure, and succeeding rows were driven between the first row and the structure, thus gradually nearing the original foundation. The spacing of the piles was still 2 ft. 6 in., but they were driven 15 ft. to 20 ft. deeper than the foundation piles, the piles in the first four or six outer rows being driven 5 ft. deeper than the remainder. The second remedial measure was to remove the sand filling, which resulted in a reduction of 20 to 23 per cent. of the total load. By these means further subsidence was prevented.
Foundations at Different Levels.

(22.4). Unequal Bearing Capacities.—An example of defects arising from the provision of foundations of unequal bearing capacities at different depths below the same building is the case of a store and several workshops at a wharf built in the year 1952 at Dunapentele (Hungary) on the River Danube. The wharf is on the sloping shore of an island and the buildings were erected on filling behind the wharf. The foundations of the structural frames comprised shallow cylinders sunk about 20 ft. to 22 ft. below the level of the filling into the original subsoil of dense coarse sand (Fig. 26). The exterior walls between the columns were carried on continuous reinforced concrete beams, whereas the interior partition walls were provided with strip foundations bearing on the unconsolidated filling at a depth of 2 ft. to 3 ft. Consequently serious unequal settlement was bound to occur. The settlement was least under the parts of the partition walls close to the frames, since part of the load was transmitted by friction to the foundations of the frames, and lateral displacement of the loose soil below the shallow strip foundations was hindered by these deeper foundations. These reliefs operated with diminishing effect towards the middle of each bay, where the settlement increased to 6 in. As a result all the partition walls cracked severely, the direction of the cracks being indicated on the drawing. The cracks were widest above the doors and windows, where they were up to $\frac{1}{4}$ in. wide (Plate IX, facing page 58), but these were of no structural importance. The rate of settlement was increased by the rapid and frequent variation of the level of the ground-water due to the changes of the level of the water in the river. This action resulted in a continuous and rapid consolidation of the filling, and may have washed the finer particles out of the soil leading to smaller collapses of the soil structure. This example shows that the foundation of a building must be dealt with in its entirety, since undue settlement of secondary parts may still lead to objectionable defects even if they do not affect the stability of the entire structure. (Reference should be made to Example 41.3 on page 120.)

Excavations Deeper than Adjacent Foundations.

A frequent cause of failure is the excavation for foundations close to, and at levels different from, existing foundations. Such circumstances frequently arise when the structural design and general planning of an industrial building are not dealt with at the same time and carefully related to each other. For example, the exact position of a machine or particulars of its foundation or other details may not be known when the foundations of the main structure are constructed, or if they are known they may be altered when the work has started. In such cases defects may occur because the foundations for the machines and trenches for drains or other services may have to be deeper than adjacent foundations. Even if the exact position and levels of the foundations and ducts are known, it is important that the work be done in the correct order. The deepest foundations should be constructed first, followed successively by the next deepest. It is very important that the density of the backfill should not be less than the original density of the soil. This result is obtainable with modern compacting equipment, but if such plant is not available the spaces between sides of the
foundation structures and the sides of the excavations should be filled with lean concrete.

(22.5). ADJACENT EXCAVATIONS.—An example of the detrimental effects of laying deep drains near an existing structure is the case of the collapse of a church at Tvrdonice, in Czechoslovakia. This church was built some years ago and had a heavy vaulted roof. Soon after the church was built cracks occurred in the building. Since landslips often occurred in the district it was assumed that the cracks were due to this cause, and it was decided that land drains should be provided around the structure to prevent water percolating below the strip foundations. The drains were laid in narrow trenches 20 ft. deep close to the walls of the church and extended below the level of the foundations (Fig. 27).

![Diagram of a church with foundations and drainage system](image)

**Fig. 27.**

The bottom of the trenches was packed with crushed stone and rock and, in order not to fill the voids in this material and to keep them free for the passage of water, a layer of straw was placed on the top of the stone. In order to assist the water to reach the drain, the earth filling in the remainder of the trench was not compacted. Soon after this work was finished the church collapsed. The lateral pressure arising from the considerable vertical pressure under the strip foundations pushed in the sides of the trenches, the intrusion of the earth into the voids of the stone packing being facilitated by its loose condition. The lateral displacement of the soil resulted in settlement below the foundations of such magnitude and variation that it resulted in the collapse of the building, which was already in a defective condition.

The reason for the initial settlement was that the church had been built on an ancient and forgotten churchyard, and the foundations of each of the exterior walls coincided with a row of ancient tombs. If this information had been
obtained by a more extensive preliminary investigation of the site the foundations would have been at a lower level, the first settlement would not have occurred, and consequently the wrong remedy that resulted in total collapse would not have been applied.

**Foundations of Different Types under the Same Building.**

Failure may occur because foundations of different types are provided for different parts of the same structure, and also because of variations in the bearing capacity of the soil under the same building. A frequent cause of failure of this type may occur with piled foundations if the lengths of the piles are not properly specified. The piles may be either too short or too long, and either case may cause failures. The worst condition is when piles of different lengths are provided below the same structure.

(23.1). **DIFFERENT LENGTHS OF PILES.—An example is the failure of the foundation of the maritime station at Le Havre in the year 1932.** The reinforced concrete frame of the building was supported on one side on an existing raft carried on timber piles 100 ft. long bearing on a layer of compact gravel, whereas the other side of the frame was supported on a new piled foundation. The new foundation comprised reinforced concrete piles 33 ft. long in groups of eight to thirteen, which were driven into a layer of sand and gravel 8 ft. thick overlying clay 60 ft. thick (*Fig. 28*), beneath which compact sandy gravel extends to a great depth. The new piles settled only \( \frac{1}{4} \) in. to \( \frac{1}{2} \) in. under loads of 60 to 100 tons, from which it was concluded that the bearing capacity of the upper layer of sandy gravel was sufficient to provide a satisfactory foundation, and the cost of providing piles of the same length as the timber piles (100 ft.) was unnecessary. Piles 33 ft. long and each designed to have a working load of 42 tons were therefore provided.

It was a matter of surprise, therefore, when considerable settlement of the building occurred during construction, when the load did not exceed 10 tons on each pile. Settlements occurred at the rate of \( \frac{1}{2} \) in. to \( \frac{3}{4} \) in. per month, and after two years the total settlement of some of the piles was about 1 ft. 2 in. under the central part of the building. The great differential settlement caused serious cracks and other defects of the building. Investigation showed that the settlement was due to the compression of the soft clay to which the load was transmitted through the upper layer of sandy gravel. This effect was unlikely to result from the small bulbs of pressure under single test piles, but the groups of piles applied a pressure which was almost uniformly distributed over a large area of the gravel and thence to the clay. The dispersion of the pressure through the relatively thin layer of gravel was limited, and considerable pressures were transmitted to the compressible clay. Compression tests on undisturbed samples of the clay demonstrated its great compressibility and its fairly high water content.

The remedial measures incorporated one of the first applications of M. Freyssinet's methods of prestressing and curing concrete.\(^{(6)}\) The columns of the original building were supported on reinforced concrete caps 15 ft. by 11 ft. in plan and 4 ft. 8 in. thick on the groups of piles. Piles 100 ft. long were inserted between the pile-caps and extended down to the compact gravel below the clay. Because of the limited headroom, and to avoid vibration, cast-in-place piles of the pressure type were used. It was necessary to provide a support sufficiently
resistant to transmit the loads on the columns to the longer piles, and also to provide a reaction to the thrust necessary to press the piles into the ground. This problem was solved by placing lightly-reinforced concrete beams between the pile-caps (Fig. 28) before inserting the new piles. The beams were then prestressed by steel bars anchored in the pile-caps, thus producing a highly-resistant horizontal beam without disturbing the existing structure. The piles were inserted through holes formed in the beams, which provided also the resistance to the jacks by means of which the piles were forced into the ground. The new cylindrical reinforced concrete piles are of 2 ft. external diameter and 15 in. internal diameter. The concrete was placed continuously in sliding shuttering. Each 20-ft. section was cured under moist conditions and allowed to harden before being forced into the ground by a force of 320 tons.

**Bearing Strata of Variable Thicknesses.**

Some failures can be attributed to the ground under a structure being of different bearing capacity for different parts of the foundation, as is exemplified in the following.

(23.21). **VARYING THICKNESS OF CLAY STRATUM.—**A cylindrical steel tank of 125 ft. diameter and 32 ft. deep (Fig. 29), in Essex, England, tilted when it was first filled. The tank bears on a 6-in. reinforced concrete raft of 126 ft. diameter, which is thicker around the perimeter where it is supported on a shallow ring-beam of plain concrete. If the weight of the tank and its contents were distributed uniformly over the entire area of the foundation the pressure on the ground would be 0.06 ton per square foot when the tank was empty and 0.95 ton per square foot when it was full. Immediately below the structure there is a layer of soft clay which is a few inches thick to the north and 7 ft. thick to the south and which overlies compact gravel. The construction of the tank began during the winter of 1945–6, and when it was filled in 1947 large differential settlements developed and interfered seriously with the working of the plant. The settlement was greater where the layer of clay was thicker, and the differential movement caused tilting of the tank and fracture of the raft. As a consequence the plant superstructure was dismantled, the tank emptied, and remedial measures were started.

Below the site there is an alluvial deposit from an adjacent river. The deposit, which is about 55 ft. deep, comprises a thin layer of post-glacial brown
clay overlying glacial sand and gravel, below which is a layer of blue clay about 10 ft. thick. The settlement varied from 4 in. at the northern edge to about 4 in. at the southern edge, and the raft fractured in two places. A record of the settlements and a contour of the surface of the underlying gravel are shown in Fig. 30. It is of interest to note that under uniform loading around the edge of

![Diagram](image)

Fig. 30.

the structure the settlement was almost directly proportional to the thickness of the layer of clay; the settlements therefore correspond to the contour of the surface of the underlying layer of compact gravel. The diagram also shows a peak in the surface of the gravel which was responsible for the fracture of the raft. The load-settlement-time records at three points on the perimeter corresponding to the least, the mean, and the greatest thicknesses of the layer of clay are shown in Fig. 31 and extend from the time when the tank was first filled. Settlement increased very rapidly when the tank was filled, after which it
continued to increase but at a slower rate. When the tank was emptied some recovery was observed, but a permanent settlement remained. The remedial works comprised underpinning the southern part of the ring-beam with reinforced concrete beams supported at their outer ends on reinforced concrete piles 10 in. square driven about 5 ft. into the gravel.

The brown clay was uniform in nature and had a plastic limit of 20 per cent. and a liquid limit of 75 per cent. The water content varied from 35 per cent. near the surface to a maximum of 50 per cent. at the water-table near the surface of the underlying gravel; the clay at a lower level was therefore softer than that nearer the surface. The shearing strength of the clay at the surface was 0.25 ton per square foot but was less at a depth of 3 ft., below which it was constant. The strength of samples taken at the perimeter of the raft was about 25 per cent. greater than that of samples taken some distance away; this difference may be accounted for by the consolidation of the soft clay under the foundation (see 22.22). Calculations showed that the softened clay at the lower levels under the southern part of the tank were overstressed in shearing by nearly 100 per cent. It should be noted that when the pressure is small in relation to the bearing capacity ordinary calculations of settlement apply, but relatively high pressures lead to local overstressing of the soil and, while not causing complete failure, produce large additional settlements. This condition is transitional between the safe and failing state of the ground, and in this case it contributed to the magnitude of the tilting.

(23.22). **Gravel overlying clay of varying thicknesses.**—Another case occurred in connection with the foundation of the main building of a public baths

![Diagram](image)

**Fig. 32.**

*(See plan in Fig. 33 for position of X-X)*

in a suburb of Budapest. The building, the plan of which is a tee, is founded on shallow strip footings extending to just below the frost-line, where the ground
is a uniform sandy and silty gravel. The original design provided for a deeper foundation extending to the underlying limestone, but in order to avoid the need to de-water the deeper excavations this design was not adopted but it was decided that the foundation plane be above the level of the ground-water. The good quality of the upper layer of sandy gravel was misleading, and the footings were designed for a permissible pressure of 2.2 tons per square foot at a depth not exceeding 3 ft. After two to three years serious cracks had occurred throughout the building.

An investigation showed that the thickness of the layer of sandy gravel did not exceed 4 ft. to 6 ft. and below it there was a layer of very compressible organic clay and silt varying in thickness from 4 in. to 3 ft. The limestone was at a depth of 5 ft. at one side of the building and 8 ft. 4 in. at the other (Fig. 32). It was evident that the cracks were caused by differential settlement due to the compression of the layer of clay, which had a high water content and a ratio of voids exceeding unity. The plasticity index of the clay was not great, being between 20 and 30 per cent., which is explained by the relatively low organic content; the loss on ignition was up to 10 per cent. The high water content was accounted for by the high level of the ground-water. The high bearing capacity of a good upper layer was useless since it was of insufficient thickness, which was further reduced by the excavation to the level of the foundations, thereby restricting the dispersion of pressure on to the underlying soft layer.

The existence of the limestone was also a disadvantage since it was too rigid to spread the concentration of pressure in the soft layer above. Calculations showed that a pressure of about 1.65 tons per square foot was transmitted to the surface of the soft organic clay, which was far in excess of its bearing capacity and resulted in lateral expansion which was not resisted by the ground at the sides because of its high void ratio and consequent high degree of compressibility. The contours of equal settlements computed from the actual measurement of the settlement (Fig. 33) indicate that differential settlement was caused by the varying thickness of the compressible organic clay on the sloping surface of the hard limestone. If the foundation had been supported directly on the rock, as was originally intended, no trouble would have been experienced. The consolidation diagrams (Fig. 33) show the difference of the settlements at points a, b, and c, and indicate that the rate of consolidation does decrease but is continuing uniformly and is likely to do so for several years. The compression modulus of the clay is about 16.5 tons per square foot and the estimated total compression may be

\[ s = \frac{pd}{M} = \frac{1.65 \times 3 \text{ ft.}}{1.65 \text{ tons per sq. ft.}} = 3\frac{1}{2} \text{ in.} \]

Considering that the yearly rate is not more than \( \frac{3}{8} \) in., and that up to the end of the period of observation the settlement was only 50 per cent. of the total settlement expected, the further duration of the period of consolidation of this thin layer is estimated to be \( \frac{1}{2} \times 3\frac{1}{2} \), that is about five years.

**Structures of Non-uniform Weight.**

Differential settlement may also be caused by the weight of a building being distributed non-uniformly on a foundation structure of uniform rigidity on ground of uniform bearing capacity.
Contour lines of equal settlements

Test boring

Scale 5 10 15

0.95
0.5
0.25
0.10

Settlement records started only after beginning of consolidation.

FIG. 33.

(See Fig. 32 for Section X-X)
(23.3) **Unequal Loading of Piles.**—An example in Holland (10) illustrates this case. A factory beside the River Maas at Zwyndrecht was erected in the year 1916 between the river bank and a dyke. The site had been reclaimed by sand filling deposited hydraulically to a depth of 15 ft. The building, which was a steel-frame structure with brick walls, was for the production of margarine, and comprised an oil mill, a refinery, and heavy machinery and tanks, and was about 60 ft. high (Fig. 34). The building was erected during the war when there was a need to construct it quickly and cheaply. As a consequence it was decided, contrary to the usual practice on such ground, not to excavate the sand filling but to drive piles through it. The pile-driving frame available could deal with piles up to 66 ft. long only. Test piles showed that for the last thirty blows the penetration was 1 ft., and on this basis it was calculated that they would carry 50 tons each. The average load proposed was only 5 tons per pile, providing a factor of safety of ten, which was considered to be ample. Piles 66 ft. long were driven into a layer of fine sand about 51 ft. below ground level, and it was considered unnecessary to drive them farther into a layer of coarse sand and gravel which was about 17 ft. deeper. On the other hand it was impossible to drive them deeper because the resistance was so great that the tops of the piles were crushed. The factory was completed within one year, but four years later the part of the building containing the heavy plant had settled seriously.

Subsequent borings showed that there was a top layer 15 ft. thick of sand filling, below which was a layer about 43 ft. thick of peat mixed with some clay over-lying a layer of fine sand 17 ft. thick, and below this was compact sand and
gravel. The resistance of the piles was considered to be due to the frictional resistances of the sand filling, the peaty clay, and the lower fine sand. This, however, was incorrect, as the great compressibility of the peaty material offered little resistance to the settlement of the pile and annulled entirely the resistance of the overlying sand filling. The only soil that offered resistance was therefore the underlyin fine sand, which had not only to provide the resistances assumed to be offered by the overlying compressible layers but also to resist the pressures due to the consolidation of the upper layers, which resulted, through negative friction, in an additional load on the piles. In fact the upper layers tended to sink even when they were not subjected to load. This was actually observed in the case of some groups of piles which were intended to support an extension of the building and for the time being had to carry only the concrete floor shown by the broken lines on the left-hand side of Fig. 34b; these lightly-loaded piles sank by the same amount as those under the main building.

The records of the settlements (Fig. 34c) indicate that there was considerable differential settlement between the two longitudinal sides of the building. The largest settlements, up to 2 ft. 2 in., occurred under the tower in which there was a water tank, and where the load was up to 18 tons on each pile, whereas at most other parts of the building the load did not exceed 3 tons per pile. This failure to use the same foundation loading for parts of a building which are loaded differently caused considerable tilt, which resulted in a horizontal displacement of about 4 ft. at the top of the 82-ft. tower. The diagrams also show that under the middle of the building the settlement-line (the full line in Fig. 34c) does not correspond to the loading line (the broken line) but indicates considerable settlement where the load was moderate. This may be an indication of the redistribution of pressure which is experienced under wide or long and relatively flexible buildings.

The settlement resulted in no structural defects because of the high quality and relative flexibility of the steel frame, but since it prevented the operation of the plant the building was levelled by means of hydraulic jacks. Underpinning was carried out by fixing short steel channels to the steel columns and supporting the channels on steel beams under which the jacks were placed. The operation was commenced at the tower and was successfully executed in stages throughout the entire building.

Excessively Rigid Foundations.

It may happen that the designer of a structure specifies foundations of excessive stability, with consequent unnecessary expenditure and a possibility that the great rigidity may itself lead to trouble. It may be that a more economical design might result by adopting a suitable constructional procedure such as the introduction of temporary hinges or expansion and settlement joints, or by designing the structure so that it would not be harmed if it were subjected to small differential settlements. Perfect fixity should not be expected from a foundation on soft ground. No foundation is static, and all settle to some extent. The soil is nearly always more compressible than the material of which the structure is built, and the effect of small settlements must be considered in relation to the cost of preventing them, as is shown by the example which follows.

(24.i) Unreasonable Requirements for a Foundation.—A garage
and workshop at Salgótarján, Hungary, comprises monolithic cast-in-place reinforced concrete frames at 30 ft. centres and a roof of precast reinforced concrete purlins supporting precast reinforced concrete slabs 1 1/2 in. thick. The span of the frames is about 55 ft. and the greatest unobstructed internal height is 13 ft. 5 in. The horizontal thrusts at the hinges of the frames are counteracted by ties in channels below the floor (Fig. 35). This type of construction is generally suitable if differential settlement is expected, and the design was said to allow for a maximum differential settlement of 3/8 in. only.

Preliminary investigation showed that below 4 ft. to 6 ft. of silty humus soil there was, to a depth of 27 ft., yellow loam having an unconfined compressive strength of 0.87 ton per square foot, an angle of internal friction of 15 deg., and a cohesion of 0.326 ton per square foot. At the level of the foundations, that is at a depth of 6 ft., it was calculated that the mean relative consistency was 0.5 and the mean void ratio 0.7. Under the footings of the columns, which were 5 ft. 8 in. square, a load of 2.17 tons per square foot was considered by the soil investigators to be unsatisfactory because preliminary calculations indicated that such a load might cause settlements of up to 2 1/2 in. The general geological character of the site (a valley bottom) was taken into account as well as the compressibility of the recent sediments as indicated by the settlement of neighbouring buildings.

The top soil was therefore replaced by sandy gravel to a depth of 6 ft. This work occupied several months, during which time the excavations and the gravel already placed became soaked with rainwater. Also the degree of compaction of the sandy gravel was unsatisfactory. Test loads resulted in non-uniform settlements, and therefore the gravel already deposited was removed and replaced under constant supervision. This replacement was of doubtful value as regards uniformity and the degree of compressibility in relation to the structural stiffness required. Examination of the design calculations showed that, contrary to previous statements, no account had been taken of possible displacement of the supports. This omission was justified by the designers because the relatively tall frame with a tie was not likely to be affected by vertical differential
settlement. Horizontal displacement could cause very little additional stress in the structure, and even this stress could have been counteracted very easily by tensioning the tie to suit the actual movement due to the dead load. Alternatively, the insertion of a temporary hinge at the ridge would have prevented the structure being damaged as a result of displacement during construction. Permission was finally given to erect the frames on the prepared gravel foundation and the structure was completed, but with a delay of nearly a year and unnecessary extra cost due to an unreasonable requirement of the designer.

It is not the absolute vertical settlement which causes additional stresses in a statically-indeterminate structure, but only differential settlement, which may be at most only half of the greatest settlement and the greater part of which will take place during construction under the growing weight of the structure, and before any rigid joints should be fixed.

Incomplete Assessment of Effects of Loads.

Failures due to incomplete assessment of the effects of loads, or failure to foresee such effects, may arise from changes in the physical properties of stored materials such as earth, ore, pulps, and vegetable products, or from changes in the stresses produced in the foundations or the soil.

(25.11). Effects of Vibration.—A diesel engine was placed on a block of reinforced concrete in the basement of an existing building (Fig. 36), and this block was kept entirely separate from the foundations of the walls and columns of the structure. Just below the floor of the basement was a layer of peaty sand 10 ft. thick, and under this a layer of gravel and sand of sufficient thickness to carry the load of the engine and its foundation. Bored piles of 1 ft. 1 in. diameter were used under the foundation block, and the concrete in the peaty soil was protected against chemical attack by a coating of bituminous material. At the level of the tops of the piles a layer of lean concrete 2 in. thick was placed, followed by a slab of reinforced concrete 1 ft. 4 in. thick and a layer of cork 2 1/2 in. thick surrounded by a frame of steel.

After a time some cracks were seen in a column of the building and these increased to such an extent that a timber scaffold was built to relieve it of load. It was found that before the cracks appeared the frequency of vibration of the engine was different from that of the foundation, but the progressive horizontal displacement of the piles (which had little horizontal rigidity) resulted in the gradual compaction of the subsoil until the frequencies of the foundation and the soil coincided and compaction and settlement resulted. The remedy adopted was to underpin the column with bored piles, and the soil in the area affected was consolidated so that its frequency would be different from that of the machine. (The foregoing is from a report by H. Press in Bautenschutz, 1934, p. 134.)

(25.12). Compaction of Filling Inside a Building.—Another unexpected result of the effect of vibration on peaty soils, fresh fillings, and soils with similar properties is shown on Fig. 37. This example shows that damage can be caused by vibration although the load imposed on the soil be light. In this case light weaving machines were erected on the ground floor, which was about 3 ft. higher than the ground outside the building. The machines were mounted on thin reinforced concrete slabs which extended below the floor and rested on a thin layer of well-compacted filling. The vibration of the machines further compacted
the soil to such an extent that the wall between the columns bulged due to its lateral pressure as seen in Plate X (facing page 59).

(25.13). ADVANTAGES OF COMPACTING.—Compaction of the soil under a foundation as a result of vibration is, however, not always detrimental. Indeed, it can sometimes be advantageous after the initial troubles have been remedied, as is shown in the following example.

A turntable at a slaughterhouse in Budapest moved, after it had been in service for some years, until it scraped the surrounding masonry at one side and the gap on the other side increased by about 1½ in. due to the tilting of the support. It was found that, instead of the rainwater being collected and led away from the structure, a gravel filter had been formed some 5 ft. from the foundation of the shaft (as shown at A in Fig. 39) from which it passed into the subsoil of fine sand. The result was that the fine sand was subjected to scour and the foundation settled at this side.

A proper drainage system was immediately provided, but the underpinning of the foundation was delayed until a full investigation of the soil was made. Laboratory tests showed that the very loose sand was composed of grains of uniform size and that it had a void ratio of 0.83, a coefficient of uniformity of 2.45, and a bulk density of 97 lb. per cubic foot, and that its density could be increased by repeated loadings within permissible limits. By this means the compressibility of the sand could be decreased and its bearing capacity consequently increased. It is seen in Fig. 38, that when the sand was subjected to 800 repetitions of a load producing a stress of 2.2 tons per square foot (which corresponded to the
stress produced by the shaft of the turntable) the modulus of compression increased from 41 tons per square foot to 935 tons per square foot, the residual settlement increased by 57 per cent., and the void ratio decreased from 0.83 to 0.63. As the turntable when in use would produce 800 repetitions of a similar load in a short period it was anticipated that the machine itself would produce this degree of compaction of the sand and that no further settlement would occur with the existing foundation. The shaft of the turntable was restored to verticality by packing and no further expense was incurred. This work was undertaken four years before the time of writing and no further trouble has been experienced.

**Stress Superposition.**

(25.21). **ADDITIONAL LOADS ON EXISTING FOUNDATIONS.**—Troubles often arise when an extension to a building is partly supported by existing walls or columns. In such cases the additional load on the existing wall may cause it to settle, and, if the distance between the existing wall and the wall of the extension is small, additional stresses may be produced in the soil under the existing wall, as is indicated in Fig. 40 (see page 60). Differential settlement caused in this way can have two unfavourable results, namely, (1) the columns may settle and cause cracks in the walls and distortion of the window frames; the settlement of the walls will be greatest nearer the columns; (2) there will be differential settlement of the frames of the existing building because of the additional load of the annexe; the additional stresses in the frames will depend upon the degree of statical indeterminacy of the frames.

(25.22). **OVERLAPPING ZONES OF STRESS.**—Differential settlement also frequently occurs when an internal bearing wall transmits to its foundation a much greater load than do the outer walls (Fig. 42, see page 61). In such cases it is usual to provide a larger footing under the internal wall, with the result that the zones of stress in the soil overlap; such an arrangement makes certain that the interior wall will settle more than the exterior walls. On the other hand, if the footings of the exterior and interior walls are of the same width there will still be a greater settlement of the interior wall. The overlapping of stresses
<table>
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<th>Depth</th>
<th>Layer Description</th>
<th>Liquid Limit (LL)</th>
<th>Plastic Limit (PL)</th>
<th>Plasticity (W)</th>
</tr>
</thead>
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<td>15'-0&quot;</td>
<td>Yellow Fine Sand</td>
<td>LL = 29%</td>
<td>PL = 20%</td>
<td>W = 27%</td>
</tr>
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<td>Yellow silt</td>
<td>LL = 32%</td>
<td>PL = 10%</td>
<td>W = 28%</td>
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<tr>
<td>6'-4&quot;</td>
<td>Yellow sand with gravel</td>
<td>LL = 71%</td>
<td>PL = 26%</td>
<td>W = 19%</td>
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<tr>
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<td>Bluish-grey clay</td>
<td>LL = 43%</td>
<td>PL = 18%</td>
<td>W = 27%</td>
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<tr>
<td>5'-0&quot;</td>
<td>Yellow-grey clay</td>
<td>LL = 71%</td>
<td>PL = 26%</td>
<td>W = 19%</td>
</tr>
</tbody>
</table>

Fig. 39.

Fig. 40.

Fig. 41.

Annex added later
Additional loading
in the soil due to the closeness of footings is a common cause of differential settlement of interior and exterior walls.

Some buildings on a residential district at Ózd, Hungary, are an example of this type of differential settlement. Shortly after they were built cracks, such as that in Fig. 41 (see page 60), appeared in all the structures and were due to the greater settlement of the more heavily-loaded central wall. The soil under the foundations is stiff plastic silty clay, with a plastic limit of 18 to 22 per cent., a moisture content of 20 to 25 per cent., and a relative consistency of 0·7 to 1·0. There is a reinforced concrete ground-slab which transmits all the weight of the structure to the footings under the outer walls and the central wall. The footings under the outer walls settled by amounts up to $\frac{3}{4}$ in., whereas the settlement of the central footing was between $1\frac{1}{4}$ in. and $2\frac{1}{4}$ in., as measured by the deflection of the eaves.

(25.23). **Excessive Loads due to Filling.**—The overlapping of stresses in the soil may also be responsible for the tilting of the abutments of bridges, although this may not be the sole cause because the effect may be partly due to the unavoidable consolidation of the filling in an approach ramp and the compaction of the soil caused by traffic. Horizontal displacements of tall abutments
in the direction of a bridge may also depend on the depth of the foundation and the nature of the soil.

An example of the tilting of an abutment of a bridge is shown in Fig. 43. The bridge is over the area inundated by the River Danube at Medve, Hungary. The abutments are supported on reinforced concrete bored piles extending 27 ft. below the foundation. The filling of earth for the approaches, which are about 35 ft. high, was placed after the abutments were built, and caused a settlement of the ground below of from 10 in. to 12 in. The load from the filling was about 2 tons per square foot, and this compressed the layers of softer soil which extended to a depth of about 20 ft. to 23 ft. below ground level. The filling also settled a further 10 in. due to its own weight and to consolidation by traffic. The total settlement was therefore from 1 ft. 8 in. to 2 ft. The stresses in the ground due to the weight of the filling extended in the direction of the abutment, and increased both the lateral and vertical forces acting on the back of the abutment and on its foundation. If there were no piles it is seen in Fig. 43 that the additional load due to the filling is so much greater under the back of the abutment than

![Diagram of stress transmitted by piles]

**Fig. 43.**

under the front that the abutment would tilt in the direction of the filling instead of in a forward direction. The drawing shows the plane of stress-distribution due to the filling, the stress-distribution diagram due to the original pressures \( \sigma_1 \) and \( \sigma_2 \), and the stress-distribution diagram when to the original pressures are added the stresses \( p_1' \) and \( p_1'' \) due to the filling.

The foregoing is generally the case when an abutment is built on a shallow foundation, but in the case of a deep foundation (as in the case of the piles at the bridge at Medve) the trapezoidal stress-distribution will be transmitted to the points of the piles. Here, however, the additional stresses due to the filling
(\(\rho_1\) and \(\rho_2\)) were spread uniformly over the ground below the piles and did not change the shape of the stress-distribution diagram. In the case of this bridge the result was that the abutment tilted forward, and this is the general case when an abutment is built on a deep foundation. The forward tilt was about 1 in. at the edge of the foundation block, corresponding to a differential settlement of \(\frac{1}{2}\) in. to \(\frac{5}{8}\) in. If the spacing of the piles had not been uniform, but arranged to conform to the trapezoidal stress-distribution diagram, the differential settlement of forward and rear edges and the tilting of the abutment could have been avoided. It should also be borne in mind that the passive resistance of earth will always reduce the magnitude of forward tilting in such cases.

An abutment (Fig. 44) of the Árpád bridge in Budapest is an example of the backward tilting of a tall reinforced concrete abutment on a shallow foundation. This abutment was built in two parts, namely one on the river face which supports the bridge and transmits the vertical forces directly to piles driven down to a stiff marl, and a wall to resist the horizontal force of an access ramp about 40 ft. high. The foundation raft of this tall and slender wall is on silt at about the level of the penetration of frost, and the wall is connected to the abutment by two reinforced concrete slabs designed as strutting beams and another slab at ground level. By these means the risk of differential settlement was successfully anticipated, because the part of the abutment supporting the bridge
was practically secure against displacement, the wall against the earth filling was flexibly supported and could follow horizontal and vertical movements, and the slabs connecting the two parts acted as freely-supported beams and could resist any torsional forces. Also the wall against the filling was very rigid and was not susceptible to movement.

![Graph showing loads and change of loadings](image)

**Fig. 45.**

The curves A in *Fig. 45* show the rate of settlement from the year 1949, when the placing of the material for the embankment began, to the year 1956, when settlement ceased. The horizontal displacement towards the ramp was 2 3/8 in., which corresponds to a differential settlement of 3/4 in. between the edges of the

![Diagram of clay layer before consolidation](image)

**Fig. 46.**
Filling carried out during and after construction

Thermo-isolated retaining wall

Slag fill
Omitted piling

Clay

Omitted piling

Fig. 47.
foundation of the wall. This tilting was made harmless by the joint, and the flexible nature of the abutment prevented the appearance of cracks.

(25.24). Unequal Pressure from Varying Depths of Filling.—The abutment of a bridge (11) at Washington, U.S.A., is an example of the backward tilting of an abutment on a deep foundation. In this case the movement was due not to horizontal pressure against the abutment of the adjoining ramp formed by back-filling but to its varying height (Fig. 46). The result was that where the height of the filling was greatest it imposed on the underlying layer of clay a load greater than the load of the abutment. Also the driving of the long piles under the abutment compacted the layer of loose sand on which the filling was placed and consequently relieved the stress in the underlying clay. The plasticity of the clay was such that the deformation of its surface due to the weight of the filling extended as far as the abutment. The piles under the abutment are about 40 ft. long and 1 ft. 2 in. in diameter at the base. They were designed according to the Engineering News formula as "friction" piles with a bearing capacity of 50 tons. While the abutment was being built the approach ramp was also being constructed to a height of 17 ft. above ground level and transmitted a vertical load of about 1 ton per square foot. The soil under the abutment consists of a layer of sandy clay on a layer of softer clay. Since the sandy clay is almost incompressible it is thought that the settlement was caused by the compression of the lower layer of stiff clay as a result of the additional weight of the filling, as indicated in the diagram. The total settle-
ment expected is about 2 ft. at one end of the abutment and from 8 in. to 10 in. at the other end. In May 1949 (that is six weeks after the work was completed) these settlements were 1.14 ft. and 0.11 ft. respectively. (25.31). NEGLECTING FUTURE LOADS.—Another frequent source of failure
is neglecting to allow in the design for the possibility of the application of different loads in the future. A typical example of this kind of failure is that of a crane-way used for carrying slag from blast furnaces at a steelworks.

The steel truss carrying the cranes has a span of 39.3 ft. 4 in. and is supported at a height of about 60 ft. on rails on gantries composed of beams and columns. It was first proposed to provide piled foundations for the columns (as indicated in Fig. 47). It was, however, later decided to dispense with piles because it was thought that the driving of piles might change the cellular structure of the clay, and that the transmission of the load to the clay under the piles would result in the loss of the stress-distributing effect of the clay between the pile caps and the bottom of the piles; also it was desired to complete the work as quickly as possible. In the case of the columns in row A the foundations were constructed directly on the clay, while the foundations of the columns of row B were on a filling of slag from 13 ft. to 17 ft. thick. When the bearing values under the foundations were calculated it was wrongly assumed that when the foundations for the columns in row B were constructed the filling of slag would be about 27 ft. above the bottom of the foundation and consequently increase the bearing capacity of the subsoil by producing a considerable lateral overburden. Actually, however, the filling was placed hurriedly while the foundations and columns were being constructed and was not properly compacted; indeed the slag was sometimes placed while it was still burning and was quenched by water after it was in the excavation; and the filling was not completed until after the foundations. The result was that the load on the foundations was increased due to the effect of the friction of the slag against the columns as the material settled, and the columns in row B settled much more than those in row A. Plate XI (facing page 59) shows the distortion of the gantry only two years after it was built, when some of the columns had to be demolished and rebuilt.

Fig. 48 shows that the settlements ranged from less than $\frac{3}{10}$ in. in row A to $4\frac{3}{8}$ in. in row B. There were also differential settlements of separate foundations, as is seen by the great horizontal displacements of some of the columns; in one case this was as much as 2 ft. 5 in. It is notable that the greatest displacement was between columns 7 and 11, where there was the greatest delay in placing the filling and where the burning slag was cooled by sprinkling it with water, both of which contributed to greater frictional loads on the columns, and also to the deterioration of the soil under the footings. The nature of the soil at bore-holes Nos. 4 and 5 (Fig. 48) is shown in Fig. 49. This example demonstrates the difference in the loads on a foundation in such cases according to whether the filling has been placed and consolidated before the foundation is constructed or whether the filling is placed while the column is being built or subsequently, and emphasises the need to take into account the most unfavourable circumstances that may arise during construction.
PART III
DEFECTS AND FAILURES DUE TO DEFECTIVE EXECUTION

A sound foundation does not depend only upon a correct design and the use of suitable materials and processes. The manner of doing the work is equally important, because the procedure adopted by the contractor and the quality of workmanship will affect the strength of the construction and may often prevent the engineer's intentions from being realised. In some cases the geological formation and the ground-water conditions may be such that a foundation can be constructed economically only by the use of a special type of construction or a special method of dewatering the soil. The correct choice of methods of construction can be the result only of sufficient theoretical knowledge and experience. Faulty structures may result from unsuitable designs or poor construction (or both) or of unsatisfactory investigation of the properties of the soil, but they are more often due to carelessness. Common mistakes may be grouped under the headings of (1) unsuitable dewatering of the soil, (2) unsuitable or unsatisfactory cofferdams or other means of enclosing excavation pits, (3) unsuitable methods of construction, and (4) poor-quality work.

Unsuitable Methods of Dewatering.

One of the most common failures in dewatering the soil occurs when water is pumped directly from sumps formed in saturated fine sand (quicksand) where the ground-water is subjected to considerable pressure. Trouble due to this cause may occur in open pits, in cofferdams, and in open caissons or wells. Failure to relate the method of dewatering to the nature of the soil, and to possible changes while the work is in progress or after its completion, are particularly likely to occur in low-lying land, in waterlogged fine sand, and in deposits of silt of low permeability and with grains of uniform size and consequently a large proportion of voids.

The foundations of structures such as collecting pits, pump-houses, and culverts erected in connection with the drainage and reclamation of marshy ground are among the most difficult foundation problems. The surface water is generally collected in channels along which it flows by gravity to the lowest point in the area to be reclaimed. Such soils are generally loose and soft alluvial deposits, often of a marshy nature, and of comparatively recent formation, and in such unfavourable soils it is necessary to construct a deep suction shaft close to the foundations of a pump-house which is at a higher level and contains machines that produce vibrations of the ground. Further complications arise from the varying levels of the river into which the water is discharged, which may cause a difference in the head of water inside the suction-shaft and in the river. Due to the change of level with the seasons of the river into which the water is discharged the soil will lose water to the river when the river is low and gain water from the river when the river is high. Changes of level of the river will cause reversals of the direction of seepage of water in the soil, and may lead to scouring
of the fine-grained loose layers, particularly under the culvert carrying the water from the pump-house to the river. The strong suction effect of the pumps combined with any leakage of the suction-shaft may cause the infiltration of sand and settlement of the ground under the structures.

(31.1) LAND-DRAINAGE STRUCTURE.—In the last decades of the nineteenth century several pump-houses were erected in connection with the drainage of marshland beside the rivers Danube and Tisza. In those days less was known of the effect of different methods of removing the water and of the design of foundations in such soils. The pump-wells were excavated within timber sheet pile cofferdams. The water was pumped directly from sumps, so that semiporous layers became buoyant and the silty sands surged upwards. It is obvious that concrete placed in such pits would be porous and of poor quality generally, and that the structure would be liable to serious settlement. Also, the engine-house, the pump-shaft, and the culvert were built together without joints to allow for differential settlement. Many of these structures needed thorough repair in the 1920’s and 1930’s, and the following is given as an example.

The structures shown in Figs. 50 and 51 are on the island of Mohács, near
Karapancsa, and were built in the year 1904. The pumping-­sump is on one of
the longer sides of the engine-house and the outlet channel, which has a trapez-
oidal cross section, is on the other longer side. The foundation of the engine-
house is a concrete raft 2 ft. thick just below the surface of the ground, and on

this raft are the foundation blocks for the diesel engines and centrifugal pumps.
The floor, that is the top of the machine foundation blocks, is about 3 ft. 8 in.
below the level of the water outside, and is on a filling 5 ft. 8 in. thick on the
foundation raft.

The pumping-sump is 36 ft. 8 in. long by 23 ft. 4 in. wide, and the concrete
walls forming the sides and the concrete bottom were cast monolithically with
the adjoining wall of the engine-house. Because the bottom of the sump was
on a fairly-­impervious layer, no dewatering was necessary at the time of con-
struction, and the site was not enclosed with sheet piles.

The first noticeable defects resulted from the incorrect arrangement of the
open outlet channel, which produced a constant head of 7 ft. 8 in. of water against
the end wall of the engine-house, through which the water seeped. To prevent
this seepage through the wall of the engine-house and the consequent loss of finer
grains of soil, short timber sheet piles were driven later along the end wall.
This was not satisfactory; indeed the smooth surface of the timber piles offered
still less resistance to seepage and increased the loss of finer particles and in-
creased scouring through the open joints between the piles below the foundation
raft.

In the years 1925 to 1927 the drainage works were extended and it was
necessary to increase the capacity of the pumping-shaft, to lower its top water-
level by 1 ft. 8 in., and to lower the bottom of the shaft by nearly 3 ft. As a
result the bottom of the shaft was close to the layer of waterlogged sand sub-
jected to artesian pressure. When the bottom of the shaft was deepened the thin
impermeable layer of soil became waterlogged and there was considerable up-
surge along the line of posts driven into it to support the shuttering. Before
the work could be completed the spring thaw set in and, in order to prevent the
land being inundated, the water from the drainage channels was collected and
pumped out of the unlined pit, the sides of which soon collapsed. The sliding
of the sides of the pit was stopped by driving timber sheet piling around it, but
the bottom remained buoyant and upsurge of the silt due to the head of water
could not be stopped. In spite of this state of affairs the new reinforced concrete
bottom was placed in a soft and waterlogged soil with which the concrete readily mixed as it was placed.

As was to be expected from such hasty measures, an examination some three years later showed that about two-thirds of the bottom was a mixture of nothing but loose silt and aggregate with no strength or impermeability. The cement had been entirely washed away, and the coarse aggregate had settled under the fine aggregate and silt. Large quantities of quicksand had surged up through this mixture adjacent to the stakes that had been driven to fix the shuttering for the new bottom. There were also serious cracks in the wall adjoining the engine-house.

Measures were then taken to remedy the defects of the work. A deeper surface-drainage system was installed to collect all the upsurging water and to lead it to vertical tubes from which it could be removed without any disturbance of the subsoil. A new reinforced concrete bottom 6 in. thick was cast monolithically with the side walls. When the bottom had hardened, the steel tubes were filled with grout and later cut off at surface level and fitted with caps. The side walls were strengthened with reinforced concrete buttresses, and the surfaces of the bottom and the walls were gunited. At the same time the pumping shaft was separated from the foundation of the engine-house, which was repaired by filling the cracks in the walls and by grouting. The outlet channel was replaced by a steel tube which discharged into the river about 200 ft. away from the engine-house. The pipe is laid on a bed of concrete 6 ft. wide, and an outlet weir 27 ft. long has been provided. Some thirty years later the structures were in excellent condition, in spite of the exceptional stresses to which they were subjected in the severe floods which occurred during the years 1955 and 1956.

(31.2.) BRIDGES.—The geological formation of the great Hungarian plain, and of some other plains in Europe, is such that thick layers of waterlogged sand occur near the surface. In such areas much trouble has been experienced due to the faulty design and construction of the foundations of small bridges. This has been accentuated by the remoteness of many of these structures, with consequent lack of competent control, and by the smallness of the works resulting in the idea that they do not warrant much thought or expense. With the object of reducing costs it is common to pump the ground-water directly from sumps in order to avoid the cost of lowering the level of the ground-water or of placing concrete under water.

An example of this type of work is a reinforced concrete bridge of 90 ft. span built at Kurd, Hungary, in 1947. The piers were built upon concrete raft foundations 10 ft. by 23 ft. in plan laid on a thick layer of quicksand. Excavation was done in a cofferdam formed of timber sheet piles which extended below the bottom of the foundation in order to prevent the lateral displacement of the quicksand. Horizontal braces were provided near the top of the piles and at a depth of about 10 ft. below the level of the ground-water, which was pumped out in stages so that the timbers could always be fixed in the dry. In order to avoid the danger that would arise below this level due to the increased lateral pressures on the cofferdam and at the bottom of the excavation it was specified that below this level pumping must cease, and that the excavation must continue and the concrete be placed under water. However, the equipment was not
available when it was required, and in order to avoid delay the men on the site decided to continue pumping. It was also thought that if the work were done quickly the dangers anticipated by the engineer would not arise! They were soon proved to be wrong, for when the water within the cofferdam had been lowered a further 3 ft. the lateral pressure on the outside pushed the bottom of the piles inwardly (Fig. 52). This movement caused the lower braces to act as a fulcrum and forced the top of the piles in an outward direction. The top braces were loosened and fell into the pit. The joints between the piles opened, and the waterlogged soil flowed into the excavation at an increasing rate. The increases of pressure that brought about these troubles are shown in Fig. 52. It is here seen that the earth pressures are only a fraction of the pressures of the waterlogged soil, which behaves like a dense fluid with a density of about 120 lb. to 130 lb. per cubic foot and has no cohesion or internal friction. The coefficient of horizontal pressure, as determined by Rankine's formula, is therefore increased to unity—at a depth of 10 ft. The pressure is 10 × 125 (average) × 1 = 1250 lb. per square foot. The pressure of waterlogged, but not liquefied, earth would be about

\[ 10 \text{ ft.} \times (110 - 62.4) \tan^2 \left\{ 45^\circ - \frac{1}{2}(30 \text{ deg.}) \right\} + (10 \text{ ft.} \times 62.4 \times 1) = 783 \text{ lb. per square foot.} \]

The pressure thus increases by about 37 per cent.

Pumping was stopped when its effects were seen, the ground-water was allowed to rise to its original level, and the collapse of the cofferdam was thereby prevented. The material was excavated from under the water, and concrete was placed under water. Pumping was resumed when the concrete had hardened. The remainder of the concrete was placed in the dry. In other works remedial
measures were not taken so quickly, with the result that the cofferdams completely collapsed.

(31.3). SINKING A CAISSON.—A site at Békeöcsaba, Hungary, had been a marsh for about a hundred years, and more recently had been covered with spoil from works in connection with the regulation of the river Körös. The upper layers therefore contained material with a high proportion of organic matter, whose decay generated gases. In the year 1955 a reinforced concrete caisson, 36 ft. by 19 ft. in plan, was to be sunk to a depth of 33 ft. below ground level (and 26 ft. below ground-water level) in connection with a sewage treatment works. The sinking was started by loosening and liquefying the soil by means of water-jets and pumping away the slurry so produced. Care was taken to keep the water within the caisson always at a higher level than the ground-water outside, but when the cutting-edges reached a layer of greenish clay from 6 ft. to 8 ft. thick (Fig. 53) the water-jets had to be disused and the sinking was
DEFECTS AND FAILURES DUE TO DEFECTIVE EXECUTION

continued by removing the water by direct pumping from sumps. This, however, resulted in the rupture of the impervious layer, which had been gradually weakened, with the result that the underlying quicksand surged upwards into the caisson due to the pressure of the prevailing water table. The continuous removal of quicksand and water from the neighbouring land also endangered some houses 67 ft. away. An attempt was then made to use a two-stage vacuum well-point system, but this also failed because the bubbles of gas rising from the decomposing organic matter prevented the maintenance of a vacuum. (The composition of the gas was 60 to 70 per cent. CH₄, 3 to 5 per cent. CO₂, 2.5 to 3.5 per cent. CO, and 21.5 to 29.5 other gases.)

By loading of the caisson and by tedious underwater excavation the cutting-edges were lowered to within 10 in. of the required depth. With the means available it could not be sunk any deeper, and at this level the bottom was concreted. However, due to the reduced depth of the cutting-edges, the concrete placed under water in the bottom seal was only about 2 ft. 4 in. thick whereas the head of water on the underside was 25 ft. Also this concrete was porous due to the bubbles of gas ascending through it before it had hardened. For this reason it was not possible to remove the water from the caisson by pumping as the water ascended through the concrete and it was still not possible to obtain an impervious bottom. Finally a prefabricated welded-steel tank was placed within the caisson and fixed to it and the space between this and the walls of the caisson was filled with cement-grout. Not only was the cost of the caisson greatly increased, but its capacity was reduced due to its decreased depth and width. Most of the troubles experienced with this work would have been avoided if the properties of the soil had been known before the work was started so that a proper procedure of sinking and excavation could have been used.

(31.4). DIFFERENTIAL SETTLEMENT.—The bridge at Tiszapolgár, Hungary, built in the years 1939 to 1941, is a remarkable case of a failure arising in spite of exhaustive examination of the site, the accurate interpretation of the results, and the production of a design that would ensure the stability of the structure, or at any rate uniform settlement. The bridge is of three spans; its continuity and the considerable depth (9 ft.) of the main beams rendered it particularly liable to damage if unequal settlement were to occur. Fig. 54 shows the result of the preliminary examination of the soil and the position of the piers. Below the top soil is a layer of bluish-grey silt with a high water content (which at some places was above the liquid limit) and a high porosity (a void content of 54.5 per cent.), and which had no load-bearing capacity. Below the silt was a layer of fine bluish-grey sand with a void content of 38 per cent. to 47 per cent. and containing from 6 per cent. to 15 per cent. of silt with a grain-diameter of about 0.04 in. which should be regarded as a Mø soil. This waterlogged sand was liable to lateral movement and to upsurge when it was subjected to vertical loads. Unfortunately the strata immediately below were not much better, and hard clay could have been reached only by pneumatic caissons. Because of the lack of suitable equipment for constructing such deeper foundations, shallower foundations were designed to be placed upon this weak layer but to be constructed within timber sheet-pile cofferdams by means of underwater excavation and the placing of concrete under water. The sheet piling was to extend 3 ft. below the bottom of the foundations in order to prevent lateral displacement of the soil beneath
the foundations, the depths of which were arranged so that there would be uniform settlements of 2 in. under all the piers. Means of adjusting any unequal settlements were provided in the form of hydraulic jacks which would be inserted between the bed-stones of the piers and the bearing-plates of the reinforced concrete superstructure, each lift being supported by wedges and additional bearing-plates. All these measures were approved by the authorities responsible for the construction, by the men who examined the site, and by the designers.

However, the specification was not followed when the foundations were built. First, because of the difficulty of obtaining timber for the sheet piles,
the use of reinforced concrete caissons was approved at the request of the contractor. It was stipulated that the caissons be sunk by underwater dredging, and that no pumping should take place until the concrete seal had been completed at foundation level. These precautions were, however, abandoned when it was found that the rate of sinking the caissons was very slow as they passed through the upper cohesive layers, and that the grabs of the excavator could not be used
satisfactorily in the sticky mud. The method of dewatering was gradually changed to pumping directly from sumps. The result was inevitable. The soil became waterlogged, the deeper-lying quicksand surged upwards, and the result was increased settlements of the piers. In Fig. 55 are given the records of the settlements of the four piers during a period of three years. The amounts of settlement that were estimated to occur are also given. Several times during the three years the bearings were raised by the jacks and the insertion of steel plates, with the result that the large differential settlements of the piers have not damaged the superstructure.

The curves in Fig. 55 are typical examples of the rate of settlement of foundations in fine-grained non-cohesive soils in that 80 per cent. of the settlement in three years took place within the first four months of completion. It will be seen that differences between the estimated settlements and the actual settlements were 4 per cent. at pier No. I, 150 per cent. at pier No. II, 200 per cent. at pier No. III, and 50 per cent. at pier No. IV. If these figures are considered in relation to the soil-profiles in Fig. 54, it is seen that the settlements are proportional to the differences in the thickness of the underlying layer of quicksand. For example, the layer of quicksand is thinnest under piers Nos. I and IV, where the settlements are least, and thicker under piers Nos. II and III, where the settlements are greater. The smaller amount of settlement to be expected where the layer of quicksand was thinnest (under pier No. I) was mentioned in the preliminary report on the properties of the soil. It was also expected that the layer of compressible clay under pier No. I would increase settlement, but this effect may have been over-estimated because the load-distributing and stress-relieving effect of the intermediate layer of dense gravel had been overlooked. The smaller settlement of pier No. IV may be due to the fact that the depth of the foundation was increased by 6 ft. during construction, so that the depth of compressible soil under it was reduced by this depth.

Generally the differential settlements that occurred were due to the loosening and deterioration of the quicksand as a result of the use of open caissons and pumping instead of, as was originally intended, excavation and placing of concrete under water within sheet-piled cofferdams.

Failures of Cofferdams.

Common causes of failures of the supports of soil around an excavation are their lack of sufficient strength, the leaking of water through the piling or timbering forming a cofferdam, the upsurge of water and silt at the bottom, and disregard of the possibility of changes of the lateral pressure of the soil. (32.11) Unsuitable Bracing of a Cofferdam.—Fig. 56 is a cross section of a cofferdam provided for the construction of a quay wall for a coal wharf beside the river Danube. The wall was to be 43 ft. deep and 150 ft. long and was to be constructed successively in three parts. The cofferdam was formed with steel sheet piles 37 ft. long, and the water was removed by pumping from sumps. It was desired to have three equally-spaced sets of walings and struts, and because of the difference in the lateral pressure at different depths the top struts were of timber and designed to resist a pressure of 3 tons per linear foot and the middle struts were of steel and designed to resist a pressure of 9 tons per linear foot; the bottom struts and walings were also designed to resist a
pressure of 9 tons per linear foot, but these were of reinforced concrete, which would form part of the quay wall, in order to save the time that would be necessary to construct temporary struts in place of the bottom braces when the masonry was erected inside the cofferdam. The spacing of the struts was arranged to permit easy access to the soil by means of grabs.

The first part of the wall was successfully constructed in this way. However, when the excavation of the second part had been completed, and the concrete for the walings and struts at the bottom of the pit had been placed but had not hardened, a flood occurred in the river. The increased flow of the water, the velocity of which was accelerated by the smooth outer surface of the steel piles, resulted in serious scouring of the adjoining bed of the river to a depth of about 10 ft. The removal of the part of the river bed which was helping to support the lower part of the piles, combined with the fact that the unhardened concrete walings and struts at the bottom were useless, meant that the bottom of the piles was supported only by the underlying marl into which they penetrated a short distance, and the surface of which was fissured. The result was that the pressure on the top timber struts was increased due to the higher level of the river and the wet struts were compressed and considerably shortened. In turn the pressure on the steel struts at mid-depth was increased, but the steel resisted the extra pressure without serious deformation. The deformation of the piles due to the yielding of the topmost struts permitted water to enter through the interlocks and saturate the marl at the bottom and reduce its resistance to shearing and its capacity to secure the ends of the piles. As a consequence the bottom of the piling was pushed inwards and collapsed for a length of 33 ft. (Plate XII, facing page 59). The rear wall of sheet piling was not damaged, because the flooding of the cofferdam equalised the pressure on both sides. It is possible that the collapse could have been prevented if the cofferdam had been filled with water to counteract the additional pressure of the flood water. This was a very unfortunate case, in that the unexpected flood occurred at the time when the concrete in the bottom struts had not hardened and therefore took no part in resisting the pressure of water.

(32.12). COFFERDAMS IN RUNNING WATER.—When sheet-pile cofferdams are built in running water they reduce the waterway and may consequently increase the speed of the current. When the bed of the river is of a silty or sandy nature this may cause serious erosion and result in the undermining and collapse of the structure. The following example and examples (32.13) and (32.14) are cases of failures due to this cause.

A weir across the river Rába in Hungary has a length of 330 ft., and was built in three parts as shown in Fig. 57. The first part to be built extended across about 45 per cent. of the width of the river, part II extended across about 25 per cent., and part III extended across about 30 per cent. The bed of the river comprised a layer of sandy gravel from 7 ft. to 10 ft. in depth, under which impervious clay extended to a great depth. The sheet piles were driven well into the clay, so that the water could be removed from sumps without risk of water surging upwards into the cofferdams, and any leaking joints could be sealed with clay in order to make the structures completely watertight. The piles on the downstream face were supported by two rows of raking struts bearing on walings at different levels. On the upstream side the piles were supported by a
timber staging built within the cofferdam and which also served to transport materials to and from the cofferdams.

Parts I and II of the weir were successfully completed. When part III came to be built the whole of the flow of the river had to pass over the completed parts of the weir, which were 5 ft. higher than the normal level of the river. This was taken into account when building the third cofferdam. It had, however, been overlooked that the reduction in the width of the river consequent upon the construction of the parts I and II and the raising of the level of the river upstream of the weir would increase not only the level but also the velocity of the current with consequent scouring of the underlying stiff clay (Fig. 59).* The raising of the water level, and the reduced penetration of the sheet piles and of the piles under the trestle which supported the upstream sheet-pile wall, resulted in the collapse of the cofferdam for a length of 13 ft. Fortunately at the time of the collapse the water had not been removed from the cofferdam, so that there were no casualties and the equipment was not damaged. The work was completed after a second wall of sheet piles had been driven to a greater depth and tied to anchors constructed upstream.

(32.13). A cofferdam at the Nag-Hammadi dam in Egypt failed from the same cause in the early 1930's. This dam across the river Nile is 3000 ft. in length, and has one hundred sluice gates 20 ft. in width. The cofferdams were formed with a single row of sheet piles, which had to remain in use for more than a year and were consequently exposed to the seasonal flood of the river. The work was done in three parts. The first part was successfully completed, and the cofferdam was formed for the construction of the second part. At this stage the river was restricted to one-third of its original width, resulting in an increase of its speed of flow and consequent erosion of the silty bed of the river, especially at the upstream corner of the cofferdam on the river side. This erosion was noticed and its progress was measured, and in order to maintain the depth of penetration of the piles, and to protect the bed of the river against

* See page 84.
excessive erosion, broken stone was tipped against the piles at this corner. With
the arrival of the flood, the level of the river, and consequently the hydrostatic
head, were greater outside the cofferdam than inside it, with the result that
water surged upward through the fine-grained non-cohesive soil at the bottom.
As a precautionary measure broken stone was tipped against the inner face of
the piling where the bottom was surging upwards, and more struts were inserted.
These precautions were, however, useless, because the loose stones did not
prevent the upward seepage due to the hydrostatic head. The hydrostatic
head may be expressed by Darcy's equation \( v = k f \) in this double-layer system,
as shown in Fig. 6e, from which a uniform critical velocity \( v \) may be determined.
If the value of the coefficient of permeability of the outer layer \( k_1 \) is much
greater than the coefficient of permeability \( k_2 \) of the subsequent layers along
the line of seepage, the total loss of hydrostatic head in this permeable layer
\( k_1 = v \left( \frac{L}{k_1} \right) \) will then be negligible and the total hydrostatic head \( H \) will act on
the subsequent layers of lower permeability. Thus \( h_2 = \frac{v}{k_2} \approx H \). If, therefore,
the value of the hydrostatic pressure of the seeping water is \( J = \frac{H}{L} \),
approaches unity, the upward pressure of the seeping water approaches the value of
submerged density, and failure of stability is likely to be demonstrated by silty water
rising through the bottom of the cofferdam. This is what happened at the Nag-
Hammadi dam. With the increasing upsurge of the silt at the bottom of the
cofferdam the silty water found its way under the sheet piles, traversed the eroded
and enlarged seepage channels, entered the cofferdam at the bottom, and com-
pletely filled it. The result was that the river overflowed the cofferdam for a
period of four months, and when it subsided it left a layer of silt to a depth of
up to 4 ft. The cofferdam was reconstructed with three rows of sheet piles,
and the work was successfully completed. This accident could have been pre-
vented if the cofferdam had been flooded immediately the water and silt were
seen to surge upwards at the bottom.

\[ 32.14] \] Part of the underground railway at Stockholm\(^{(12)}\)\(^{(13)}\) was built in the
open water of the Norrstrom Channel, where a cofferdam 467 ft. long, 83 ft. wide,
and 40 ft. deep was constructed (Fig. 6f). Most of the piles were driven to granite of a somewhat weathered nature, and others into clay overlying the rock.
The excavation extended well below the level of the bottom of the piles, which
were supported at the top by a trussed beam and at the bottom by a concrete
waling. The rock was excavated by blasting. On a Saturday afternoon, when
the men had left the site, a slight infiltration of water was seen on the surface
of the freshly blasted rock. This was quickly followed by an intrush of muddy
water and the collapse of the sheet piles.\(^{(14)}\)

The subsequent investigation showed that the rock was composed of granite and
gneiss, together with badly fissured pegmatite interspersed with very thin
layers of clay. It was thought that a layer of fine sandy clay occurred in a fissure
in the rock that had been widened by the blasting, and that when this clay had
been wetted by the infiltrating water the wedge of rock marked \( C \) in Fig. 6f slid
into the pit, taking the sheet piling with it. This fault in the rock was not re-
vealed by preliminary examination of the site, for borings at the site of the failure
indicated solid rock only and gave no intimation of the presence of fissures or
of the thin layers of clay that had been deposited in them.

As shown to the left of Fig. 6f, where similar conditions occurred in the re-
mainder of the work the concrete waling at the bottom of the sheet piles was
anchored by means of prestressed cables or bolts into the sound rock below.

\* See page 84.
Different methods of anchorage were used according to the properties of the rock. Where there were no weaknesses in the bedrock, bolts of \( r_{1/2} \) in. diameter and up to 40 ft. in length were inserted in drilled holes which were filled with cement grout before the bolts were driven into them. Where the rock was weathered hollow bolts of \( r_{1/4} \) in. diameter were hammered into it and then grouted in groups. Where the rock contained layers of sand and clay, bolts of high-tensile steel with a breaking strength of 112,000 lb. per square inch were used and post-tensioned. Wedge-shaped projections were welded to the bolts, and the lowest 8 ft. were embedded in cement grout. A pull of 25 tons was then applied to the top of the bolt and maintained for seven days. The tensioning force was then removed and the hole was filled with grout throughout its entire length. When the grout had hardened a pull of 20 tons was applied to the top of the bolt, and this was
maintained by means of a nut throughout the work. By this means the work was completed without further difficulty.

** Failures of Earth Banks. **

The following example is a case of the collapse of earth banks at the sides of excavations.

(322). ** Delay in Completing Foundations.**—It was proposed to build a large industrial works on both sides of a valley in Hungary, the slopes of which were originally covered with forests. A study of the vegetation, and the fact that the upper layers of the soil contained from 8 per cent. to 10 per cent. of montmorillonite, indicated that these layers had very expansive properties, and that there would be a risk of landslides if they became saturated and their small degree of stability were disturbed by applied loads. Traces of former landslides and displacements could be seen, but at the time it seemed that the soil had come to rest, largely as a result of the protecting and stabilising effect of the vegetation.

The buildings were to be on three terraces one above the other (Fig. 58 on pages 82 and 83), which would result in the slopes between the terraces being steeper than the natural slopes. The experts who examined the site recommended that it was unsuitable for the purpose, and that it would be better to find another site. They particularly mentioned that it would be absolutely necessary to prevent the upper layers of fine, colloidal, and expansive bentonitic clay from becoming saturated, and they emphasised the great importance of proper surface drainage, the prevention of water entering the excavations for the foundations, the protection and revetment of open channels for collecting surface water, and the establishment of grass on the slopes of the excavations. It was, however, decided that the urgency of the work was such that there was not time to find another site, and the work proceeded.

With a view to getting the work done as quickly as possible, no exact calculations were made of the stability of the slopes of the excavations, but the general
principles of relevant codes of practice were applied. This was even more
dangerous because earthen embankments from 10 ft. to 20 ft. high were to be
formed between the terraces and would impose extra load on the slopes above
each terrace. The trees and vegetation were immediately removed from the
sides of the valley, and excavation for foundations was started simultaneously
on all the three terraces. When the excavations were partly completed on one
side of the valley the work was stopped and the machinery taken for use on another
site, with the result that the partly-excavated pits were filled with rainwater
during the autumn and winter; no effort had been made to protect the pits by
collecting the precipitation or diverting the water running down the side of the
valley. In the following year the foundations were completed and the erection
of the buildings was started but not completed, and again no effort was made to
protect the site from rainwater, although it had been seen that in places the surface
was scaling away from the slopes of the excavations. The construction was com-
pleted in the following year. By this time the soil was in places sliding down
the sides of the excavations. Drainage trenches were then constructed and the
slopes were strengthened by stone revetments and drainage ribs; this work was,
however, not done properly, because of the permeable nature of the revetment
and the porous nature of the stone. The weight of the earthen embankments
formed above the terraces, together with the saturation of the soil following an
exceptionally wet autumn and winter, caused the soil to slide to such an extent
as to endanger the buildings on the terraces, and the slopes had to be supported
eventually by reinforced concrete retaining walls.

On the other side of the valley, where the work was continued without
interruption and the precipitation was carefully collected and diverted, there
was no slipping of the soil. The soil was similar to that on the other side of the
valley except that its condition was somewhat better in that the void ratio of
the bentonitic clay was 0.6 to 0.7, the CaCO₃ content 8 per cent., and the FeO
content 20 per cent. On the side of the valley where sliding occurred the void
ratio was from 0.7 to 0.8, the CaCO₃ content was about 4.5 per cent., and the FeO
content about 8 per cent. The properties of the soil on the side of the valley
where sliding took place was therefore less stable than that on the other side where
no difficulty was experienced.

This example shows the importance of protecting the slopes of pits and founda-
tion works generally against precipitation and the risks of not proceeding con-
tinuously with foundation works in open excavations. The influence of the
mineral constituents of soil should also be noted. In the case of large excavation
pits in sensitive clays the choice of a safe slope must not be based upon
empirical codes and regulations but on local experience and careful calculations for
a particular site.

Ineffective Bracing of Lining of Excavation.

The case which follows is an example of the collapse of the lining of an
excavation due to ineffective bracing.

(32.3). DEFECTIVE SUPPORT OF EXCAVATION.—The following account of
the failure of the support of the side of an excavation for part of the underground
railway in Berlin is abstracted from a report prepared by Professor Franz
Dischinguer.(15)
The tunnels were constructed by a cut-and-cover method which had been standardised after many years of experience. The excavation was generally 33 ft. deep and about 60 ft. wide, and the sides were supported by substantial steel joists driven to a depth of 5 ft. below the bottom of the excavation and supporting horizontal timber planks. The joists were at 6 ft. 8 in. centres, and the ends of the planks were wedged against the flanges. Across the width of the excavation opposing joists were supported by timber struts (which were interrupted at their mid-length in order to reduce buckling) and by horizontal steel channels fixed to vertical steel posts spaced at 20 ft. centres and driven 10 ft. into the bottom of the excavation (Fig. 62). Between every fourth or fifth pair of these intermediate steel posts a diagonal transverse bracing was inserted in a
vertical plane between the struts at various levels (see lower part of Fig. 62). The lowest set of this bracing had to be removed before the concrete forming the bottom of the tunnel was placed, but as the work progressed this concrete was placed so quickly after the diagonal struts were fixed that the bottom struts were omitted. As the enclosure was not watertight, and the excavation was to extend well below the level of the ground-water, the soil was dewatered by the well-point method.

This method had been used successfully in the construction of many miles of this subway. At one part of the work, however, some changes were made. The depth of the excavation was increased by 3 ft. 3 in., with the result that the penetration of the steel joists at the sides was reduced from 5 ft. to 1 ft. 9 in. and the intermediate joists penetrated 6 ft. 9 in. instead of 10 ft. Also the width of the excavation was increased by a third bay about 20 ft. wide on the eastern side (Fig. 63). A wall of similar construction was built to support the soil on the eastern side, and the steel joists were driven to a depth to allow for the deeper excavation, but, as is seen in Fig. 64, these joists were not in line with the joists in the other part of the excavation (see joists Nos. 76, 77, and 79 in Fig. 64). This non-alignment was made more serious because, in order to use short pieces of timber which were on the site, it was necessary to use timber packing to join the transverse timbers to the horizontal channels fixed to the steel posts, with the result that the possibility of buckling was increased. The change of method was due to a desire to avoid the use of new bracing, which would have been necessary if the joists had been driven to the former depth, and also to avoid delay in the work.

![Fig. 63.](image-url)
Plate XIX (see page 128)

Plate XX (see page 131)
As a result the joists in the inner wall of the eastern part tilted some 20 ft. at the top for a length of 200 ft. and the wall collapsed. Twenty-nine men were killed and many injured. Professor Dischinger states that the failure was due to four causes, namely (1) insufficient penetration of the inner row of steel joists; (2) absence of three-dimensional bracing between the vertical struts; (3) the absence of a connection between the upper struts and bracing with the bottom of the excavation; and (4) the presence of a lift shaft at one end of the central and eastern parts of the excavation, where the struts were also not sufficiently stiff in the longitudinal direction. This accident emphasises the need for horizontal stiffening of bracings and struts across large distances. Such structures are unstable because of the hinge-like joints between the posts and struts. It is not sufficient to allow only for the lateral pressure of the retained earth, but the possible non-uniform distribution of pressure of different layers of soil, vibration caused by vehicles and machines, and other local variations of stress, which cause oblique stresses, must be considered. It is better to support the posts by means of anchors, but whatever construction is used it is essential that it be rigid in three dimensions.

Faulty Construction.

An entirely satisfactory design may fail if the construction is not done in a proper manner. Much trouble has been experienced due to the faulty construction of piled foundations, for example if the piles are too long or too short, if the type of pile is unsuitable, or if the method of driving, or of forming piles cast in place, is unsatisfactory.

(33.1). Changing the Load on Tapered Piles.—The two-story building at Budapest of which a cross section is given in Fig. 65 was carried on continuous
footings constructed on piles so that no load would be placed on the compressible peaty soil. A pile with a taper of about 1 in 30, cast in place in a steel tube, was used, and the tube was left in position in order to give to the concrete some protection against aggressive chemicals in the peat. Each spring the building settled slightly due to the seasonal higher level of the ground-water saturating the upper layers of peat and silt and reducing their cohesion and internal friction. The load-bearing capacity of the soil under the points of the piles was also reduced. As a result there was some settlement of the piles but, due to their tapered shape, the settlement produced a lateral pressure on the surrounding soil which quickly halted the settlement.

After the building had been in use for some time an opening for a door was broken into an outer wall immediately over a pile, which was then relieved of load. The neighbouring piles consequently became overloaded and settled, the pile under the door opening acted as a support, and the large negative bending moment produced over this support caused the large crack shown in Plate XIII (facing page 90).

It was then decided to construct a raft foundation in order to ensure uniform settlement. As the building was supported on the piles the soil was removed to a depth of 3 ft. over the whole of the site (Plate XIV, facing page 90). Unfortunately, when the soil had been removed a storm occurred and the excavation was inundated, the upper layers of soil were saturated, and there was considerable differential settlement of the piles, two of which were cracked at the top (Plate XV, facing page 90). The settlement took place within a few hours and serious cracks occurred in the building, which was then underpinned with temporary cribs. The reinforced concrete raft was then constructed and the building was repaired. The result was satisfactory, but expensive. The large area of the raft reduced the compression in the layers of weak top-soil, and, although it produced a downward extension of the zone of compression, the stronger soils below did not suffer any considerable deformation in spite of the increased stress.

(33.2). UNSUITABLE SPACING OF PILES.—In the construction of a large electricity-generating station at Palkonya, Hungary, no failure occurred, but a great deal of money was wasted as a consequence of unsuitable spacing of the foundation piles. The site was carefully examined, and it was decided that the columns of the building should be supported on foundation blocks 34 ft. 4 in. long and 7 ft. 8 in. wide and extending to a depth of 14 ft. below ground level. The contractor was given the option of excavating in cofferdams of steel sheet piling or in pits lined and braced with timber. It was agreed that the water might be removed from the pits by pumping from sumps, because the level of the ground-water would not be more than 4 ft. higher than the level of the bottom of the foundation. The bearing layer would have been coarse yellow sand about 4 ft. 4 in. thick (Fig. 66) overlying dense gravel and sand extending to a considerable depth, the permissible load on which was estimated to be from 2\(\frac{1}{2}\) to 3\(\frac{1}{2}\) tons per square foot. The idea of building the foundations on the higher layer of yellow loam was considered but, although its relative consistency was 1-0, the idea was abandoned because the design of the superstructure was such that it would be seriously damaged by differential settlement.

The work suddenly became very urgent, and had to be started in winter with the consequent risk due to the higher water-level in the following spring,
In view of the high cost of cofferdams or other means of supporting the sides of the excavations, which would be saturated, the possible risks of dewatering and the time required to drive and extract sheet piling, the contractor proposed that piled foundations be used. The piles were to be precast and driven to depths of from 17 ft. to 23 ft., that is from about 5 ft. to 11 ft. below the proposed level of the bottom of the foundation blocks. The cost of piling was much less than the cost of foundation blocks built within sheet-piled cofferdams, and the suggestion was adopted. The spacing of the piles was unusual. They were driven in two rows around the periphery of the foundations, and spaced only from 7 in. to 8 in. apart, the idea being that they would prevent lateral displacement of the earth enclosed by the piles and so reduce the risk of settlement. This, however, was false reasoning because, due to its high compressibility, the enclosed earth could not conform with the settlement of the piles, and even if the surface of the soil initially carried some of the load it could not indefinitely relieve the load on the piles because it would be transmitted to the piles in the form of friction. As a consequence of their close spacing the piles could not be driven to the required depth, and many of them were damaged in the attempt to do so. Around each foundation, which was 34 ft. 4 in. by 7 ft. 8 in. in plan, 77 piles were driven in two rows. Such a large number was quite unnecessary, as was shown by test loadings, which also showed that the bearing capacity of piles tested as groups was much less than that of piles tested singly. The result was that foundations of unnecessarily high bearing capacity were provided at what proved to be a greater cost, whereas had the work been started earlier, and not in the winter, the block foundations could have been constructed with safety and with little dewatering of the excavations.

(33.3). PILES IN WRONG POSITIONS.—Damage occurred to an apartment
DEFECTS AND FAILURES DUE TO DEFECTIVE EXECUTION

house six stories high, built at Budapest, near the river Danube. The site comprised fresh filling to a depth of 17 ft., and tapered piles (as described in example 33.1) were used to support the bottom members of the reinforced concrete framed structure as shown in Fig. 67. The floor beams are supported by the interior wall (K) and the outer walls (Sz), but the spacing of the piles was the same under the non-loadbearing walls as under the loadbearing walls. Because all the walls were supported on equally rigid foundations, it was inevitable that the loadbearing walls settled more than the partition walls. Because of the rigidity of the structure damage was confined to the lower stories, where there was slight bulging and cracking of the non-loadbearing walls and some distortion of the floor beams (Fig. 68). This is a good example of the fact that stronger foundations may do more harm than good if they result in differential settlement due to non-uniform disposal of the loads of the superstructure; in such cases the possibility of future changes in the position and of the magnitude of the live loads must also be considered.

(33.4). Piles Spaced Too Closely.—The settlement of an electricity generating station at Muskegon, Michigan, U.S.A., (16) was due to the same cause as example 33.2, namely the spacing of piles too closely together. The columns of the building each transmitted a load of about 440 tons to six continuous beams supported on piles, and the beams were connected transversely at frequent intervals, the spaces between the connecting beams being left open. The site of the generating station was on the shore of a lake into which sawdust and other refuse from an adjacent logging and lumber works had been dumped to an apparent depth of 18 ft. This material was removed by dredging. The bottom of the lake consisted of fine silt and sand. No borings were made, but test piles were driven and withstood a load of 30 tons. In the design the load to be carried by each pile was restricted to 16 tons, and this required about 2000 piles spaced at centres of 2 ft. to 3 ft. 8 in., according to the different loads of the columns. Timber piles 40 ft. long were used, and driven to a penetration of $\frac{1}{4}$ in. per blow by a hammer weighing 2200 lb. falling 5 ft. Due to the resistance to dynamic loads of some of the soil at lower depths the penetration required was achieved at lesser depths and considerable lengths of the piles had to be cut off.

When the building was completed settlement occurred, but the grid of foundation beams was sufficiently rigid to prevent differential settlement even under the columns carrying the greatest loads. It was, however, decided that the settlement should be stopped, and an investigation of the site was undertaken. Boring showed that in some parts the refuse from the sawmill extended to a depth of 45 ft., below which was a layer of sand and clay to a depth of 75 ft. overlying hard clay. A check of the design showed that the members of the foundation were correctly dimensioned in accordance with the original assumptions. However, whereas the test piles were driven at 8 ft. centres the piles in the work were driven at centres averaging 2 ft. 8 in., resulting in a distribution of stress in the soil which considerably decreased their load-bearing capacity. Also, it is the writer's opinion that the test based on a maximum penetration per blow of the hammer could not give a reliable result because of the excessive fall of the hammer (5 ft.), and the consequent loss of impact-energy; in addition, the driving of the piles would compact the loose material between the piles and increase the friction between the soil and the piles as they were driven so closely together. Further
settlement was successfully prevented by the use of cylinders in the spaces between the beams of the foundation grid. The steel cylinders were 1 ft. 4 in. in diameter, and were driven to the hard clay at a depth of 75 ft. Because of the restricted headroom the cylinders were driven in lengths of 10 ft. connected by internal sleeves. When the cylinders were driven the soil was removed from them by compressed air and they were filled with concrete, and a grid of reinforced concrete beams was constructed on top of them.

(33.5). PILES OF UNSUITABLE LENGTH.—Experience in constructing the foundation of a building eight stories high at Budapest shows that failure can be caused by the use of piles of too great a length. In this case no preliminary examination of the site was made, as experience of other buildings in the neighbourhood indicated that piles driven to a set of 1·2 in. with ten blows of the hammer into the layer of dense sandy gravel at a depth of from 16 ft. to 20 ft. would be satisfactory (see the full line in Fig. 69). The precast concrete piles available were in standard lengths from 17 ft. to 23 ft. When some of the longer piles were being driven it was found that the required set was nearly obtained at depths of less than 20 ft. (dash line in Fig. 69), but that below this level the penetration per blow of the hammer was greater and did not diminish when the piles were driven a further 6 ft.

Borings were made to ascertain the reasons for this variation in penetration. In the layer of loose sand underlying the dense sandy gravel there was a striking uniformity of the size of the grains (U = 2·5). It was desired to ascertain the relative density of this waterlogged non-cohesive soil, but it was not thought possible to obtain an undisturbed sample, and it was possible to do so only by the use of the newly-invented Varga-Makkai system of sampling,* which is based on the freezing effect of evaporated liquid air. From the samples so obtained it was found that at a depth of 20 ft. there occurred a layer of loose quicksand 12 ft. thick, the grains of which were of uniform size and which was readily displaced laterally when it was subjected to a vertical load. This was the reason for the easier penetration of the longer piles when they reached this layer. The points of the shorter (17 ft.) piles were in the layer of dense sandy gravel from 3 ft. to 5 ft. thick which had an angle of internal friction of 35 deg., an index of uniformity of 20 to 35, a pore volume of 38 to 39 per cent., and a relative density of 0·60. The loose quicksand into which the longer piles penetrated had an angle of internal friction of 33 deg., an index of uniformity of 2·5, a pore volume of 35 to 40 per cent., and a relative density of 0·3. Tests showed that whereas the shorter piles had a bearing capacity of 80 tons to 85 tons, the bearing capacity of the longer piles was only from 40 to 60 tons. This was not discovered until all the piles had been driven.

In order to avoid extracting the longer piles, cement grout was pumped into tubes driven around the groups of piles. This increased the density and internal stability of the loose soil and the cohesion of the grains, and so increased the bearing capacity of the piles. This example indicates that the use of longer piles does not necessarily increase their bearing capacity but, perhaps more important, it shows that a thorough examination of the site is indispensable before foundation work is undertaken.

(33.6). SLIDING OF A QUAY WALL.—In the year 1952 a quay wall 1733 ft.

long and 46 ft. high was constructed in the river Danube at Dunapentele. The wall is built of reinforced concrete caissons (Fig. 70) each 40 ft. long and 33 ft. 4 in. wide, which were built to a height of 33 ft. 4 in. at the side of the river from whence they were launched and towed into position. At the site the bed of the river was dredged to a depth of from 3 ft. to 5 ft., and into this excavation the caissons were sunk by admitting water into the inner compartments. Back-filling of sandy gravel was then placed to the level + 319 ft. shown in Fig. 70. The wall was later completed to its full height by the construction of a reinforced concrete wall to elevation + 331 ft. 8 in. together with an upper gangway and stiffening beam at the top. Back-filling was then placed to the top of the wall.

It was known that there had been many cases of such walls sliding forward due to the pressure of the back-filling and, in order to increase the frictional resistance under the caissons, a layer of crushed stone 3 ft. in thickness was placed on the bottom of the trench. The effects of the pressure of the filling and of any moving loads on the quay were to be resisted only by this friction; due to the small depth of the trench and the looseness of the soil on the river side, this soil was not considered to take any part in resisting forward movement of the caissons. When about half of the caissons were in position, part of the wall for a length of 666 ft., where the first stage of back-filling was nearly completed, moved forward distances up to 18 ft. 4 in. (see Fig. 70 and Plate XVI, following page 90). The movement was found to be due to several causes as follows.

(1) Where the movement occurred the soil immediately under the dredged trench was a very plastic and slippery clay which had almost no cohesion or internal friction due to the upward artesian water-pressure from the water-bearing layer below. The frictional resistance of the surface of the clay was therefore much less than that at the top of the stone on which the caissons rested,
DEFECTS AND FAILURES DUE TO DEFECTIVE EXECUTION

in spite of the greater normal pressure of the overlying layers of gravel and sand.

(2) The slope of the natural embankment on which the back-filling was placed was covered with silt from the river, and this presented a slippery surface.

(3) The material for the back-filling was delivered by water and transferred to the bank of the river by floating elevators and stored in heaps from 20 ft. to 26 ft. high, which were so far from the bank of the river that it was thought that they could not cause the earth to slide forward. By this arrangement the back-filling could be placed easily and quickly, irrespective of the level of the river. However, this sudden loading squeezed out the water from the underlying layer of fine silty clay and much reduced its frictional resistance, and the time required for the restoration of internal shearing resistance was not available. As a result the surcharge of the heaps of back-filling material combined with the back-filling already in position behind the caissons produced a pressure considerably greater that the calculated earth pressure, and the shearing resistance along the underlying inclined slippery soil-surface was exceeded. The result was that sliding occurred on the surface of slippery silt which extended to below the bottoms of the caissons, thus forming an uninterrupted sliding surface, and the back-filling, the broken stone under the caissons, and the caissons all moved forward together. The difference between the calculated and the actual volume of soil liable to slide is shown in Fig. 70, where the lines ABC represent the calculated area and lines DEFGA represent the actual area.

(4) At the place where the movement occurred the dredged trench under the caissons was wider than elsewhere and the layer of broken stone was also wider, but this provided no extra frictional resistance. When the caissons were lifted it was found that the broken stone was in the same relative position beneath them, thus proving that the caissons and the stone moved together.

When the work was reconstructed larger stone was placed in the trench, and this was not placed until immediately before the caissons were to be sunk. (During the original construction some months often elapsed between the placing of the broken stone in the trench and the sinking of the caissons, and during these periods the surface of the stone became covered with slippery silt deposited by the sluggish river.) Before the back-filling was placed, stone packing, in the shape of a wedge, was placed against the back wall of the caisson, so reducing the horizontal component of the soil-pressure by 32 per cent. The back-filling of ballast was then placed at a slower rate, and each layer of 3 ft. was inundated for forty-eight hours before the next layer was placed. The stone packing was covered with an inverted granular filter in order to prevent the fine particles of the ballast filling the voids in the stone packing, as this would have reduced its angle of internal friction and consequently its capacity of relieving stress. Later on some smaller forward movements occurred at other parts of the quay, but these were due only to overloading the deck of the quay beyond the load provided for in the design and defective caulking of the joints between the caissons.

(337). U P W A R D P R E S S U R E I N S I L T Y C L A Y.—The collapse of a quay wall at Leningrad was the result of failure to take into account the possibility of an upward pressure arising under the wall. The following notes are abstracted from a report on this accident by Mr. N. M. Gersevanov. The wall (Fig. 71) was of the gravity type, and was built on a silty clay. The resultant of the
load $W$, the horizontal component of the water pressure, and the active earth pressure $E$ was an inclined force $R$ acting eccentrically and tending to overturn the wall. The contact pressure diagram of this overturning force had the shape AEBDC, in which the area BCD represents compression and the area AEB represents tension at the bottom of the wall. In a clay soil compression will be immediately resisted by the incompressible water in the pores, and the hydrostatic head will be immediately changed. If the original height be denoted by $H$, the pressure at A will decrease to $\frac{AE}{\gamma_w}$, the pressure at B will not be altered, and at C it will increase to $H + \frac{CD}{\gamma_w}$. Assuming that $\frac{AE}{\gamma_w}$ is greater than $H$, then at some point between A and B the hydrostatic pressure will be zero. When the wall tilts, seepage of the pore-water will occur, due partly to the difference in pressure and partly to the formation of a gap between the wall and the soil along the line AB due to the lifting of the wall. This results in the loosening of the structure of the soil by the washing out of its finer particles and in establishing a free way and offering a full surface subjected to upward pressure. In cohesive soils the regression of pore-water may take place very slowly; in this case the wall gradually tilted to an angle of 25 deg. and collapsed fifteen years after it was built.

(33.8). Subsidence During Sinking of a Caisson.—The difficulties experienced in the sinking of two caissons on each bank of the river Danube at Káposztásmegyer, Hungary, demonstrate one of the possible causes of failure of deep caissons or wells. In connection with the installation of a tunnel for carrying water under the river, two shafts (Fig. 72) from which to drive the tunnel were required on each bank of the river. The rectangular shafts had inner dimensions of 13 ft. by 29 ft., and they were to extend to a depth of 55 ft. into a layer of impermeable clay. Above the clay the soil consisted of a top layer 30 ft. to 33 ft. thick of waterlogged sand and gravel and layers of clay and silt from 23 ft. to 27 ft. thick. The shafts were to be formed by sinking open caissons
with walls tapering from 4 ft. 4 in. thick at the bottom to 1 ft. 8 in. thick at the top; this taper was necessary to assist the sinking, which was started by dredging. This was successful only to a small depth in the non-cohesive waterlogged gravel and sand. When the cohesive layers were reached direct pumping and bailing by hand were used, partly because underwater dredging failed in this material and partly because the soil had to be removed from under the cutting-edges. As this proceeded the work became more difficult due to the increasing resistance under the cutting-edges, the increased friction on the walls, and the increase in the amount of water to be removed. Further difficulties occurred when the anchorage of the steel cutting-edges was broken from the concrete and twisted as a result of the high stresses present when the cutting-edges entered the harder clay. These cutting-edges were cut away and removed, and the less effective bare reinforced concrete edges had to be used for cutting. The increased rate at which the ground-water infiltrated, and was removed, resulted in more and more gravel, sand, and silt from the upper layers being washed into a sinking-cone which formed around the caisson and causing a subsidence of the surface of about 2 ft. extending to distances of 30 ft. to 50 ft. from the
caisson. One of the caissons could be sunk to the required level, with the aid of top loading; the last 15 ft. of depth of the other caisson on the same side of the river was effected by subsequent underpinning.

The large sinking-cone extended also through the layer of impermeable soil and became filled with sand and gravel which formed a channel for the seepage of water from the top layer. The infiltration of water should have been prevented by grouting, but in this case the provision of sufficient grouting holes was not possible. Therefore after the concrete seal had been placed in the shafts, the tunnel-shield was erected in an airtight bottom chamber because the breaking through of the wall to start horizontal tunnelling with the shield was to be done in compressed air in order to prevent an inrush of water and cohesionless soil which would be drawn down through the cone formed during the sinking of the shaft.

The lesson to be learnt from this example is that circular shafts and caissons are best when they are to be sunk to considerable depths—any rectangular shape is more difficult to sink and produces larger cones of subsidence. It also shows that special care should be taken to ensure that the fixing of the cutting-edge is sufficiently strong, and that the infiltration of water from upper layers of water-bearing soil must be prevented by grouting, at any rate if the bottom of the shaft is to be broken through for the construction of a tunnel or other purpose. As a result of the difficulties described the shafts on the other bank were of circular cross section, the wall had less taper, and the steel cutting-edges were more firmly fixed to the reinforced concrete; these shafts were sunk, and the hole for the shield was cut out, without any unusual difficulty.

(33.9). Non-uniform Compaction of Loose Soil.—The favourable effect of well-compacted soil on the distribution of stress is well known, but it has happened that insufficient consideration has been given to changes of the degree of compaction, as is shown in the following example.

The design of a steel rolling-mill at Díosgyőr, Hungary, included reinforced concrete bowstring girders 50 ft. 8 in. long, spaced 30 ft. apart, and the foundations were to be on a layer of somewhat plastic clay at depths of 14 ft. to 27 ft. under a filling of blast furnace slag. The intention was to remove the slag and form braced and strutted pits over the clay and to construct reinforced concrete cellular block foundations in these pits (see left of Fig. 73). It was found, however, that this would produce a load on the clay of 34 tons per square foot, which would result in settlement and a long period of consolidation. After the pits were excavated it was decided to fill them partly with well-compacted and graded granular soil on which the foundations of the columns would be constructed (see right of Fig. 73). It was thought that the compacted soil would be able to carry the load with only a small settlement, and also that it would be beneficial in transmitting considerable pressure to the adjacent loose slag. This method would also reduce the load on the underlying clay, and would also be cheaper than the original design.

However, the soil used was poorly graded and was not thoroughly compacted. In addition, a layer of concrete 6 ft. thick was first deposited on the clay at the bottom of the pit, with the result that the load on the clay was increased and the depth available for new soil to distribute stresses to the slag was reduced. When it was noticed that the soil was being dumped into the pits without any attempt
to compact it, some loading tests were made and it was found that considerable settlement of some of the foundation blocks had occurred. This indicated that not only was the filled soil compressed vertically but that it had also expanded laterally and penetrated into the loose slag at the sides of the pit, which in turn resulted in more settlement. This lateral displacement of the soil below the foundation of the column produced horizontal stresses equal to 0.2 of the normal pressure, which represented a considerable thrust against the loose slag.

An attempt was made to increase the resistance of the slag by the use of cement grout to fill the voids and bind the particles together but, chiefly due to the non-uniformity of voids in the material, this was not successful, and the original design using cellular foundations to the full depth of the excavation was resorted to wherever the depth of the filling exceeded 10 ft. When the work was completed the settlements were recorded, and the results are shown in Fig. 74. It is seen that the foundations built upon the filling settled at a slower rate than the cellular structures founded on the clay, and that the total settlement after four years was less; this may have been because the plastic clay was continuously in a state of consolidation. During the first year or so, the cellular foundations settled
Fig. 75.
very quickly, after which they settled at a slower rate but at about twice the rate of the foundations on the filling. At the end of three years, however, the difference in the total settlements was not unduly great. Fig. 75 (see page 104) is a record of the worst settlements of the foundations on filling in the two rows of columns, and Fig. 76 (see page 105) gives similar information relating to the columns on cellular foundations. As was to be expected, in view of the fact that the girders span 31 ft. and are less than 30 ft. apart, the distortion of the superstructure was somewhat greater in the longitudinal than in the transverse direction. The settlement under the crane beam was also somewhat greater than the settlement under the columns supporting the girders, but no damage has occurred, due to the flexibility of the girders and the adjustability of their supports.

This example shows that in such cases the changing of the soil is not satisfactory for supporting loads directly, but only as a means of distributing loads. Also, in order to distribute the load effectively, the sides of the excavation should be inclined so that at the top the sides extend far beyond the area of the bottom in order to ensure good compaction of the surrounding loose or soft soil—vertical sides must be avoided.

Defective Workmanship and Materials.

The trend in design is to reduce cost by refinements in calculations that lead to higher stresses in materials and by more economical arrangement of structural members. Some principles relating to the actual construction have also been the subject of codes and regulations. The work in the design office can be easily controlled, but it is generally far less easy to control the work on the site and to ensure that it is strictly in accordance with the specification. The result is that the margin of safety may be less, due to the use of higher stresses, because the quality of the concrete used in foundations, of the construction of cofferdams, and of methods of dewatering have not always kept pace with the improvements in design. In the design of foundations a thorough knowledge of the properties and the structural behaviour of the soil is indispensable, but this is of no avail if the work is faulty due to a striving for economy and speedier construction. The strength of soil, and of materials used in foundations, depends more than most other structural materials on the quality of workmanship. There is often a tendency for men on the site to attach little importance to some of the requirements of the engineer; they must be strictly controlled by the engineer or his representative, for it has been seen in some of the examples already given that foremen and others can depart from the engineer’s instructions in ways that can be described as irresponsible. Some of the commonest defects in workmanship and materials are described in the following examples.

Defective Concrete.

In the case of concrete deposited in layers under water it is particularly important that it be of sufficient strength and free from voids.

When water is pumped from sumps at the bottom of a pit there is a great risk that the cement will be washed out of concrete placed on the bottom of the pit, and that the subsoil may surge upwards through the freshly placed concrete. If this should occur, not only will the concrete have little strength and imper-
meability, but the completed work will be liable to continuous scouring and undermining.

When concrete is placed under water by means of a tremie it is essential that the work be continuous, for any interruption in the process will result in weak patches. This is particularly important in the construction of bored piles, in which case any interruption in the flow of concrete, or the raising of the casing too soon, may allow the pile to be damaged by the inflow of soil as well as water.

(34.11). **Defective Compaction of Concrete in Piles.**—Mr. R. D. Chellis(48) has described the defects found in 188 cased concrete piles forming the foundation of a gas holder tank. The piles were 25 ft. long and were cast in corrugated steel shells of \(14\frac{1}{2}\) in. outside diameter. The upper 15 ft. of the piles were reinforced with four \(\frac{5}{8}\)-in. bars with a helical binding of 10 in. diameter at 2 in. pitch. The shells were driven 17 ft. into the ground. When some of the steel shells were removed it was seen that some parts had not been filled with concrete. This was attributed to the use of poorly proportioned concrete of such a stiffness that, with the means of compaction used, could not fill all the space between the helical reinforcement and the casing; also, there were voids within the helical reinforcement due to the arching action of the stiff concrete. As a remedial measure the casings were removed to below ground level, the voids that could be seen were filled with gunite, and the upper parts of the piles were encased with gunite 2 in. thick on a steel fabric with 3-in. meshes.

(34.12). **Defective Bored Piles.**—In the early 1890's a residential building five stories high was built on a raft foundation adjacent to the embankment on the river Danube at Budapest. The soil under the raft comprised alternating layers of peaty and organic silt under a layer of fresh filling. These soils were subjected to settlements due to compaction, and also to the collapse of the grain structure as a result of the washing out of fine particles due to the changes of the level of the river. The building had been continuously subjected to settlement, but in the 1920's a differential settlement of such magnitude occurred that the southern wing of the building had to be demolished.

Later on, after an examination of the site, it was proposed that the remaining parts of the building be underpinned with bored piles, in which the concrete was placed by means of a tremie extending to a layer of stiff clay at a greater depth. The bearing capacity of the piles was ascertained by test-loading, and whereas most of the piles carried 40 tons one of them would not carry 20 tons. When the weak pile was extracted it was seen that the concrete was very honeycombed, and was therefore not suitable for placing under water. The point of the pile contained very little cement, which indicated that the tremie did not reach the point of the pile and that the concrete was allowed to fall through water. At several places the steel reinforcement had no cover of concrete; this indicated that the flow of concrete was not always continuous as a result of the stiff concrete clogging the tremie. At other parts of the pile the cement had been washed out and the concrete was so weak that it could be crumbled in the hand. As a result the piles were all replaced with longer ones of better quality.

In an attempt to avoid some of the defects found in the piles described in the preceding paragraph it was decided to use grouted piles under a residential building erected in Budapest in the year 1958. As is shown in Fig. 77, the soil comprised a top layer of filling about 6 ft. thick under which was a layer of
quicksand about 11 ft. thick, a layer of peat and organic silt about 6 ft. thick, and below that a layer of dense sandy gravel about 10 ft. thick which should have formed a reliable foundation material. It was therefore decided to found the building on this layer, as it was important to avoid differential settlement under the building which was designed as a rigid frame. The upper surface of the layer of sandy gravel was 22 ft. 4 in. below ground level, and an attempt was first made to sink wells to a depth of 27 ft. It was, however, found that the layer of peat and silt was so tough that wells could be sunk through it only with the aid of divers, and that furthermore the infiltration of fine sand from the layer of quicksand above the peat was so great that serious subsidence of the neighbouring surface occurred.

It was therefore decided to use bored piles instead of sinking wells, most of which would have been near the shallow strip-foundations of adjacent buildings, and to use grouted concrete for the piles. The piles were to be 1 ft. 2 in. in diameter and 27 ft. long. The casings were sunk, the reinforcement placed in position, and the casings then filled with river gravel from 0.2 in. to 1.2 in. in size. The grout was composed of 36 per cent. of sand with a maximum size of 0.08 in., 37 per cent. of cement, and 27 per cent. of water, all measured by volume; previous experiments with grout of this composition had given satisfactory results. Because it was thought that the grout might extend to considerable lateral distances in the layer of quicksand, and that this might hinder the sinking of the casings for neighbouring piles, it was decided that groups of four piles should be grouted at the same time. When the first groups of piles formed in this way were tested it was found that the load-bearing capacity was much less than that calculated. These piles were therefore extracted, and their condition is indicated in Fig. 77 (see page 108). It is seen that for the greater part of its length the reinforcement was not embedded in concrete at all, but surrounded only by the ungrouted gravel. Instead of being filled with cement grout the voids between the aggregate were filled only with fine sand where the piles passed through the layer of quicksand. It is thought that this was an effect of the procedure of grouting the piles in groups of four, as a consequence of which the aggregate was not grouted for periods of two or three days after the extraction of the casings. During this period the grains of quicksand with a uniformity-index of 2 and a grain size of 0.036 in. could, with the assistance of the current of the ground-water, have infiltrated into the voids between the aggregate. Consequently only the sand content of the grout was retained in the pile, the cement escaping into the surrounding soil without binding it together. Further investigation showed that the bottom of a bore-hole was filled to a depth of 3 ft. to 4 ft. with fine sand that had infiltrated into the gravel at a higher level during one night only.

To prevent the entrance of this fine sand, the piles were subsequently formed as follows. The reinforcement was enclosed for its full length in a canvas sack and then lowered into the casing. The aggregate was poured into the sack while the casing was slowly extracted, and the grout was later also poured into the sack. This was entirely successful in preventing the entrance of the fine sand, and tests showed that the piles exceeded the bearing capacity for which they were designed. (19)

(34-13). Faulty Underwater Concreting.—The collapse of a bridge over the river Oder at Gartz (20) is an outstanding example of the failure of a
structure due to bad workmanship. The structure was of reinforced concrete and comprised three bow-string spans of 125 ft. 8 in., 194 ft., and 125 ft. 8 in. respectively. It was built in the years 1925 and 1926, and one river pier and two of the spans collapsed a week before it was to be opened to traffic. The pier that collapsed had been built within a steel sheet-pile cofferdam (Fig. 79), which was 18 ft. 4 in. by 44 ft. in plan and was driven to a depth of 23 ft. below the bed of the river. Excavation was by means of underwater dredging with grabs to the level of the gravel at a depth of 15 ft. below the bed of the river, and on this bearing layer the foundation block, 15 ft. thick, was placed under water. On to this foundation block was lowered the prefabricated shuttering for the pier, and this was filled with concrete, placed under water, to a level of 6 ft. 8 in. below the mean water-level of the river. Above this level the cofferdam was
dewatered by direct pumping and the remainder of the pier was cast in the dry. When the superstructure was built, the sheet piling was cut by underwater flame at the level of the bed of the river and removed. When the last section of sheet piling, 13 ft. in length, was removed by the crane, the pier was seen suddenly to sink vertically, to crack, to tilt laterally, and finally to collapse entirely into the river (Fig. 78, page 108).

The subsequent inquiry showed that the collapse was entirely due to carelessness in placing concrete under water. The concrete varied greatly in strength. In some parts it could be pierced only by the hardest drills, whereas in other parts it could be baled out. Material gained from bores showed that some of the material contained no cement, particularly in the part of the pier placed under water. As a result of investigation the stratification of the pier was found to be as shown in Fig. 80. It was also found that the sides of the pier comprised
large quantities of a soft mixture of cement, slime, and silt, indicating that in placing the concrete the materials had separated.

The underwater concrete was placed with the aid of a tremie tube 1 ft. in diameter, mounted on a platform at the top of the cofferdam so that it could be moved in two lateral directions as well as vertically. The concrete was not placed continuously; indeed this was not always possible, because the inclined tube through which concrete was delivered to the tremie was blocked on occasions when batches of concrete were so stiff that the material would not flow along the delivery tube. When this happened the flow of concrete was started again by shaking or knocking the delivery pipe or by flushing it with water. The results were that the delivery of concrete to the tremie was intermittent and its consistency varied throughout the concreting of the pier. Also, when it was seen that the tremie pipe would be emptied due to the blocking of the delivery pipe, it was the practice to move the discharge end of the tremie to another part of the pier until the flow of concrete started again. It was specified that the discharge end of the tremie should be kept 1 ft. 4 in. to 2 ft. below the surface of the concrete, but sometimes it was above the level of the concrete already placed. The intermittent delivery of concrete, the sudden changes of the place of discharge, and the falling of the concrete through water resulted in loss of cement from the concrete to the water (see Fig. 81).

There was no control on the rate of placing the concrete. Also, in assembling the formwork, spaces were left between the boards to allow for the swelling of the timber when it was under water; these spaces were, however, so wide that they did not fully close, with the result that cement and fine aggregate escaped; this
was assisted by the increased hydraulic pressure against the shutters as the depth of concrete increased. It was also stated that the movement of the water due to the passage of vessels caused deformations of up to 8 in. in the sheet piling, with the result that river sand passed through spaces between the piles and was found amongst the mixture of cement and mud within the cofferdam.

**Defective Sheet-piling.**

Sheet-piling must be watertight and properly driven, otherwise water and soil will enter the pit through gaps between the piles, the difficulty of dewatering the pit will be increased, the piles will be more liable to be deformed, and they will be difficult to extract. This applies more particularly to timber sheet piles, which less easily penetrate obstructions than do steel piles. A suitable type of driving-cap or ram must be used. The piles must be kept in alignment and vertical by means of walings, and the driving must be such that the piles are not distorted in shape or deflected.

The driving of sheet piles more than 40 ft. long into a resistant soil to form a cofferdam, and their extraction when the work is completed, are among the most difficult problems in foundation work. Failures due to defective interlocking, insufficient bracing, and insufficient length are common. Examples are given in (32.11), (32.12), (32.13), and (32.14) on pages 78, 80, 81 and 83 respectively. (34.2). **Damage to Sheet-Piling due to Unnecessary Driving.**—In the following example the steel sheet-piling was required only to support the sides of an excavation, and watertightness was not necessary. One side (*Plate XVIII, following page 90*) of the excavation was close to a main road carrying much traffic, and the piling was tied to anchor-blocks placed well below the road. The length of piles required was 30 ft., but because piles from 45 ft. to 50 ft. in length were available these were used. The first two piles to be driven met with little resistance, and were driven from 5 ft. to 6 ft. below the required level. Later on it became increasingly difficult to drive the piles through the last 5 ft. to 6 ft. of depth. Unfortunately this change in resistance to driving coincided with a change of the resident engineer, and no one on the site was told that this extra depth was
not necessary for the safety of the wall; indeed the contrary was the case, for the
harder driving necessary to achieve the extra depth would be harmful rather
than beneficial. In the event all the piles were driven to this unnecessary depth.
The result was unnecessary extra cost of driving, but still greater unnecessary
cost was incurred in the damage suffered by the piles and in the greater difficulty
of extracting them. The bottom edges of many of the piles were seriously
damaged and distorted, some of the interlockings were deformed, and some of the
webs of the piles were cracked, all as a result of the quite unnecessary hard driving.
These parts had to be cut away, and so shortened the reclaimed piles and reduced
their value. Also, the deformation of the piles caused large quantities of adhering
sandy gravel to be brought up with them, so increasing the cost of extraction.
Plate XVII (following page 90) shows how the hard driving had damaged some
of the piles. This unnecessary driving added greatly to the cost of the work
but did not cause a failure. A common cause of the failure of sheet-piling is
insufficient depth of driving, as is shown in example (12.2) on page 13.

Faulty Waterproofing.

Waterproofing membranes of clay, although not strictly part of a foundation,
are particularly liable to suffer from the results of poor workmanship if the
supervision is not very close. This is important, for in some cases the entire
safety or the undisturbed use of the foundation may depend on the quality of
the insulation. Also, defective insulation may lead to the complete failure of a
foundation, as when faulty insulation permits Portland cement concrete to be
attacked by sulphates or other chemicals in the adjacent soil.

(34.3). Clay-Waterproofing of Bottoms of Tanks.—The water for use
in a large factory was to be obtained from a nearby small watercourse, and the
storage tanks were built close to the river upon material deposited by the river.
There are four tanks (Fig. 82) with a surface area of water of 67 ft. by 100 ft.
For reasons of economy, in the case of tanks Nos. I and II only the side walls were
built of reinforced concrete. The floor comprised concrete slabs 13 ft. square
and 4 in. thick, jointed with asphalt applied hot. Under this was a layer 4 in.
thick of a sandy-gravel filtering material, and below this a layer of clay 6 in. thick
that was intended to be waterproof (Fig. 83). However, the properties of the
clay were not properly specified and the material was not properly placed, with
the result that water escaped when the tanks were filled. It was then found that
a local but not sufficiently colloidal clay had been used and was not properly
granulated and wetted so as to ensure effective compaction.

The bottoms were removed and remade as shown in Fig. 84. The underlying
river-drift was first thoroughly compacted by surface vibration to provide a firm
base for the whole structure. Clay of a colloidal nature, suitably granulated and
moistened, was obtained from a brickfield and compacted to a thickness of 1 ft.
with the aid of flat tampers and mechanical rams; the thickness was increased
to 3 ft. at the junctions with the walls of the tanks. On the clay was placed 4 in.
of sandy gravel to act as drainage, and this was covered with a reinforced concrete
slab 6 in. thick with fewer joints. The joints were filled with hot asphalt, which
was protected by a hard bituminous paste. Observation tubes, as shown in
Fig. 82, were placed inside and outside the tanks so that the differences in the
water levels could be measured. Tests showed that these tanks were satisfactory,
and that the other two could be constructed in the same manner. This example shows the importance of a precise specification and of good workmanship when the quality of an important part of a structure is mainly dependent on the manner in which the work is done.

**Removal of Water.**

Sumps should be to the full depth from which the water is to be drained, and preferably they should be outside the main excavation. The surface-drainage channels leading to the sumps should be of sufficient capacity to ensure that the
whole of the working area is continuously drained. It is better to have several small pumps rather than one large one, as the extra capacity will be necessary in the initial stages and the consequences of the breakdown of one pump will be less serious. This is of vital importance, because if the pumping should stop not only will seepage of water into the pit damage the stagings, shuttering, unhardened concrete, and so on, but the consequent loosening and liquefaction of the subsoil will increase the inflow of water at later stages of the work.
PART IV

FAILURES DUE TO EXTERNAL INFLUENCES

This part deals with failures due, either directly or indirectly, to natural causes, such as changes in the resistance of soil, changes in water pressure, scouring, the sliding of soils, changes of temperature, and biological effects.

Defects due to Ground-water.

The presence of water in the soil has a material influence on the manner in which foundations are constructed. Also, as a constituent of the soil, water has an important influence on its strength and structure. The level of ground-water, the amount of water contained in the soil, and the velocity of its flow are all liable to change during construction as well as after the foundation is built, and such changes may seriously influence the method of construction and also affect the safety of the structure. Attempts to anticipate such changes may influence the design and method of constructing a foundation. Among the problems to be considered are the loss of ground due to the scouring action of the seepage of ground-water, changes in the level of ground-water, the effect of floods, and the changes in the structural properties of the soil with changes in water content. Examples of failures due to most of these causes are given in the following.

(41.11). Scouring due to Seepage of Ground-water.—The main Customs House at Budapest was built in the year 1890 on a concrete raft 4 ft. 2 in. thick laid on an alluvial river-deposit of sandy silt 11 ft. 8 in. thick. In the early 1920's cracks appeared in the main walls at the north-west corner, and the floors settled several inches. The cracks and settlements increased to such an extent that this part of the building could not be used.

It was found that scouring of the soil due to the seepage of the ground-water had resulted in the formation of voids under the raft, and that these were the cause of its differential settlement and consequent cracking. The seepage was due to the natural changes in the level of ground-water near the watercourse, and which followed the changes in the level of the river. When the water in a river is low the ground-water in the banks will move towards it, and the speed of this movement will be greatest in the soil nearest to the river. On the other hand, when the river is high the direction of flow of the ground-water will be reversed, and the level of the ground-water will be raised. The intensity of the seepage of the ground-water, and the amount of scouring of the soil, are greatest when the river falls suddenly after a flood. The effects mentioned in the foregoing are illustrated in Fig. 85.

The soils under the foundation are indicated in Fig. 86. The foundation is at a distance of 200 ft. from the bed of the river, and it is obvious that the velocity of the ground-water could be great enough to wash away the silty sand and loose sand that occur to a depth of 3 ft. 4 in. under the foundation. The differences in the flow of the ground-water with changes of the level of the river, as shown in Fig. 85, were the cause of the settlement being greater at the north-west corner than at other parts. It was also found that a main sewer adjacent to the
north-west corner was cracked, and the flow of water from this sewer towards the river was no doubt responsible for increasing the amount of soil removed from under this part of the foundation.

The method of ensuring the stability of the structure is shown in Fig. 86. A trench was excavated to a depth of 25 ft. along the end and part of one side of the building at the north-west corner, and in this were driven walls of steel sheet piles to a depth of 33 ft. The trench was then filled with puddled clay to the
level of the top of the foundation raft and with the excavated material to ground level. Also cement grout was injected into the soil below, with a view to filling the voids caused by the scouring. About 200 holes of 2 in. diameter were drilled through the floor of the cellar and the underlying concrete raft, and 1 : 3 cement-sand grout containing about 250 tons of cement was pumped into the soil at a pressure of from 3 to 4 tons per square foot. Thirty years after this work was done there were no further signs of settlement.

(41.12). Fluctuation of Ground-water.—According to the results of experiments made by Professor-Dr. J. Jáky, the mere oscillation of ground-water may cause considerable compression of a cohesionless soil supporting a load. The magnitude of the compression so produced depends upon the magnitude of the applied load, the compression increasing to a maximum with a certain increase of load but then decreasing as the load is further increased. When the level of the water rises at a constant rate the amount of settlement will be related to the amount of consolidation. The rate of settlement also depends on the density of a cohesionless soil; for example, with a relative density of 0.57 the settlement was two or three times as great as when the relative density was 0.88. The amount of compression also depends upon the size and distribution of the grains of the soil; for example, a loose sandy gravel with grains of mixed sizes had a specific compression value of 4.5 per cent., a pea gravel of uniform size had a compression value of 3 per cent., and a coarse sand had a compression value of 2 per cent.

The quay wall near the Customs House referred to in example (41.11) was built in 1880, and is shown to the right of Fig. 86. The wall was built on a block of concrete placed within a sheet-piled cofferdam, and the timber piles were left in position as a protection against the scouring action of the river. In the 1930's it was seen that a length of about 533 ft. of the wall was tilting towards the river, the greatest displacement at the top being 6 in. to 8 in. It is seen in Fig. 87.
that the bottom of the foundation was a little above the lowest water-level of the river. The probable cause of the tilting of the wall is that the short timber piles did not reduce the velocity of the seepage of the ground-water sufficiently to prevent fine particles of the soil below the foundation from being washed away at times of lowest water level in the river. The broken stone placed against the face of the foundation was also frequently washed away by the current of the river, and had no effect in reducing seepage from the river to the soil under the foundation. The trouble was aggravated by the wash from vessels using the river, which increased, by wave-action, the lateral movement of the ground-water. In the year 1939 this part of the quay wall was strengthened by the construction of two rows of bored piles which extend to a depth of 20 ft. to 23 ft. below the foundation (Fig. 87). The piles were made with grouted concrete and were capped with a continuous reinforced concrete beam which is tied back to anchor-blocks at intervals of 15 ft. So far the wall has not tilted farther.

(41.2.) Absence of Weep-holes.—The retaining wall shown in Fig. 88 was built by the Turkish Government at Baghdad (22) in the year 1880. It is on the left bank of the river Tigris, and was built to protect some large office buildings. The wall was built of excellent masonry, but no weep-holes were provided. The difference in the levels of the river is up to 30 ft., and considerable scouring occurs during floods. As a result of high floods in the year 1923, a number of pools of water, which almost surrounded the city, remained when the river had subsided to its normal level. Several months later cracks appeared in the walls of the office buildings, and when the river was at its lowest level the river-wall for a length of half a mile collapsed, together with all the adjacent buildings. It was first thought that the wall had overturned as a result of the earth being scoured away from under its front, but it was found in fact that the wall had tilted away from the river as shown in Fig. 88. It seems to have been agreed that the collapse was due to water from the inundations seeping back to the river when its level had fallen. As no weep-holes had been provided, this water had to pass under the wall, where it washed away the finer particles of the soil and so reduced
its bearing capacity. It was also thought that the soil behind the lower part of
the wall had been brought to a semi-liquid state, and that its pressure forced
the lower part of the wall towards the river, as shown in the sketch. It is obvious
that this collapse was due solely to the absence of weep-holes in a well-
built wall, for no trouble was experienced with other quay walls in Baghdad
which were generally badly built with many holes and cracks and unfilled joints
through which water could pass freely.

(41.3). Oscillation of Water-Level.—The effect of the oscillation of
water-level on the compaction and settlement of cohesionless soils is discussed on
page 118, and a failure due to this cause occurred during the construction of a pier
of the Severins bridge at Cologne. In the construction of the pier near the
right bank of the river, a cofferdam of steel sheet piles was formed in the bed of
the river, which was a sandy gravel. The cofferdam was filled with similar
material dredged from the bed of the river and graded in order to ensure that it
would be as dense as possible. On this artificial island was constructed a rein-
forced concrete caisson about 100 ft. long by 30 ft. wide, the shuttering being
built up on timber beams laid on the compacted filling within the cofferdam.
When the shuttering was removed, the sill-beams were removed in a sequence
that was calculated to prevent the caisson from tilting. However, when nearly
all the sill-beams had been removed from the longer sides, and the caisson was
supported on only fourteen sill-beams at the middle, the structure tilted towards
the middle of the river. At this side the cutting-edges of the caisson penetrated
deeply into the filling, and only the presence of the sheet-pile wall of the cofferdam
prevented the caisson from overturning into the river; this accident caused some
loss of life. The caisson was returned to a vertical position by excavating soil
from under the opposite edge, and was sunk without further difficulty.

While the caisson was being built the river was several times in flood, and it
is the opinion of the writer that the accident was due to these changes of the
level of the river scouring away the finer particles of the sandy gravel within the
cofferdam.

No doubt, when the river was in flood the soil within the cofferdam became
saturated, and the level of the water within the cofferdam sank at a slower
rate than the level of the river, due to the presence of the sheet-pile wall. The
result was that the pressure head caused seepage which washed out the finer
particles of soil through gaps between the piles, and that this loss was aggra-
vated by the suction effect of the flow of the river, which varied between 6 ft.
and 10 ft. per second. When further floods occurred in the river the soil within
the cofferdam was again loosened, and more fine particles were lost when the river
subsided. The result was the formation of voids within the cofferdam, but there
was no settlement, due to the arching effect of the larger grains. However, when
the weight of the caisson was imposed, the "arches" broke down and the soil
subsided. The reason why the caisson settled on the side nearer to the middle of
the river was that the current was faster along this side with consequently greater
suction and greater loss of fine particles of soil.

It should here be mentioned that such failures may not always be due to the
loss of finer particles from cohesionless soil. The example in (43.1), which is dealt
with on page 126, is a case where failure resulted from the saturation of a cohesive
soil.
(41.4). **Change of Use of Adjacent Land.**—Damage to structures can be caused by a change in the properties of soil due to agricultural activities as is shown in the case of a building at a monastery at Zirc, Hungary, which had been safe for a period of ninety years and suffered damage only when adjacent pasture land was ploughed for growing corn. In this district there is a layer of loess 23 ft. thick, overlying rock. The foundations of the ancient monastery are on the rock, but the foundation of an adjoining library built in 1840 was in the loess some feet above the rock. At that time the loess was very hard, and could be removed only with pickaxes. The change in use of the adjacent land took place in the autumn of 1930, and in the spring of 1931 ground-water appeared in the cellar of the library, sudden settlement occurred, the library separated from the church against which it was built, and cracks appeared in the marble staircase. The compressibility of most soils is affected by their water content. In this case the rainwater and the water released by the spring thaws had previously run off the slopes of the pastures, but after the land was ploughed the surface-water infiltrated into the soil vertically until it reached the rock and then passed, on the surface of the rock, laterally under the library on its way to the bottom of the valley. Soil into which no water had penetrated for at least ninety years thus became saturated and its load-bearing capacity was consequently reduced.

The remedial measures included the provision, at the level of the rock, of a drain in which the water was collected before it reached the site of the library and which led it to a creek at the bottom of the valley. The area of the bases of the columns was increased by underpinning in order to spread their load over a larger area of the softened soil. This work also stopped the water from entering the cellar, and there has been no more settlement.

(41.5). **Unexpected Saturation.**—The Geological Institute of the University of Belgrade is another example of damage to a building as a result of the unexpected saturation of loess soil. The building (Fig. 89, page 122) is tee-shape in plan, and no trouble was experienced until some years later, when serious cracks occurred at the junction of parts A and B and the end wall of part A settled distances between 10 in. and 1 ft. 3 in. The foundations are on a layer of loess 33 ft. thick, and it was found that the settlement was due to the saturation of this soil as a result of a change in the disposal of surface water due to developments near the building and also to the cracking of a sewer near the foundation of wall A. Further settlement was prevented by underpinning the foundation with a row of piles formed of precast elements pressed into the soil by hydraulic jacks.

**Damage due to Floods.**

Sudden floods can be very destructive due to the great and uncontrollable pressure of large volumes of water moving at considerable speeds. Not only are floods a source of danger to completed foundations, but they are also liable to inundate foundation works in progress. In districts where floods occur frequently the maximum loads and stresses they may be expected to produce should be taken into account in the design of structures to be built in such areas.

(42.1) **Scouring due to Floods.**—The effects of the flow of large volumes of water are to be seen in the case of a pier of a road bridge over the river Tisza at Vásárosnamény, in Hungary, and this is typical of many other cases of piers in rivers. The bridge, shown by broken lines in Fig. 90, was built in 1886 and is
Fig. 89.

Fig. 90.
a three-span truss of the Gerber type. The side spans were of the upper-deck lattice-girder type, whereas the middle span was of the lower-deck Patton type. The three piers were built on timber piles from 50 ft. to 54 ft. long, driven into a layer of stiff clay and capped with a thick concrete slab, the bottom of which was at the level of the bed of the river. These capping blocks were constructed within a cofferdam 15 ft. wide formed as a crib-wall of two rows of timber piles spaced
FOUNDATION FAILURES

8 ft. apart and extending into a layer of loose fine sand. The space between the two rows of piles was filled with watertight clay-puddle (Fig. 91). Large quantities of broken stone were heaped against the foundation blocks as a protection against erosion. The water was removed from the cofferdam by pumping from sumps. The walls of the cofferdam were sufficiently watertight, but the loose fine sand surged up at the bottom of the excavation due to the considerable upward pressure on the loose fine sand. This upsurge from the bed of the river was accentuated by the vibration caused by the driving of the piles within the cofferdam, and the result was a considerable settlement of pier No. II (Fig. 90). As a protection against erosion, heaps of broken stone were placed around the cofferdam and on the adjacent bed of the river.

The construction of the piers and the presence of the heaps of broken stone reduced the cross-sectional area of the watercourse and increased the velocity of the current, and this in turn increased the erosion at the piers. As a protective measure further quantities of broken stone were heaped against the piers. As a result the velocity of the current was further increased, and during the period 1886 to 1922 the bed of the river was lowered by as much as 36 ft. near pier No. II. This was discovered in 1922 when a survey was made in connection with the proposed replacement of the middle span, which was demolished during the war of 1914–18. Further quantities of broken stone were then tipped against pier No. II, but this could not provide a long-term protection.

In the year 1934 a further survey disclosed that the effects of erosion and scour had become so serious that the stability of the bridge was endangered, and it was decided to enlarge the cross-sectional area of the river by constructing an additional span of 140 ft. towards the right bank as shown in Fig. 90. The original abutment was reconstructed to act as a river pier (No. I in Fig. 90) and a new abutment was built on this side of the river. This arrangement seemed to be effective; there was no further lowering of the bed of the river, and ballast was deposited to a depth of 8 ft. near pier No. II.

During the war of 1939–45 the whole of the superstructure was demolished, and it has since been rebuilt according to the design shown in full lines in Fig. 90. This provides a central span of 340 ft. between piers Nos. I and III. Pier No. II was used as a temporary support while the new welded-steel span was being erected, and was demolished later. The work of removing pier No. II started with the extraction of the sheet piles forming the cofferdam, and as soon as these were removed the foundation block cracked and the whole of the pier sank. It was shown that the safety of the pier was endangered by the scouring and erosive action of the river, and also that the concrete in the foundation-block was of poor quality as a consequence of the upsurge of water due to unsuitable dewatering of the cofferdam by pumping from sumps, against a large pressure-head in loose cohesionless soil.

(42.2). COMBINED FLOOD AND SEEPAGE.—A pump-house built in the year 1898 on the dike at Rudolfsgnád, Yugoslavia, on the left shore of the river Danube collapsed as a result of an exceptionally high level of the river. It is remarkable that the pump-house was built where the dike was closest to the river, the bank of which was only 80 ft. distant. The water was discharged through a cast-iron pipe of 4 ft. diameter and 175 ft. long surrounded with concrete 7 ft. thick, the bottom of which was 33 ft. higher than the level of the river at low
water. From the end of this pipe the water was carried to the river in an open channel 87 ft. long and with a width at the bottom of 10 ft. At the pump-house the pipe started from a reception chamber which was connected directly to the pumping-shaft, the bottom of which was 6 ft. below the intake of the pipe. As is indicated in Fig. 92 the load of the dike was unevenly distributed over the pipe and this was accentuated by the arrangement of the platform provided for operating the outlet-valve, inasmuch as it formed an additional earth-filled embankment 53 ft. wide adjoining the slope of the dike towards the river. One end of the pipe and its concrete casing was thus rigidly supported at the pump-house while the other end rested on loose soil only. In addition it carried large and unevenly distributed loads which produced high bending stresses. The changes of the level of the river caused seepage of the ground-water, which moved towards the pump-house at high water and back to the river at low water. This repeated reversal of the direction of the movement of the ground-water no doubt loosened the layers of silt and fine sand under the pipe, and several cracks appeared in its concrete casing. An approximate calculation (see bending-moment diagram in Fig. 92) indicates that the largest cracks should have occurred near the valve-chamber. These cracks would have permitted the entry to the pipe of fine particles of soil when the river was high, and these would be flushed out to the river when the collected water was discharged through the culvert to the river at low water level. At periods of low water some of the water passing along the pipe would also escape through the cracks and wash away the fine particles of the soil under the culvert. During periods of floods the direction of the flow of the seepage was reversed and particles scoured from under the culvert again entered the pipe at a greater pressure-head, because the water was continuously
removed and its level in the pumping-shaft kept low by an auxiliary pipe-line under the crest of the dike. This seasonally-recurring process increased the erosion of the soil under the culvert. As a result of this deteriorating process, in the year 1907 a high flood broke through the dike at the site of the discharge pipe, resulting in its complete collapse and the inundation of a large area of valuable agricultural land. A break occurred between the junction of the pipe with the open channel shown in Fig. 92 and the deeper foundation of the pump-house, which remained intact.

The lessons to be learnt from this failure are: (a) Foundations of such culverts must be of uniform stiffness; (b) The distance between the pump-house and the river into which the water is discharged must be as great as possible so that the velocity of the discharged water, and consequently its eroding action, are lessened; (c) Platforms from which the valves are operated must be on separate reinforced concrete frames, and not in the form of an adjoining embankment; (d) Culverts should be either perfectly rigid so that they will safely span over any unequal settlements along their whole length or they should be so flexible that they can deflect with unequal settlements without cracking. As is seen in the case just described, the development of cracks in a culvert increases the scouring action of seeping ground-water and may lead to collapse.

Changes of Water-content of Soil and the Application of Additional Loads.

An increase of the water-content of a soil will reduce the angle of internal friction \( \phi \) and its cohesion, and this results in an increase of the active pressure of the soil represented by its factor \( K_a \), that is

\[
K_a = \tan^2 (45^\circ - \frac{1}{2}\phi) - 2c \tan (45^\circ - \frac{1}{2}\phi).
\]

This in turn reduces the bearing strength of the soil as represented by the coefficient of passive earth pressure \( K_p \), that is

\[
K_p = \tan^2 (45^\circ + \frac{1}{2}\phi) + 2c \tan (45^\circ - \frac{1}{2}\phi),
\]

which is an essential constituent of the resistance-factors of all soil-rupture formulæ. The reduction of the shearing strength of soils with increasing water-content may also result in a reduction of frictional resistance and lead to sliding of the soil.

The properties of a soil may also be changed by the loosening effect of adjacent excavations or by vibration. The removal or reduction of surface loads, adjacent excavation, or upward pressure in the subsoil may reduce the shearing strength of a soil, as also may extra compaction due to vibration or lateral pressure from the surrounding soil, as, for example, the pressure of back-filling.

(43.1) Increased Water Content of Soil.—The collapse of a reinforced concrete bridge of 40 ft. span built over a tributary of the river Tisza is an example of a failure due to the effect of additional water-content of a soil increasing its lateral pressure and reducing its load-bearing capacity. The level of the water in the tributary is controlled by the level of the water of the Tisza. It was decided to provide open reinforced concrete caissons which would be sunk to a layer of harder clay at a depth of 13 ft. 4 in. under the buttresses. However, at the request of the contractor, it was agreed that the foundations of the abutments be constructed as a solid block of concrete in a braced pit, from which the water
would be removed by pumping from sumps. As the excavation proceeded the
loose alluvial clay penetrated (the ratio of voids was 1.10) became more and more
plastic and had a completely remoulded condition due to the water entering the
pit, until it was in a nearly liquid state. When the lower clay was reached the
concrete was placed without ascertaining either the thickness or the angle of
slope of the layer of clay. Thus the bottom of the concrete was actually 3 ft.
above the depth that had been specified. On completion of the bridge, back-filling
was placed against the vertical ends of the frame to form the approaches, and
this was supported by reinforced concrete wing-walls built monolithically with
the frame of the bridge. The back-filling was loose alluvial silty clay from the
site with a liquid limit of 40 per cent., a plastic limit of 22.5 per cent., an angle
of internal friction of 9 deg., a value of cohesion of 0.2 ton per square foot,
and a voids ratio of 0.96. The back-filling was placed in winter, and was not
thoroughly compacted.

Soon after the bridge was opened for traffic in the following spring, the river
rose to within about 10 ft. of the underside of the deck and the back-filling was
saturated. The flood subsided suddenly, but the water in the back-filling seeped
away slowly, leaving a back pressure-head of water which increased the horizontal
pressure on the vertical parts of the frame. Also the saturation of the back-
filling reduced its internal frictional resistance and cohesion, and this added
further pressure on the vertical parts of the frame \( K_s \) increased from 0.70 to
0.84 and \( K_p \) was reduced from 1.43 to 1.19). Still more serious was a reduction
in the shearing strength at a possible cylindrical surface of sliding from 0.28 ton per
square foot to 0.1 ton per square foot. The result of these changes in the prop-
erties of the soil, together with the higher level of the bottom of the foundation,
was as shown in Fig. 93, where it is seen that sliding of the foundation of the
right-hand abutment resulted in the breaking of the deck of the bridge. Plate
XIX (facing page 91) shows the appearance of the bridge after this movement
had taken place.

(43.2). Water-content of Stored Material.—A change in the water
content of stored material was responsible for the collapse of a wall of a tank
for the storage of iron-ore sludge with a density of 124.8 lb. per cubic foot and
an angle of internal friction of 40 deg., corresponding to a consolidated material
with a water content of 15 to 20 per cent. The right-hand wall supported earth
to a height of 23 ft., and within a few feet of the wall was a railway track for
transporting the ore-sludge. The left-hand wall was free-standing, and fixed at
the bottom to the floor of an adjoining building. The tank had four compart-
ments, and the ore was to be stored so that at the middle it was 3 ft. 4 in. above
the top of the walls (Fig. 94). After the tank had been in use for two years the
floor of the two interior compartments cracked near the left-hand wall, which
tilted to the angle shown in the figure.

At the time of the failure it was a coincidence that cast-iron pigs were being
smelted in a blastfurnace situated about 160 ft. from the wall, and an excep-
tionally heavy load was being moved by the crane travelling towards the left-
hand wall. The additional stresses in the soil due to the shocks from the blast-
furnace and the extra load on the crane could not alone have caused the col-
lapse, but they were sufficient to do so in conjunction with the changed prop-
erties of the stored material and unexpected changes in its angle of internal
friction. When the tank was designed it was assumed that the stored material would be iron ore with a water-content of 22 per cent., but in fact the water-content was always greater than 25 per cent. A water-content of 30 per cent. would reduce the angle of internal friction to less than 15 deg. (see Fig. 95). The effect of the extra water-content was also to increase the active pressure (that is the value of $K_a$) of the material and to change the direction of this pressure nearer to the horizontal. Because of the lack of drainage from the tank and the consequent slow rate of consolidation of its contents this additional pressure was
a permanent load acting on the wall because the contained material was in an almost liquid condition, and was increased because the storage-level allowed for in the design was regularly exceeded by several feet. It was also discovered that manganese ore was stored in the tank as well as iron ore, and that when it was delivered the manganese ore had a much smaller angle of internal friction than the iron ore. The plastic limit of the manganese ore was 51.2 per cent., its liquid limit was 81.5 per cent., its water-content was between 29.2 and 41 per cent., and its angle of internal friction was less than 16 deg. The material was therefore much more colloidal and slower to consolidate than the material on which the design of the tank was based, and which had a plastic limit of 25.4 per cent. and a liquid limit of 42 per cent. It was also found that when the material was subjected to a load of 1.1 tons per square foot it was brought to a perfectly liquid state, with an angle of internal friction of only 2 to 4 per cent., as a result of the squeezing out of the pore water. As a consequence the side wall was subjected to the hydrostatic pressure of a viscous fluid of higher specific gravity, and this fluid was subjected to the shocks from the blastfurnace which were transmitted through the soil and by the liquid sludge transmitted to the wall. The very great effect of the lack of consolidation of the material was shown by the fact that, while the lower layers had an angle of internal friction of 36 deg. and a weight of 137 lb. per cubic foot, the upper layers had an angle of internal friction of 3 deg. to 8 deg. and a weight of 112.3 lb. per cubic foot.

(43.3). PRECAUTIONS TO BE TAKEN IN EXCAVATING PITS.—If it is not properly drained, or if surface-water is not prevented from entering it, the properties of a cohesive soil at the bottom of an excavation can be completely changed while it is being excavated, whatever means be used for the excavation. If excavators on crawler tracks are used the tracks will sink into the soil and the depressions they leave will become filled with water. When the machine again passes over the same track the surface of the soil will be mixed with the water in the depressions and its properties will be changed. If the excavation is by hand the treading of men's boots will form depressions in which water will collect, and the nature of the soil will be changed when the next spit is taken out. This kneading process will penetrate deeper and deeper as the work proceeds and, depending upon the quantity of water and the weight of the machine and the number of times it passes over the soil, the effect may extend to a depth of 3 ft. If the concrete is placed directly on soil that has been mixed with water in this way there will be much settlement and often uneven settlement. Such settlement could be avoided by removing the soil to the depth to which it has been disturbed, but if this is done the water must first be drained from the surface or the process of removing the top layer will continue to change the properties of the soil to a still greater depth and no good will have been done. The same difficulty may occur if the soil on which the foundation is to be built becomes saturated and softened by rain or other moisture in the period between completing the excavation and placing the concrete. Professor A. Myslivec (of Prague) suggests that if such delay is anticipated the last 8 in. of the excavation be left and removed immediately before starting to place the concrete. Another useful precaution is to cover the bottom of the excavation with 4 in. of sandy gravel, which will drain away the surface-water and also help to consolidate the soil below. It is worse than useless to place large stones loosely on the foundation level, for they
will sink into the soft soil, which will then rise into the spaces between the stones and cause settlement; the result would be similar to the case of a water-bound macadam surface applied to a soil that has not been properly graded and drained. These remarks indicate the importance of surface drainage of cohesive soils in which the bottom of the drainage channels must always be below the level of the bottom of the excavation.

**Landslides.**

(44.1). **Bridge Abutment.**—An outstanding example of a failure brought about by a landslide is that of a bridge over the Peace River in British Columbia in the year 1957. The structure was part of the Alcan Highway and, although the adequacy of the foundation was doubted when it was being constructed, a risk was taken because the bridge was urgently needed in connection with the war. After fifteen years of service the northern bank of the river slid about 12 ft. towards the river,\(^{(23)}\) carrying with it the northern abutment, the first span, and the first trestle, as is seen in Plate XX (facing page 91). The next span, 465 ft. in length, over the river also collapsed. Although the top of the northern abutment tilted about 10 ft. towards the river its foundation did not appear to have moved. It was not discovered whether the earth slid on the inclined surface of an underlying layer of slate or whether the plane of rupture was in the weaker upper layers only. It was decided that it would not be feasible to repair the bridge, but that it be replaced by a structure with a continuous deck which would not exert horizontal pressure on the bank of the river, which consisted of deposits of glacial clay, gravel, and soft shale.

(44.2). **Deterioration of the Strength of Clay.**—The movement of a retaining wall \(^{(24)}\) (Fig. 96) at Wembley, near London, was probably due to the reduction of cohesion, with the passage of time, of the London clay which it was built to support. The wall was built in the year 1905 beside a railway. It was 31 ft. high and had a greatest thickness of about 15 ft., and concrete buttresses about 10 ft. long and extending into the clay to a depth of 14 ft. below the bottom of the wall were built at intervals of about 60 ft. It was not until thirteen years

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\(^{1}\)
later that a forward movement of the wall was noticed, and shortly afterwards, within less than half an hour, some 530 linear feet of the wall moved forward a distance of 20 ft., pushing the soil in front of it and displacing three of the four railway tracks. In spite of this large movement, the wall remained nearly vertical. A new wall was built to a height of only 20 ft. above the level of the rails, with an inclined base, and with counterforts at 60 ft. centres.\(^{25}\)

**Frost, Changes of Temperature, Drought, and Vegetation.**

It is generally the case that soils of more or less uniform grain-size and containing more than 3 per cent. of particles smaller than 0.02 mm., and soils of mixed grain-sizes containing more than 10 per cent. of particles smaller than 0.02 mm., are particularly liable to be affected by freezing, because in such soils the ground-water may rise by capillarity into the upper layer of soil subjected to freezing. For this reason it is always advisable to place a layer of granular material under the floors of buildings used for refrigeration or otherwise exposed to the effects of freezing.

(45.1). **Frost under a Basement Floor.**—An example of the effect of the freezing of the soil under a building is a four-story structure\(^{26}\) in Germany. The building was not completed before the winter set in, and the openings for doors and windows for the basement were temporarily covered with ill-fitting wooden shutters. The floor of the basement was 3 ft. thick, but a severe frost penetrated to the soil below and caused an upward movement of about 4 in. at the middle of the floor (Fig. 97). No such movement occurred near the outer walls without openings, where there was snow-covered soil against the walls of the basement; it is interesting to note that the soil and snow afforded a better protection than the floor of the basement in the part of the building where the door and window openings were not effectively sealed. The upward movement of the floor of the basement was transmitted by the partition walls and the temporary supports to the floor above, and cracks occurred as shown in the figure. The level of the ground-water was 6 ft. below the surface and there was silty clay at a depth of 6 ft. 8 in.; the silty clay did not have quite such a large proportion of fine particles as that mentioned at the beginning of this part (page 116), but the high level of the ground-water assisted the penetration of the frost. When the winter had passed, the basement floor settled to its original level.

This trouble would have been avoided if the floor of the basement had been constructed after the winter. In similar circumstances it is also advisable to postpone the placing of bitumen as an insulation against the infiltration of ground-water, because the bitumen would be cracked with the upheaval of the soil and consequently be ineffective. Also, if a layer of bitumen be placed under the floor and the main walls at the same time as the walls and floor are constructed, the bitumen may be broken if the walls settle more than the floor.

(45.2). **Higher Temperatures after Construction.**—The tilt of 3 ft. at the top of the chimney shown in Fig. 98 was due to the temperature of the soil under its foundation being raised when the chimney was in use. The chimney, which was at a glassworks, was 133 ft. high and was built on a raft foundation of 17 ft. diameter. The foundations of the chimney and of all the other structures at the works rest on a layer of silty Mo, under which is a layer of organic silt
Plan of cellar-floor

Cross section A-B

Fig. 97.

Fig. 98.
FOUNDATION FAILURES

3 ft. 4 in. thick. The graph on Fig. 98 (page 133) indicates the temperature of the soil at depths of 10 ft. and 23 ft. when the plant is working, and also the void ratio of undisturbed samples taken from under the foundation of the chimney. It is seen that the original void ratio $e_0$ of 0.92 was reduced as a result of shrinking to 0.67 ($e_1$) at one side of the foundation, to 0.54 ($e_2$) at the other side. The relative settlement $\varepsilon$ may be expressed

$$
\varepsilon = \frac{\Delta e}{1 + e} = \frac{e_1 - e_2}{1 + e_1} = \frac{0.67 - 0.54}{1 + 0.67} = 0.078,
$$

and the corresponding differential settlement in the layer of organic silt 3 ft. 4 in. thick may be expressed as

$$
\Delta s = \Delta e \cdot h = 0.078 \times 40 = 3.1 \text{ in.}
$$

Relating this value to the diameter of the raft and the height of the chimney the horizontal displacement was 24 in. Differential settlement was not restricted to the layer of organic silt, but also occurred in the layer above, on which the foundation of the chimney was built. The chimney was demolished and another was built farther from the factory as shown by dotted lines in Fig. 98, which also indicates the temperatures and the void ratios of the soil at the new site.

(45.3). Effect of Drought.—Much damage was done to buildings in the clay soil at Usto n. Orlici, in eastern Bohemia (Czechoslovakia), as the result of a prolonged drought in the year 1947. The effect on a school building has been described by Q. Záruba and J. Havliček, from whose report the following notes are abstracted. The clay is plastic and much fissured, and contains from 60 to 70 per cent. of particles smaller than 0.004 in. The school was built in the year 1915 and has two wings, one of which (the southern) includes the staircase and the walls of which are supported on a continuous footing extending 3 ft. to 4 ft. below ground level. In the autumn of 1947 and during 1948 serious cracks occurred in this part of the building, particularly in and near the columns and in the wall between this part of the building and the main part (Fig. 99). The cracks were between 1.2 in. and 1.6 in. wide at the tops of the columns and diminished in width as they extended downwards. Investigation showed that the coefficient of compressibility of the subsoil was between 31 and 33½ tons per square foot, that the angle of internal friction was from 28 deg. to 29 deg. 40 min. and that the cohesion was 0.05 to 0.08 ton per square foot. When the structure was built the water-content of the clay was 26.7 per cent., but after the dry season of 1947 it was much less, as is shown in line 2 of Fig. 100. The drying of the clay caused it to shrink and this resulted in the settlement of the foundation. The ratio of linear shrinkage ($R$) was calculated from the shrinkage limit ($SL$), and the settlement ($\Delta s$) to be expected was calculated from the formula

$$
\Delta s = \frac{h \cdot R \cdot \Delta w}{h + R (W - SL)}
$$

in which $h$ is the thickness of the layer of soil considered, $\Delta w$ is the decrease in water-content, and $W$ is the original water-content. This gave a settlement of about 11 in., which was sufficient to cause the cracking that occurred. It is interesting to note that no settlement or cracking occurred near test-pit No. 3, where there was no change in the water-content of the clay and consequently no
settlement. The report\(^{(27)}\) states that the drought caused shrinkage of the clay to a depth of 13 ft., but the opinion is expressed that, in view of the small effect at the lower levels, a depth of 8 ft. is sufficient for foundations on such sensitive clay soils, as the settlement due to shrinkage at this depth would not exceed 0.3 in.

This report is confirmed by numerous observations made, by Professor Á. Kézdi, of two apartment houses on shrinkable clay in Budapest. The southern and south-western sides of these buildings are subjected to settlement every summer and this is recovered in the autumn; it seems that serious trouble will not be experienced on such soils if the depth of foundations is not less than 5 ft. to 6 ft.

\((45.4)\). Effect of Roots of Trees.—It is well known that the roots of trees absorb moisture from the soil and can consequently cause damage to adjacent buildings, particularly if the soil is a shrinkable clay. The reduction of water content results in a reduction of the void content of the clay and causes settlement. This movement is reversible, and may be seasonable, because in periods when the rainfall is more than sufficient for the needs of the trees, both through the roots and by absorption through the foliage, less water will be taken from the soil. If trees are to be planted near a building their distance from the walls should be at least equal to their height, as the lateral extension of the roots is about the same as the height of the tree.

An account of the harmful effects of planting a row of poplar trees about 35 ft. from a wall of a theatre at Stamford Hill, London, is given in a paper\(^{(28)}\) by Professor W. A. Skempton. The theatre was built in the year 1928, and in 1930 the trees were planted to give some relief to the large area of plain wall at

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![Diagram](image-url)
one end of the building (Fig. 101). Some slight cracks were seen in 1935, but in
1942, when the trees were several years old, the cracks became serious and in 1947
a crack near the roof had a greatest width of 1\(\frac{3}{4}\) in. (Before this date the walls of
an adjoining room were so seriously cracked that they had to be underpinned.)
Smaller cracks appeared in the wall of the theatre, mostly near the doors, and steel
channels were bolted to the wall to act as ties across the cracks. Investigation
showed that the clay had a water content of 27 per cent., a liquid limit (LL) of
70 per cent., and a plastic limit (PL) of 23 per cent., so that it was a fairly col-
lloid clay liable to shrinkage. The bottom of the foundation was 4 ft. 6 in.
below ground level, and roots of the trees were found from 6 ft. to 7 ft. beneath the
foundations. There were no cracks in any of the other walls of the theatre,
although their foundations supported heavier loads. About fourteen years after
the trees were planted, the cracks quickly became more numerous and more seri-
ous. In Professor Skempton's view the cracks were due to settlement caused
by the absorption of moisture from the clay by the roots of the trees, which was
accentuated by the impervious surface of tar-macadam between the trees and the
wall, which prevented the penetration of precipitation. Professor Skempton
expresses the opinion that it may be safe to plant trees at a distance from a
building equal to one and a half times their height when they are fully grown,
but that a distance of less than the height of the trees may be satisfactory if the
foundations extend to a depth greater than the depth of the roots or if the bottom
of the foundation is below the level of the ground-water.

Swelling of Clay.

The National Research Institute of South Africa has made some investiga-
tions and experiments on the expansion of some classes of clay when they become
wet. (29) Cracks in old houses built on expansive clay resembled those that
occurred when the corners of a building settled, but it was found that they were
caused by the upward movement of the clay under the middle of the structures.
This is explained by the fact that the soil will be wetter under the middle of the
building, where there is no opportunity for the infiltrated surface water to be
evaporated as is the case when soil is exposed to the atmosphere. A structure was
built at Leeuhof, and samples of the soil taken at various depths under and around
the building were tested. The results are shown in Fig. 102. Pegs were driven
to various depths inside and outside the building, and their movement was observed
for a period of 5\(\frac{1}{2}\) years. The results are given in the following table.

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>Rise (in.)</th>
<th>Thickness of layer of soil (ft.)</th>
<th>Expansion of soil at depth in col. 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>3(\frac{1}{2})</td>
<td>6 to 12</td>
<td>1(\frac{3}{8})</td>
</tr>
<tr>
<td>12</td>
<td>1(\frac{7}{8})</td>
<td>12 to 18</td>
<td>1(\frac{11}{16})</td>
</tr>
<tr>
<td>18</td>
<td>1(\frac{1}{2})</td>
<td>18 to 24</td>
<td>1(\frac{13}{16})</td>
</tr>
<tr>
<td>24</td>
<td>3(\frac{3}{4})</td>
<td>below 24</td>
<td>3(\frac{1}{4})</td>
</tr>
</tbody>
</table>

It is seen that more than half of the upward movement of the pegs was due to the
expansion of the soil at depths between 6 ft. and 12 ft., and 80 per cent. was due
<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Soil profile</th>
<th>Plastic limit</th>
<th>Liquid limit</th>
<th>Plasticity index</th>
<th>Linear shrinkage</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Leached topsoil</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lateritic material</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Shattered clay, brown, yellow and grey patches containing ferruginous concretions</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>Abundance of varved clay</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>Layer of pebbles and boulders</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>Yellow bedded clay with some varved clay</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>Varved clay</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Black organic clay</td>
<td></td>
<td></td>
<td></td>
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Fig. 102.

Fig. 103.
to the expansion of the soil at depths above 18 ft. This corresponds with the
nature of the soil (see Fig. 102) at different depths, for the top 6 ft. of soil is not
of an expansive nature and the soil at depths below 12 ft. would not be seriously
affected by rainfall. Fig. 103 shows the relationship between rainfall and the
upward movement of the pegs. After the first eight months or so the relationship
takes the form of an almost straight line until the end of the third year, when
there was probably some degree of saturation, and the upward movement was
very slight during the following two years. The greatest upward movement of
the pegs inside the building was 3 3/8 in., and the average was 2 5/8 in. The least
upward movement of the pegs outside the building was 1 1/8 in., so that the difference
was 2 1/4 in., which was 93 per cent. of the average rise of all the pegs of 2 5/8 in.
The resulting differential settlement caused a great many cracks more than 1/8 in.
wide and a few of them were up to 2 in. wide in the walls of the building; the
average length of the cracks was 4 ft.

Some tests were made by the Institute on the effect of the expansive soil on
piles from 10 ft. to 30 ft. long with enlarged bases and each supporting a load of
5 tons. The piles were all cracked, due to the upward tensile forces acting upon
them above their bases.
BIBLIOGRAPHICAL REFERENCES


(14) Stockholm Tidningen, December 14, 1952.


(17) Gersevanov, N. M. "Osnovi dinamiki gruntovih mass." 1931.


(24) "Reconstruction of a Retaining Wall on the Great Central Railway." The Engineer, December 1918.


(26) Bautenschutz, 1932.


INDEX

Bearing capacities, Unequal, 42
Bridge abutment due to landslide, Failure of, 131
  " Tiling of, 61
  " anchor-block, Movement of, 23
  " Collapse of, 35, 109, 126
  " damaged by scouring, 121
  " Settlement of, 10, 21, 75
Buildings,
  Baths, Public, 50
  Boiler house, 27
  Church, 44
  Customs house, 116
  Eight-story, 96
  Factory, 53
  Garage and workshop, 54
  Generating station, 13, 92, 95
  Hospital, 5, 6
  House, 7, 61, 91, 93
  Monastery, 121
  Pumphouse, 70, 124
  Railway station, 45
  School, 134
  Slaughterhouse, 58
  Store and workshop, 42
  Steel-works, 102
  Theatre, 136
  Underpinning, 5, 6, 7
  University, 121

Caisson, Sinking of a, 74, 100
  " Sliding of a, 96
  " Tiling of a, 120
Chimney, Tiling of a, 132
Clay, Deterioration of the strength of, 131
  " Stratum, Varying thickness of, 47, 50
  " Swelling of, 137
  " Upward pressure of silty, 99
Coal store, 59
Cofferdam, Collapse of, 72, 81
  " for railway, 83
  " " silo, 39
  " " in running water, 80
  " " Unsuitable bracing of, 78
Construction, Advantages of, 58
  " of concrete in piles, Defective, 107
  " of filling, Damage by, 56
  " of loose soil, Non-uniform, 102
Concrete, Defective, 106
Concreting, Faulty under-water, 109
Co-operation, Incomplete, 16
Costly construction, 10
Craneway, Settlement of a, 16, 67

Dam, Cofferdam for a, 81
Delay in completing foundations, 86
Dewatering, Unsuitable methods of, 69
Drought, The effects of, 154

Earth banks, 86
Excavating, Precautions to be taken during, 130
Excavation, Defective support of, 87
  " Sheet-piling for, 112
Excavations, Adjacent, 44

Filling, Damage due to compaction of, 56
  " Excessive loads due to, 62
  " Unequal pressure from, 66
Flood and seepage, Combined, 124
  " Scouring due to, 121
Footings, Settlement of, 50
Forces on foundations, Horizontal, 23, 26
Frost under a basement floor, 132
Gas-holder tank, Defective piles for a, 107
Ground-water, Neglecting, 73

Investigation, Unsatisfactory preliminary, 10

 Landslides, 131
Load, Rapid application of, 30
Loads due to filling, Excessive, 61

Loads, Incomplete assessment of, 56
  " Neglecting future, 67
  " on existing foundations, Additional, 59
Machine foundation, Vibration of, 56
Materials, Defective, 106

Oil-tank, Settlement of, 30

Piers for a store, Foundation, 42
  " Underpinning a building with, 5
Piled foundation for silo, 37
  " Unsafe, 37
Piles, Changing the load on, 91
  " Defective bored, 107
  " Defective compaction of concrete in, 107
  " Different lengths of, 45
  " for a railway station, 45
  " in wrong positions, 93
  " Superfuous, 35
  " Underpinning with, 7
  " Unequal loading of, 53
  " Unnecessary driving of, 112
  " Unsuitable length of, 95
  " Unsuitable spacing of, 92, 95
Piling, Defective sheet, 112
  " Unnecessary, 37
Pressure, Excessive bearing, 21, 27
  " from filling, Unequal, 66
  " in silty clay, Upward, 99
Quay wall, Collapse of 99
  " " a cofferdam for, 78
  " " Sliding of, 96
  " " Tiling of, 118

Raft foundation for a tank, 47
  " " building, 107
Retaining wall without weep-holes, 119
Rigid foundation, Excessively, 54

Safety-measures, Excessive, 55
Saturation, Unexpected, 121
Scouring due to floods, 121
Seepage, Combined flood and, 124
Sewage works, Caisson for, 74
Silo, Settlement of a, 27, 37
  " Site investigation, 5
  " Slab-foundation for oil tank, 31
Slabs, Underpinning a building with, 6
  " Soil, Non-uniform compaction of loose, 102
  " Sliding, 19
  " The effects of ground-water on, 16
  " Variation of, 56
  " Strata of variable thickness, Bearing, 47
Stress, Overlapping zones of, 59
Superstructures, Unsuitable, 21

Tank, Collapse of a wall of a, 128
  " Tiling of a, 47
Temperatures after construction, High, 131
Temporary supports, Cofferdam of, 34
Trees, Effects of roots of, 136
Tunnel, Sinking of a caisson for a, 100
Underpinning a building, 5, 6, 7
Use of land, Change of, 121

Vibration, Effects of, 56
Water content of soil, Increased, 126
  " " stored material, 128
  " Ground, 23, 116, 118
  " " level, Oscillation of, 120
  " on soil, The effects of surface, 16
  " Removal of, 112
  " Waterproofing bottoms of tanks, Clay, 113
  " Faulty, 113
Weir, Cofferdam for a, 80
Workmanship, Defective, 106
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