Acknowledgement

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1. INTRODUCTION

The term "failure" implies not only structural collapse, but also a wide range of non-conformity with design expectations or requirements, including undesired settlements, deformations, cracks, bulges, and misalignments.

Most failures of structures are probably caused by the failure of their foundations. Although the cost of a foundation seldom exceeds one-tenth of the total cost of a structure, the security and performance of the structure depends almost entirely on the foundation. Moreover, 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.

2. CAUSES OF FAILURES

Generally, 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.
4. Insufficient provision in the design for exceptional natural phenomena such as thermal conditions, rainfalls, and floods greater than those expected to occur during the life of the structure.

Today, it is the exception rather than the rule to design a foundation without an investigation of the site. It is important that the engineer and the contractor should be aware of all the results of the soil tests, and particularly, that they should take into account the variations of the properties of any different strata below the surface.

It must be kept in mind that all the important properties of a soil cannot be accurately judged from the results of preliminary borings and laboratory tests. Unexpected changes may be detected only when the excavation is made or when the foundation is being built. It is for this reason that there must be intimate collaboration between the site investigator, the engineer and the contractor when the design is made, and throughout the time of construction.

A frequent source of defective data is the insufficient investigation of the behaviour of groundwater at the site, particularly changes in the run-off and infiltration due to changes in the surface vegatation.

In the design of large hydraulic works, an unsatisfactory design may be the result of failure to make tests on 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 (i.e. the deformation of the subsoil when it is subjected to loading, with the result that there may be an uneven settlement on soils with non-uniform compressibility). 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 have been correctly taken into account, or, proper consideration may not have been given to possible changes in the subsoil due to vibration, scour, changes in the levels of the ground water, or the erection of other structures nearby.

In the following, an attempt is made to explain the major causes of each failure examined.

2.1 SITE INVESTIGATION STAGE

2.1.1 Introduction

The exploration determines the nature and the dimensions of the underlying materials. It tells whether the soil is gravel, sand, silt, clay or a mixture of clay and sand or silt. The water table is always located if practicable. The depth and thickness of all layers are determined. If rock is encountered, the extent of its surface is defined and its thickness and nature determined.

If cohesionless material (gravel, sand, or silt) is found, it should be examined to determine if it is clean and cohesionless or it if contains a clay binder or other material. It should also be determined whether the cohesionless material is coarse, medium or fine gravel, sand or silt and whether the grains are of uniform size or graded from fine to coarse. The shape of the grains should also be noted. An attempt should be made to determine whether the material is dense or loose and, as nearly as possible, to determine the degree of density.

If the material is clay, it should be examined for plasticity. Texture as indicated by the approximate amount of gravel, sand of silt, should be noted. The natural water content should be determined, especially for cohesive soils. Some attempt should be made to determine whether the clay is a recent deposit or an old one that has been compressed by previous overburden which has been removed by erosion, and whether the clay has been subjected to cycles of wetting and drying. Consistency, odor, and color are indications of properties which are significant in guiding the judgement in the choise of a foundation, and these properties should be noted.

Usually the cheapest and most common method of examining the soil below the surface is by means of bore holes. Test pits are too expensive for general exploration, but they may be used for further and more careful examination if found to be needed after a preliminary examination. The method adopted or allowed for drilling the holes should be one that will allow the taking of samples in as nearly their natural state and possible, and one that will facilitate obtaining as much data as possible. Holes above the water table should usually be drilled dry. Obviously, any method that uses drilling water above the water table will not supply the greatest amount of information.

The depth to which the exploration is carried depends upon the type of soil encountered and the loading conditions. If rock is encountered, drilling must stop and other methods must be used for determining the quality and thickness of the rock layers.

2.1.2 Absence of Preliminary Investigation

Absence of preliminary site investigation frequently becomes one of the factors of foundation failures. Before any other task is undertaken in the design of foundation, tests have to be performed on the soil under consideration, and its behaviour understood and interpreted correctly.

The examples in fig. 1 shows a cross-section of a large hospital and its foundation where 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 had been subjected to differential settlement ever since it was built, the rate of settlement increased when the level of the ground water was raised, and serious cracks appeared in the facades over doors, windows and in the staircase.

A site investigation revealed that the subsoils indicated in fig. 1, where it can be seen that the upper layer of sand has a thickness of 2.5 ft and overlies a layer of very compressible peat from 4 to 5 ft thick, the water level rose and as a result the water content in the peat layer was increase.

It was decided to underpin the structure by constructing shallow piers extending from under the basement to the gravel as shown in fig. 1; and to support the main walls on reinforced concrete beams carried on these piers. However, while the work was underway, more mistakes were made. In order to construct the piers, shallow wells were formed from which the water was pumped out. This procedure was taken with the idea of collecting the water from several wells into the central well in order to reduce the number of pumps required. This method resulted in the upsurge of the sand layer in the well and the subsidence of the adjacent area of the ground. The ground water was finally allowed to find its former level and the wells were sunk by the dredging and placing of concrete.

There were two mistakes made in the design of the foundation. The first was that the differential settlement of the layer of peat was ignored, the second was that similar foundations were used for the outer walls and the main inferior walls in spite of the fact that the interior walls carried heavier loads than the outer walls.. The end result of these errors was that the interior of the building settled considerably more than the exterior.

Another case involves the underpinning of a building with a slab. Fig. 2 shows a building which was built on similar ground as the previous example. Strip foundations were used under the walls

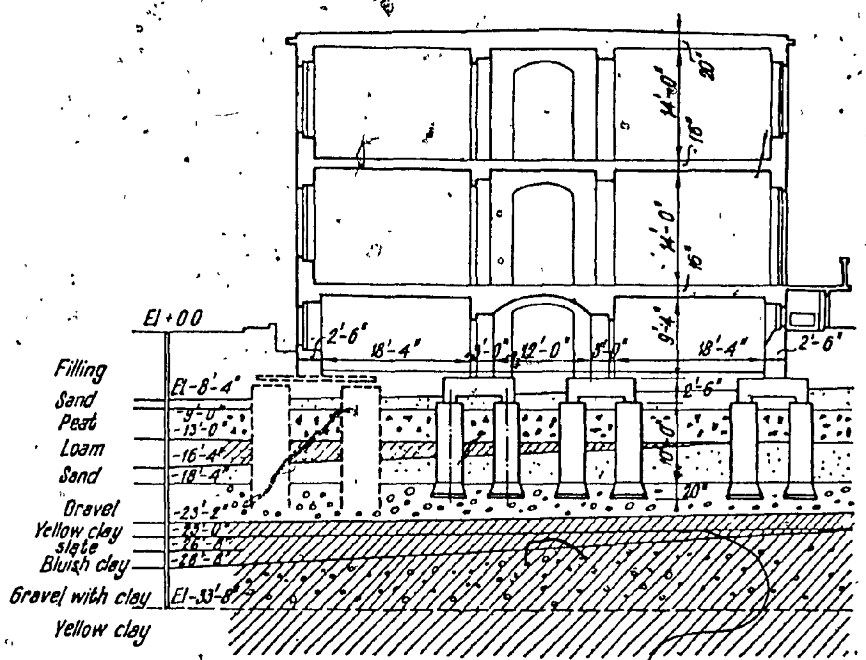

Fig. 1

This cross section of the hospital, built in a site that was a marsh area in Budapest, during the beginning of this century, shows the layers of the underground which were investigated after the building was subjected to differential settlements. The concrete piers used to underpin the building are also indicated on the left.

without an investigation of the soil below the sand layer which extended to a depth of only 5 ft from ground level and under which were layers of peat and silt. Forty years after the building was built, differential settlements of up to 1 ft occurred as well as very large cracks in the structure. The settlement was greater every spring. A result of the higher water level in a nearby creek causing an increase in the water content of the soft peak underlying the sand.

To prevent further differential settlement, a reinforced concrete raft was constructed at ground level under the entire building as shown in fig. 2. The reinforcement was placed in the areas between the strip foundations first, and half the bars in the transverse direction were passed through holes made at intervals in the footings. The slabs were concreted and the holes filled with concrete. When the concrete had hardened the remainder of the footing was broken through and the bars left projecting from the slabs were passed through from both sides. The bars were then wired together, and the holes filled with concrete. The result is a uniform pressure on the shole 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. By this method, the 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.

Settlements due to loading the homogeneous materials are always proportional to the area of the pressure bulb, but in the case

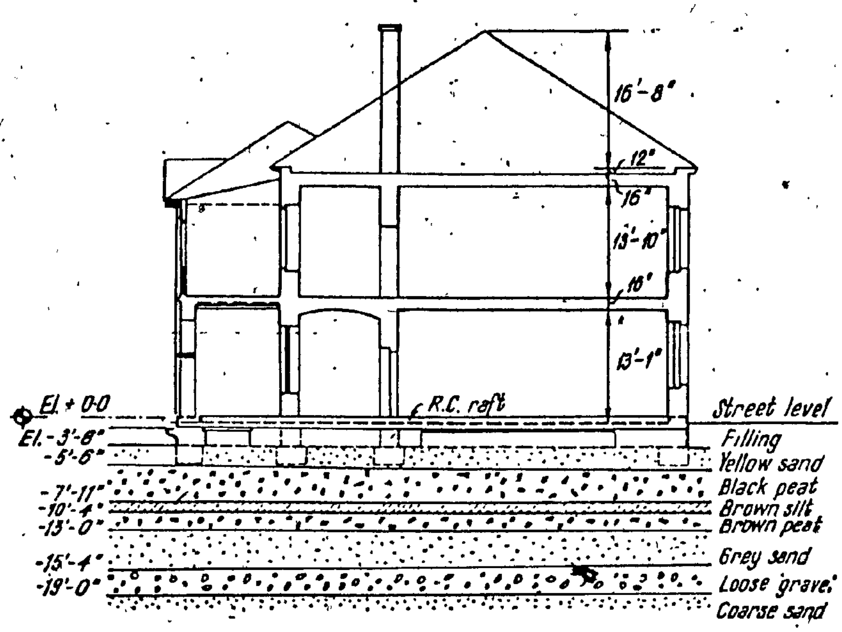

Fig. 2

This commercial building was built in 1904 in Germany. The weak subsoil layers which were investigated after serious settlements occured are shown above. Also shown in the reinforced concrete used to underpin the building.

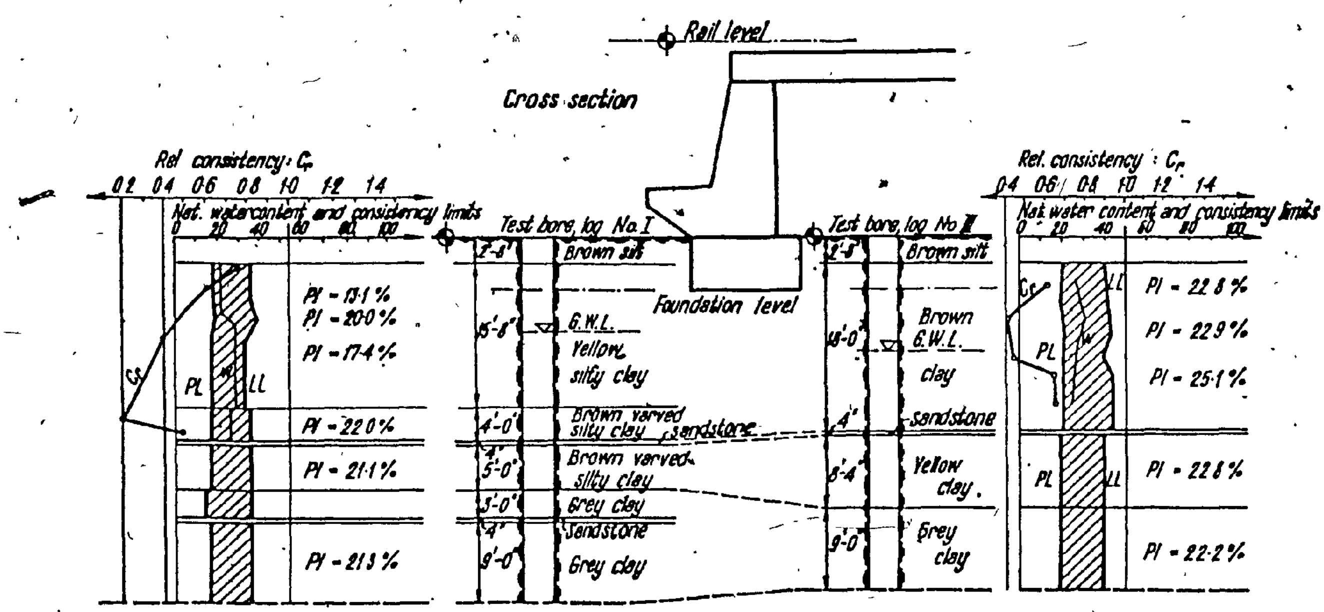

Fig. 3

This railway bridge pier was built in England in 1950. Due to a cursory investigation of the soil before construction, the pier settled considerably and was eventually abandonned. The cross section shows the results of intensive investigation afterwards.

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

$$Si = \frac{(Pi)(\Delta hi)}{Mi}$$

Where: Pi $=$ the intensity of the stress at mid-depth of the layer

Δ hi $=$ the thickness

Mi $=$ the modules of compression of the layer

2.1.3 Unsatisfactory Preliminary Investigation

A frequent source of defective data is the faculty investigation of behavior of the ground water at the site, particularly changes in the run-off and infiltration due to possible changes in the surface vegetation. 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 these factors may cause movement of sliding of the surface.

Observations of settlement were made on a railway bridge which was constructed on a weak soil foundation. The three bores indicated that there was a layer of stiff brown clay which was fairly near the surface and appeared to be uniform for a considerable depth. Based on experience, a safe pressure on this material of 3.3 ton per square foot and a raft foundation was assumed to be suitable without dewatering the subsoil. As soon as the construction of the abutments were completed, a settlement of 4 in. appeared and consequently the design of the superstructure was changed from concrete to a lighter steel design 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 with depth.

At one bore hole, the relative consistency was .75 at a depth of 3.5 ft, 0.60 at 5 ft, 0.40 at 10 ft, 0.20 at 20 ft, and 0.50 at 21 ft. The void-ratios were 0.83 at a depth of 5 ft, 0.94 at 10 ft, 0.80 at 13 ft, and 0.74 at 18 ft. The plastic limit was about 0.16 throughout the entire depth, and the liquid limit varied only from 0.30 to 0.44 as shown in fig. 3. These variations of the condition of the clay were the cause of the trouble and should have been known before the foundations were designed.

As shown in fig. 4, the plan of the foundation is irregular because the axis of the bridge is not at right angles to the railway.

The curves in fig. 5 show the settlement during a period of years (the number in circles indicate the positions numbered on the plan in fig. 4). It can be seen that the greatest settlement that occured 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

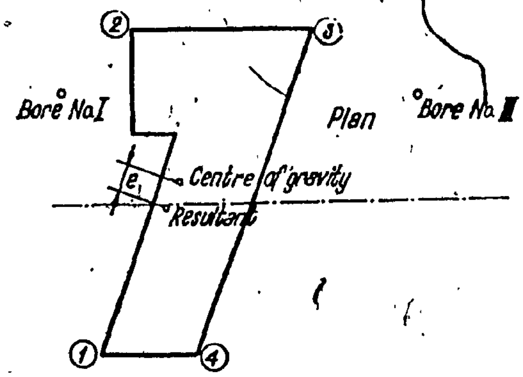

Fig. 4.

This figure indicates layout of railway pier foundation from fig. 5, with bore hole locations during the intensive soil investigation.

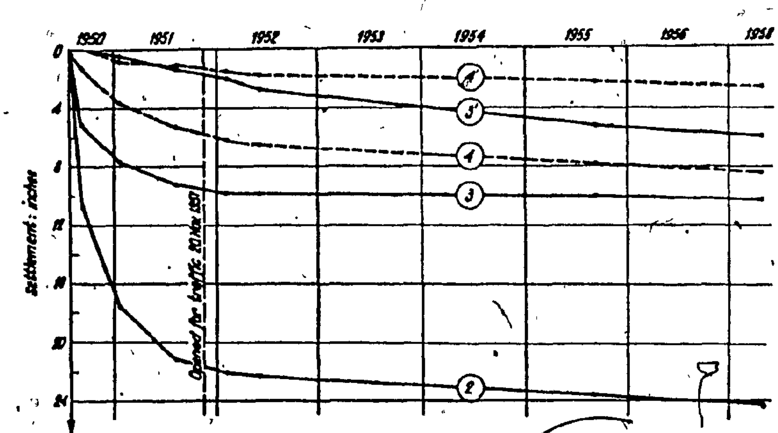

Fig. 5.

This figure idicates the results of the soil investigation of the railway pier. The numbers indicate the position numbered on the plan in fig. 4.

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 consequently less. The settlement at these places has since however, increased, probably as a result of a transfer of load or of later increase of moisture content. At (2) the settlement has exceeded 2 ft by 1958, as shown in fig. 5, and was increasing at a rate of 3/8 in a year. However, at the other places the magnitude of settlement was less than 1 ft and was increasing at the rate of 1/4 in per year. The other abutment had settled twice as much as is shown. Due to the high settlements, the bridge was abandoned, and obviously a complete loss.

Figs. 6 and 7 illustrate works comprising of three main parts each differing greatly in weight and purpose. Preliminary bores indicated sufficiently well the physical characteristics of the subsoil; but no examination was made of the ground water conditions. When the excavation reached a depth of 5 ft, ground water surged up around one of the wooden piles and the inflow increased to such an extent that the work was stopped and a different method of construction adopted.

2.1.4 Surface Water Effects on Soil

The foundations for a crane in a stockyard at a steelworks

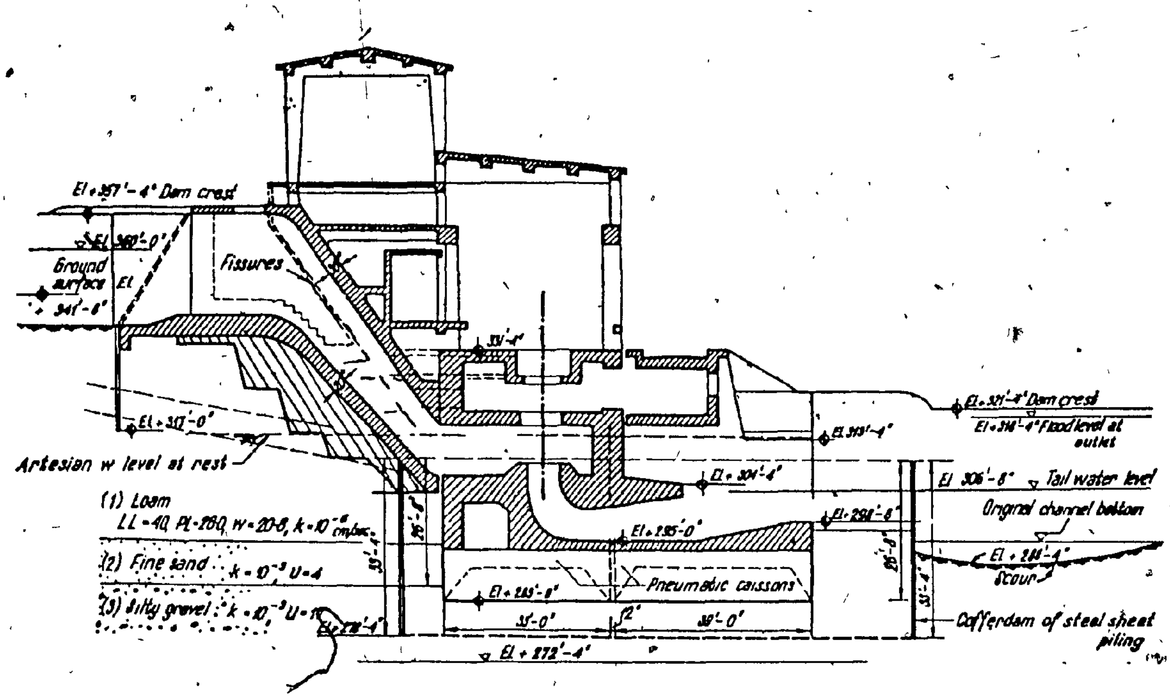

Fig. 6

Cross section of turbine-house of a hydro-electric plan, Sweden, where wooden piles were used and failed.

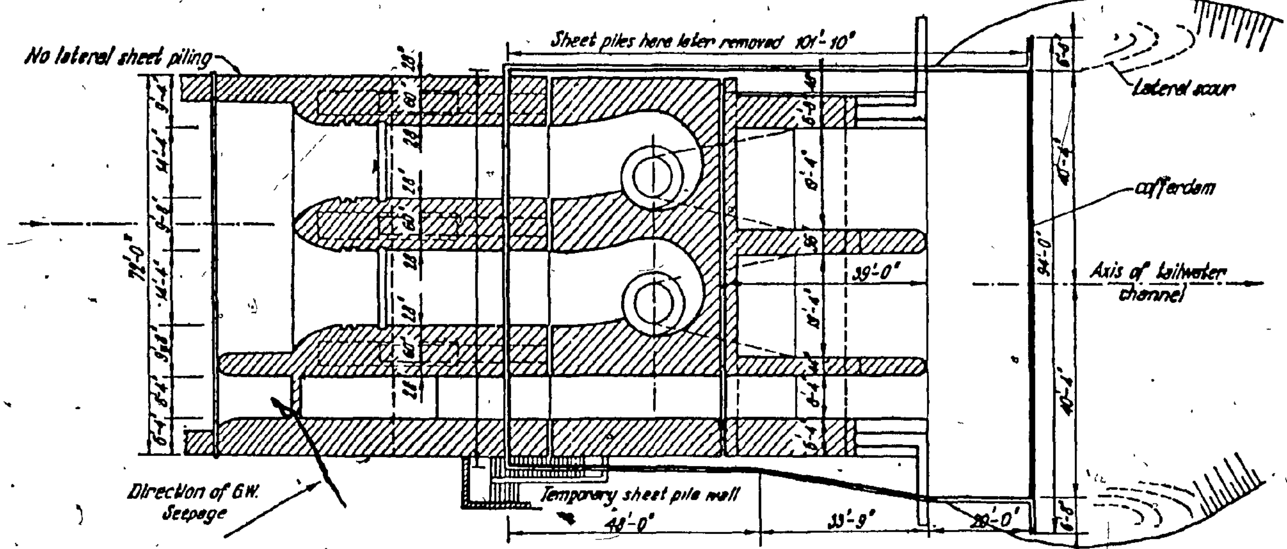

Fig. 7

Partial Layout plan of Fig. 6

is another example of failure which shows that the examination of the site is of no avail unless the results are used carefully by the designer. The soil report described the soil properties and also advised the designers on the best type of foundation suitable in the area. The bearing capacities were accurately given, together with suitable depths and dimensions for the foundations under each masonry pier, see fig. 8. The need for making exact calculations of the settlements in relation to the sensitivity of the structure were mentioned in the report as well as the need to provide drainage to divert the surface water, but there was no mention of how to deal with the problem The foundations as shown in fig. 8 are in silty sand with an angle of internal friction of 22 degrees to 32 degrees, and a void ratio of 0.61 to 0.68 and the water content exceeds the plastic limit. These properties are related to the surface water moving down the slope, which would not only cause scour but would also vary the bearing properties of the soil as the water content was reduced. The recommendations to make exact calculations of settlements and to provide drainage channels were ignored.

Differential settlement occurred during the construction of the superstructure was suspended. The settlement record is given in fig. 9, which continues after a year of completion of construction. Under pier 1 and 2, the layer of silty sand is thinner and the settlement was sudden due to the underlying cohesionless soil. As can be seen from the record of the other piers, the greater the thickness of the intermediate layer of silty sand the lower the rate of settlement. Both the parties involved in this case are at fault, the soil

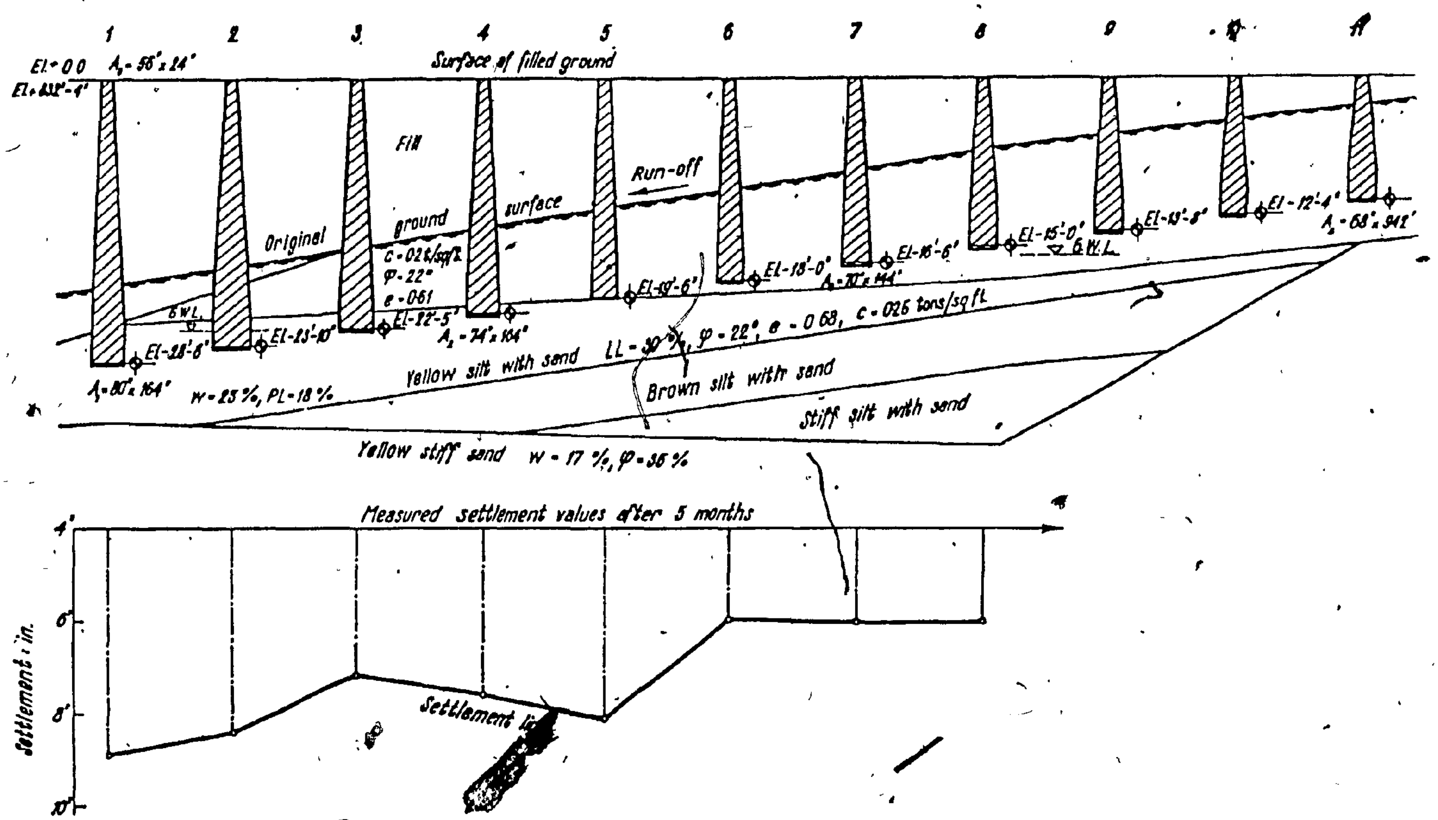

Fig. 8

Cross section of crane foundation in Germany where incomplete co-operation between design and soil report resulted in uneven settlements as shown.

investigators who did not stress the emphasis for drainage and accurate calculation for settlement so as to draw more attention from the designers, as well as the designers who did not properly investigate the soil report and pay more attention to its suggestions.

2.1.5 Neglect of Possibility of Soil Sliding

The results of disregarding the possibility of the sliding of the soil as a result of excavation is shown in fig. 9. This is a coal store in soil which, due to its geological formation, was susceptible to sliding. 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.

2.2 DESIGN STAGE

2.2.1. Introduction

A foundation is usually considered to be that portion of a structure that transmits the loads from the superstructure to the earth. But the foundation is part of the structure, and should be designed as an integral part of the structure. The purpose of the foundation is to transmit loads from the superstructure to earth in such a manner as to limit deformation of the structure to values that the materials of the structure can experience without impairing its function. By general, the foundation and the superstructure perform as a unit.

No one type of foundation, regardless of cost, is the best foundation of all conditions. In some cases, the cheapest foundation may fulfill requirements better than any other. Sometimes, as a simple matter of economy, it is necessary to use a type of foundation which limits deformation to what is permissible rather than to what is desirable. For the same soil conditions, one might use a different type of foundation under a structure that can undergo a large deformation without damage to its materials or impairment of its function, than under a structure that can withstand only a small deformation without damage. In the design of a foundation, two factors must be considered: deformation, and failure. Any load, however small, placed on a beam, however strong, causes some deflection. To fulfill its function, the beam must be designed sufficiently stiff to limit deflection to that which the materials supported by the beam can withstand without failure, and at the same time strong enough that it can carry its superimposed load without failure of the material of the beam and resultant collapse of the structure. Any load on a foundation causes settlement, even though the soil supporting the foundation is capable of supporting a much heavier load without failure. The function of the foundation is not to prevent settlements but to limit and control them. The stresses in the soil produced by the foundation loads must be low enough so that they do not cause failure of the soil under the foundation.

Sometimes a foundation is selected for its greater feasibility from the standpoint of economical and construction procedures, even though, from a functional viewpoint, another type of foundation would be a better choice. Occasionally, a highly desirable type of foundation has to be eliminated from consideration because it is impractical to construct.

Usually the most common type of foundation is the continuous foundation consisting of a continuous footing slab on which rests a foundation wall that extends to the desired height above the surface and carries the building loads to the footing. The load is transferred by the footing to the earth as two Cantilevers. If reinforcement is needed in the footing, it should be placed near the bottom perpendicular to the wall. No reinforcement is needed lengthwise in the footing. The strength lengthwise is provided by the foundation wall, which should be reinforced top and bottom to transfer nonuniform loads to the footing and to bridge over soft spots in the soil.

Although continuous foundations are commonly used for light buildings, they are one of the least suitable for general use. Probably, more foundation trouble is caused by the use of continuous foundations in soil unsuited to their use than from any other course. In dense sands and gravels, where the footing is placed deep enough to prevent exposure by erosion and to develop sufficient shear strength to support the superimposed loads, continous foundations are suitable and economical for many buildings.

Another type of foundation consists of independent footings of the proper size to transmit the building loads to ground with a constant contact pressure of to hold differential settlements within certain limits. The load is transmitted to the footings by means of columns that carry the reactions from grade beams, which take the place of the foundation wall in the continuous type.

An advantage of the independent column footing type of foundation is that the sizes of footings can be adjusted for the same or different contract pressures, and footings can be located so as to distribute the load advantageously over the site. For depths greater than 4 or 5 ft, the independent column footing foundation becomes more economical than the continuous type. All that is needed to increase the ~~depth~~ of this type of foundation is the deeper excavation of the footing area and the extension of the length of the small column.

A disadvantage of the independent column footing type as described above is the cost of the necessary hand excavation for footings.

2.2.2 Unsuitable foundations

The more frequent causes of failures are unsuitable designs and construction methods, which are due mainly to insufficient knowledge of the soil conditions at the site. The result is that foundations have been designed which cannot be constructed economically, safely or which are impracticable. When dealing with heavy structures, deep foundation could provide acceptable solution. This is, however, not always true, in some conditions deep foundations may even cause defects.

2.2.3 Excessive Bearing Pressure

Failures due to excessive pressure on the ground are becoming increasingly unusual, mainly because of the investigation of the soil at the site, and the testing of soil samples now undertaken in most cases of large structures.

A well-known failure of a foundation is that of the grain silo at Transcona, Canada. 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 of plastic clay, and was essentially a general shearing failure. The silo is 7u 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 the fact that this pressure had been used in the calculations for neighbouring structures. 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, after a thaw set in after heavy winter snow, the clay under an adjacent railway embankment, 30 ft high and composed of ballast,

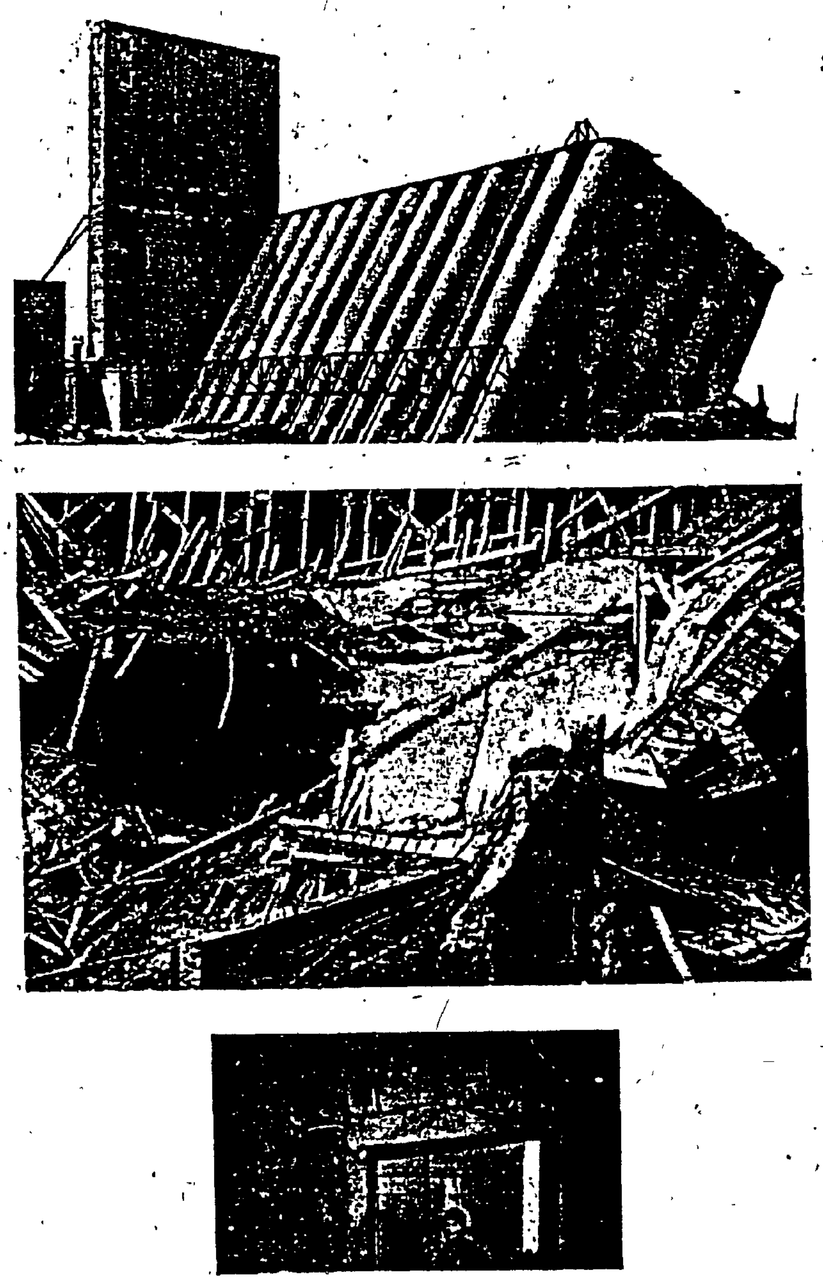

Fig. 11

Photographs showing the silos at Transcona, after they have failed.

subsided several feet and forced 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 comprised 42.5 per cent of the total weight when it was full. Filling of 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 degrees, 53 minutes from the vertical, the west side being 24 ft below and the east side 5 ft above the original level (fig. 12). 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 momolith and there were only a few superficial cracks. Fig. 11 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.

From examination of undisturbed samples of the clay 60 ft away, 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 tons 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. 13). This condition is demonstrated by the fact that the ratio of compressive strength in natural and in remoulded states in two. The bearing capacity q_n can be determined from the formula, $q_n = cN_c = 1/2\ q_n N_c$, in which N_c is $5(1+\frac{B}{5L})\ (1+\frac{D}{5B})$, in which B and L are the breath and length of the foundation and D is the depth. Substituting the data in the foregoing,

$$Nc = 5(1+\frac{77}{5 \times 195})\ (1+\frac{12}{5 \times 77}) = 5.56$$

and, with the average value of q_n, the theoretical ultimate capacity sould be 1/2 (0.93 x 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_u is 1.80 tons per square foot. It is not good practice to base a design of average values, for a weak layer will produce concentrations of stress in a stronger layer above it. Recently, Dr. A. Hanna (1978) has developed a theory for footing on a strong layer overlying a weak layer. This theory may be used for such soil conditions, it will predict less bearing capacity, and failure may be avoided.

A calculation made indicates that the rate of loading the ground

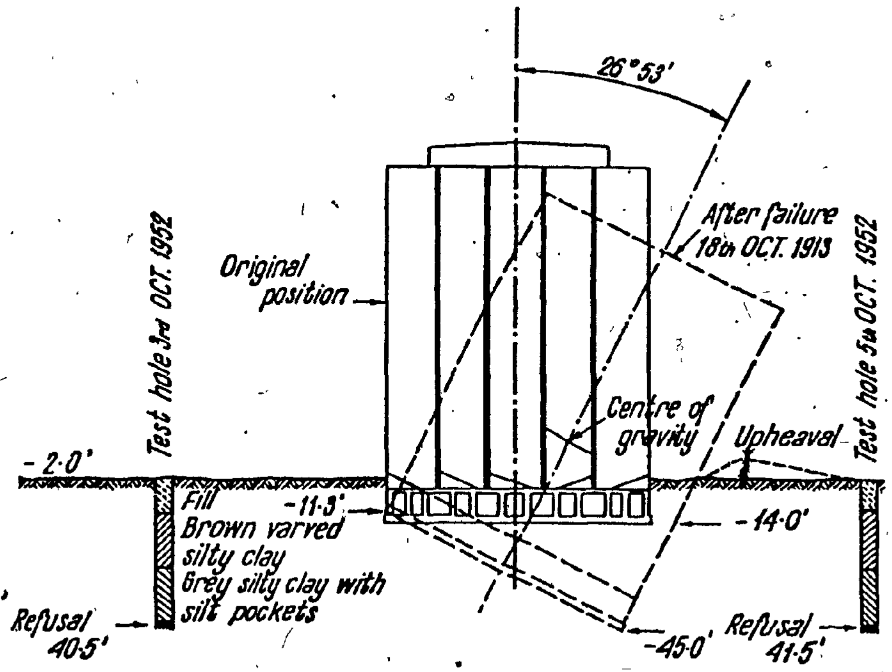

Fig. 12

Schematic diagram showing the silos before, and after tilt occurred.

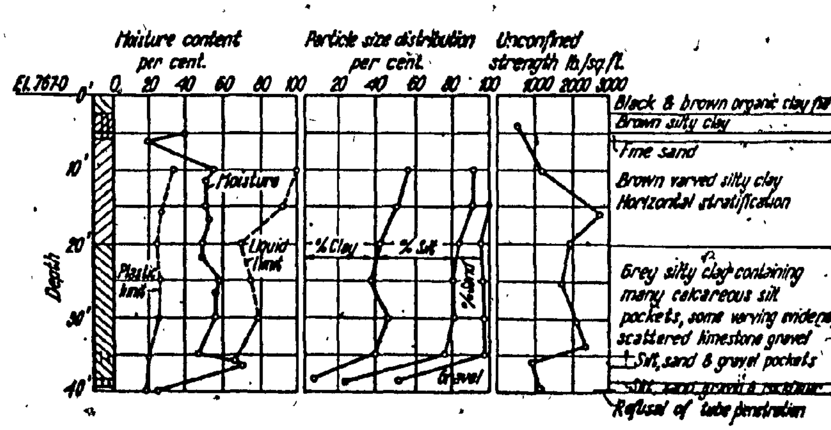

Fig. 13

Results from soil tests after the collapse occurred.

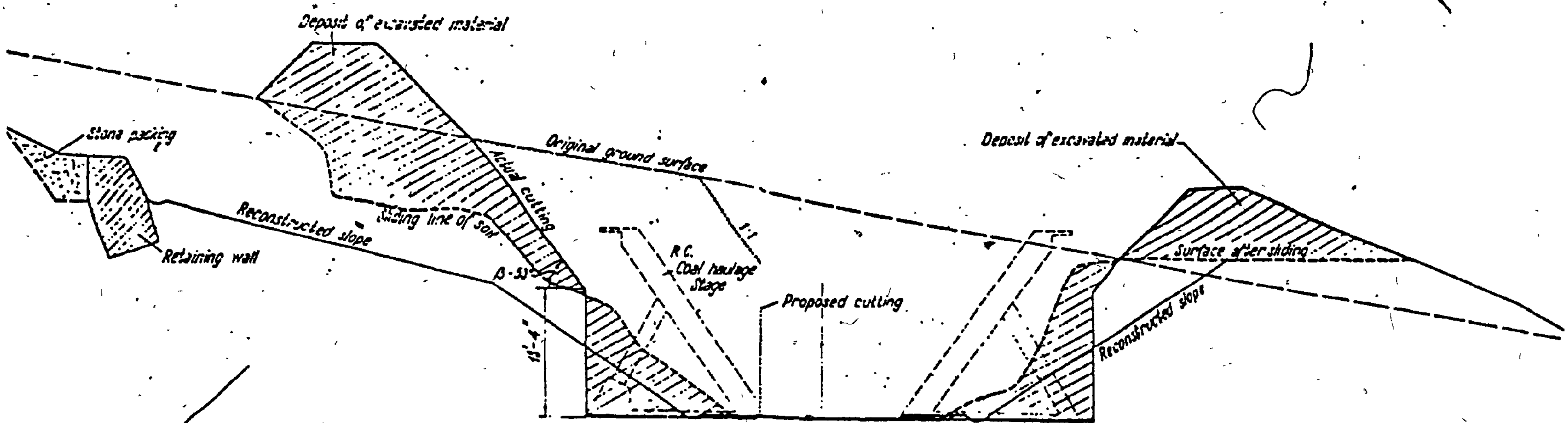

Fig. 10

Cross section of coal store area England, where sliding occurred after construction due to neglect in design.

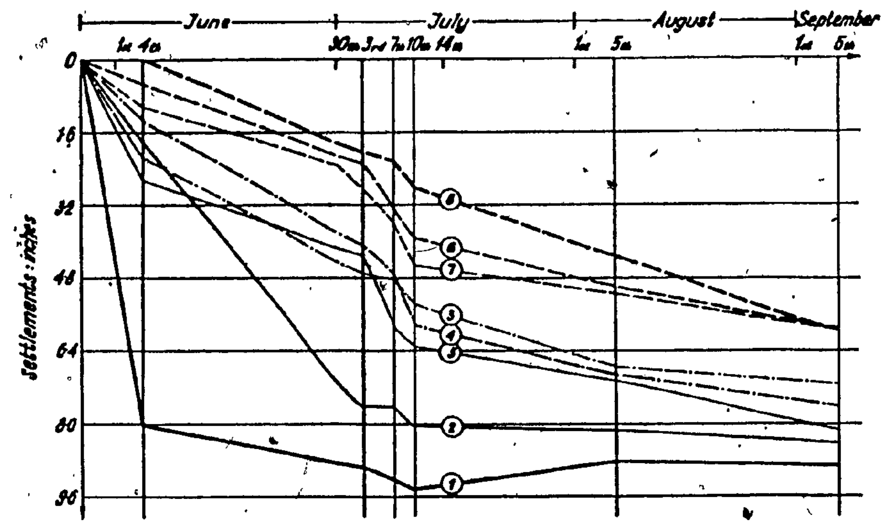

Fig. 9

Results of settlements from fig. 8.

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 collected data, it has been computed that the consolidated shearing strength would develop in about a year, wheras 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, 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 $\dfrac{\pi D \gamma}{\cot\phi - (\frac{\pi}{2} - \phi)}$ for progressively consolidated loading in which ϕ is the angle of internal friction and γ 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 progressively consolidated loading, the difference being about 20 per cent.

2.2.4 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 length of the piles are properly specified.

The reinforced concrete frame of the building in the present example was supportedon one side on an existing raft carried on timber piles 100 ft long, bearing on a layer of compact gravel, where as 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. 14), beneath which compact sandy gravel extends to a great depth. The new piles settled only 1/4 inch to 1/2 inch under loads of 60 to 100 tons, from which it was concluded that the bearing capacity of the upper layer of sand 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 therfore 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 1/4 in. to 5/8 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 as well as its fairly high water content.

The remedial measures incorporated one of the first applications of prestressing and curing concrete. 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. 14) 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 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

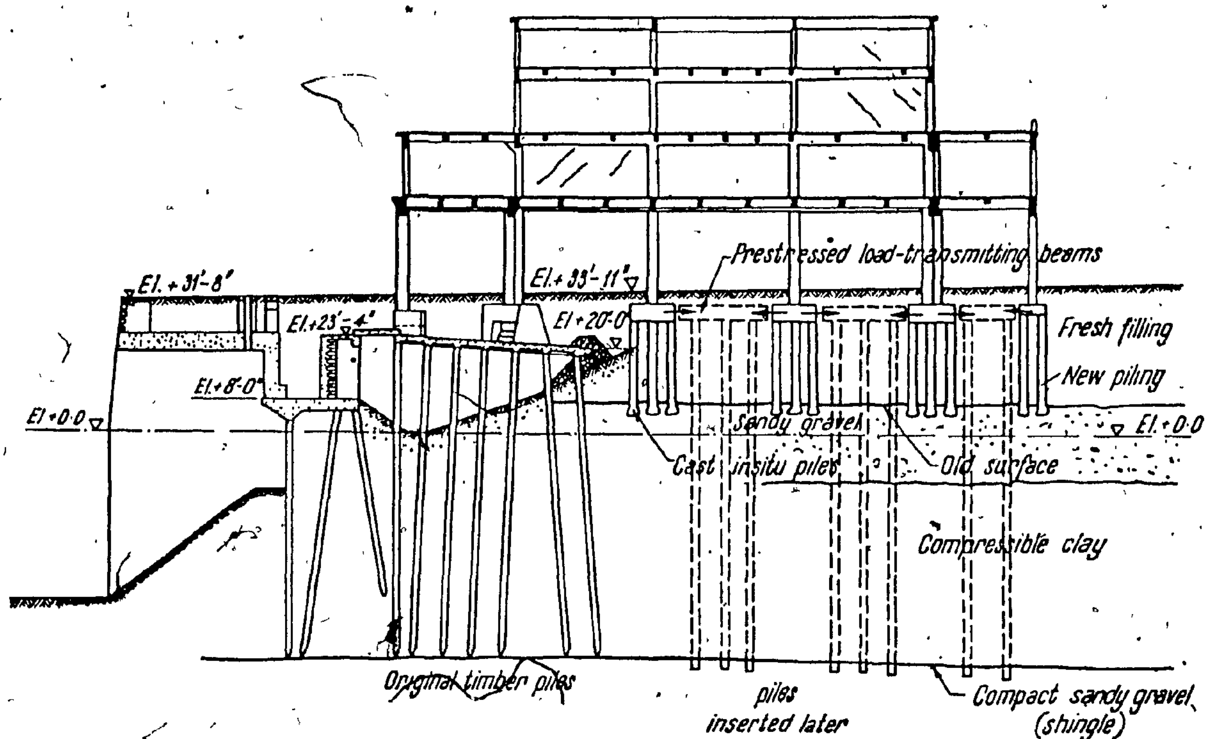

Fig. 14

Pile foundation industrial building at Le Havre France, 1932. Showing are the old and new piles with ore-stressed concrete beams used for remedy.

being forced into the ground by a force of 320 tons.

2.2.5 Gravel Overlying Clay of Varying Thickness

Another case occurred in connection with the foundation of the main building of a public bath. The building, the plan of which is a tee, is founded on shallow strip footing extending to just below the frost-line, where the ground is 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. 15). It was evident that the cracks were caused by differential settlement due to the compression of the layer of clay, which had a 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

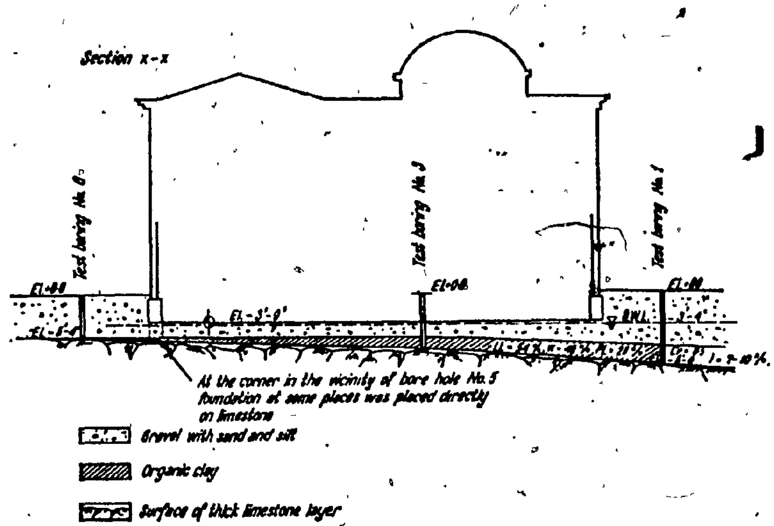

Fig. 15

Cross section of public baths building in Budapest Rumania.

for by the high level of the ground water. The high bearing capacity of 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 onto 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 measurements of the settlement (Fig. 16) 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. 16) show the difference of the settlement 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 was about 16.5 tons per square foot and the estimated total compression may be

$$S = \frac{pd}{M} \quad \frac{1.65 \text{ x } 3 \text{ ft}}{1.65 \text{ tons per sq. ft.}} = 3\ 5/8 \text{ in.}$$

Considering that the yearly rate is not more than 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 $\frac{1/2 \times 3\ 5/8}{3/8}$ or about five years.

2.2.6 Failure of Piles Unequally Loaded

A factory beside a river in Holland 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. 17). 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

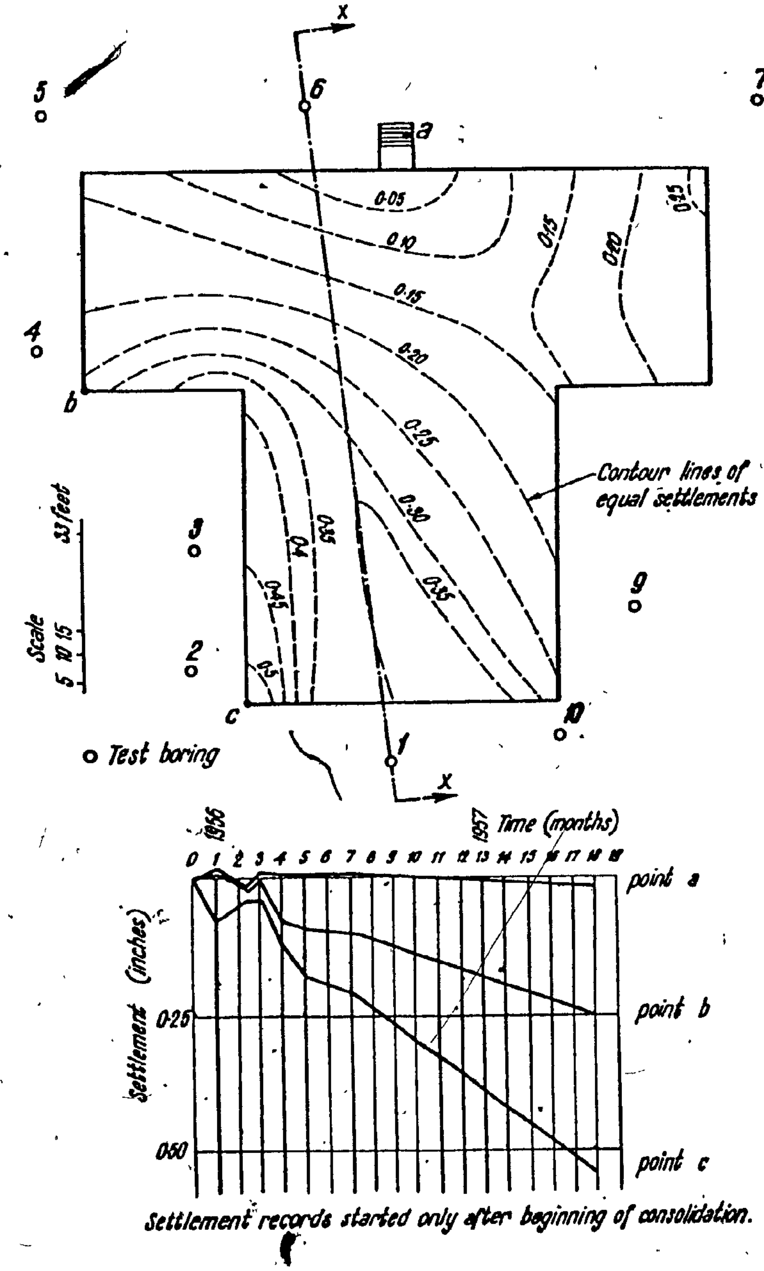

Fig. 16

Layout of Public Baths Building of fig. 15, and results of a differential settlement due to compression of the layer of clay.

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 overlying 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 peat 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 underlying 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. 17b, these lightly-loaded piles sand by the same amount as those under the main building.

The records of the settlements (Fig. 17c) indicate that there was considerable differential settlement between the long 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

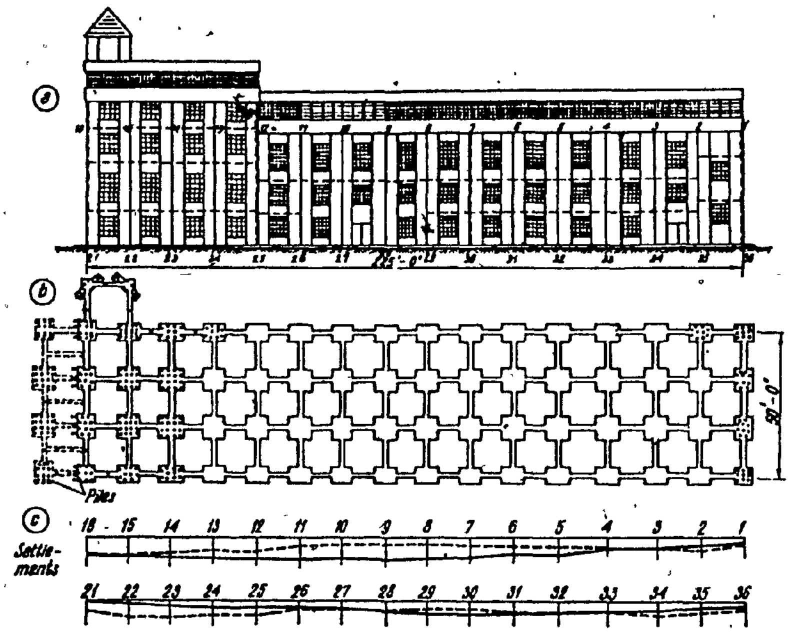

Fig. 17

a) Factory in Holland supported on piles

b) Layout of foundation and extention of the main building

c) Records of differential settlements which occurred between the two longitudinal sides of the building.

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. 17) 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 means under which the jacks were placed. The operation commenced at the tower and was successfully executed in stages throughout the entire building.

2.2.7 Effects of Vibration

In order to exemplify the effects of vibration on foundations, we will examine a case in which a diesel engine was placed on a block of reinforced concrete in the basement of an existing building (Fig. 18). The reinforced concrete mounting block was kept entirely separate from the foundations of the walls and columns of the structure to minimize vibration

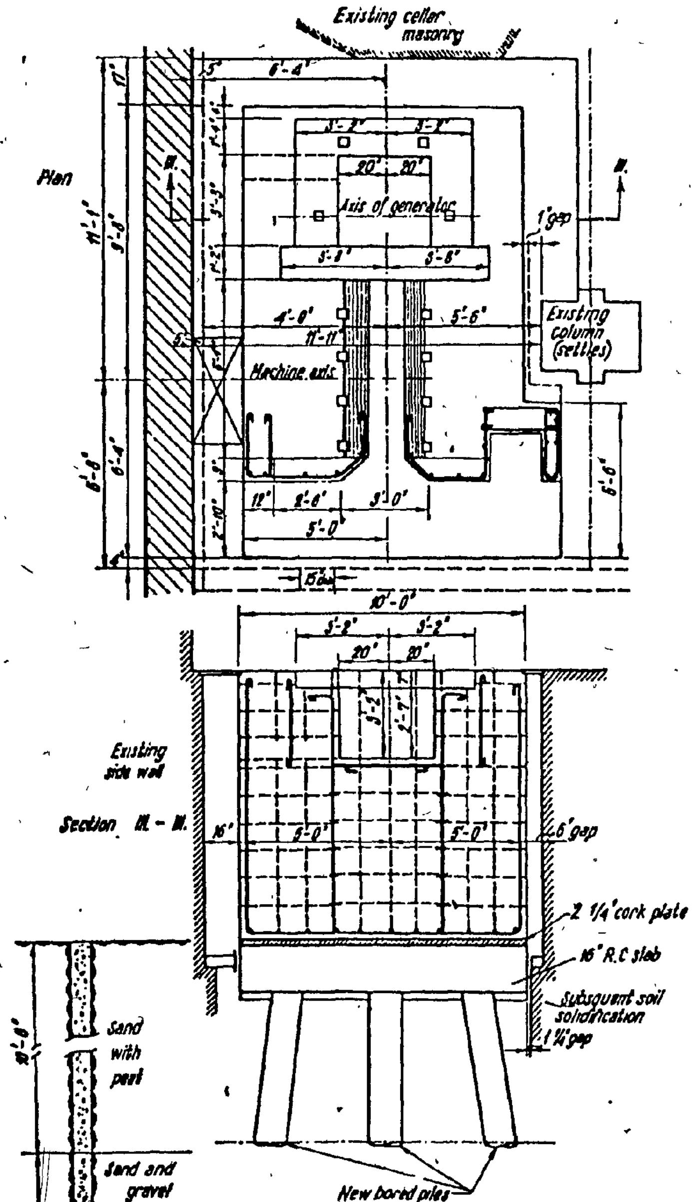

Fig. 18

Foundation layout of diesel engine, and cross section of underpin column with bored piles.

damage. 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 coke 2 1/4 in. thick surrounded by a frame of steel.

After a time, cracks appeared in a column of the building and 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 concided and compaction 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.

2.2.8 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. 19). 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 in the soil due to the closeness of footings is a common cause of differential settlement of interior and exterior walls.

The building described as follows provide an example of this type of differential settlement. Shortly after a group of identical buildings were constructed, cracks such as that in Fig. 19, appeared in all the structures and were found to be due to the greater settlement of the more heavily-loaded central wall. The soil under the foundations is stiff 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 3/4 in., whereas the settlement of the central footing was between 1½ in., as was measured from the deflection of the eaves.

2.2.9 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 also be due to the unavoidable consolidation of the filling in an approach ramp and the compaction of the soil caused by traffic. Horizontal displacements of tail abutments in the direction of a bridge may also depend on the depth of the foundation and the nature of the soil.

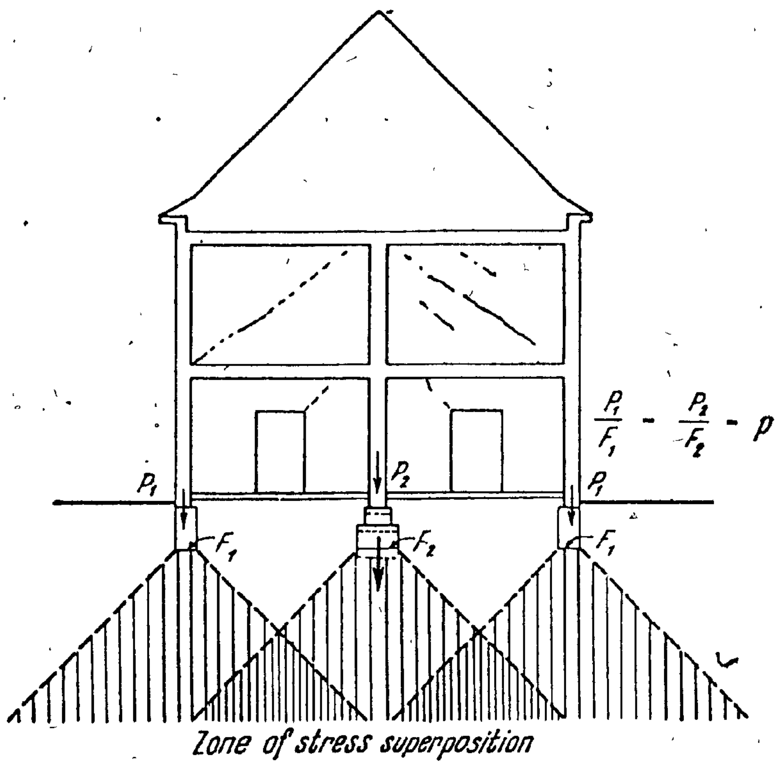

Fig. 19

In this cross section the overlapping zones of stress are demonstrated graphically. Obviously the middle column transmits more load and therefore that area is more susceptible to failure. The cracks that appeared on the building as shown are due to the overlapping stress.

An example of the tilting of an abutment of a bridge is shown in Fig. 20. THe abutments are supported on reinforced concrete bored piles extending 27 ft below the foundation. The fill 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 10 in, to 12 in. The load from a filling was about 2 tons per square foot, and this compressed the layers of softer soil which extended to a depth of 20 ft to 23 ft below ground level. The filling also settled a further 10 in. due to its own weight and to consolidation caused 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. 20 that the additional load due to the filling is so much greater under the back of the abutment than 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-distritution due to the filling, the stress-distribution diagram due to the original pressures σ_1 and σ_2, and the stess-distribution diagram when to the original pressures are added the 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 deep foundation the trapezoidal stress-distribution will be transmitted to the points of the piles. Here, however, the additional stresses due to the filling (P_2' and P_2'') were spread uniformly over the ground below the piles and did not change the shape of the stress-distribution diagram. In the

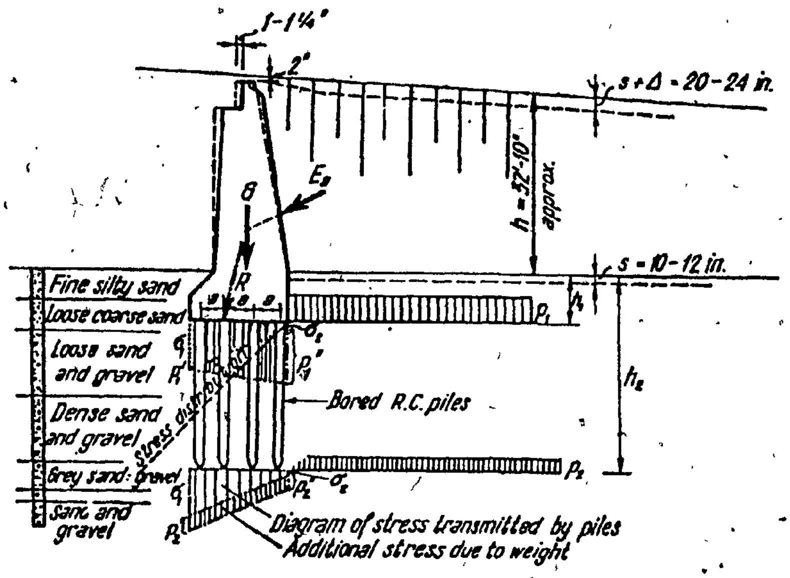

Fig. 20

Bridge abutment supported a reinforced concrete bored piles approximately 27 ft below the foundation is shown above. Also shown are stress distribution diagram due to the original pressures and the stress-distribution diagram when the original pressures are added due to the filling, as well as settlement and tilt. This abutment was built at Medue, Danube River.

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 ½ in. to 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 kept in mind that the passive resistance of earth will always serve to reduce the magnitude of forward tilting in such cases.

Another abutment (Fig. 21), is an example of the backward tilting of a tall reinforced concrete abutment on a shallow foundation. This abutment was built in two parts, one on the river face which supports the bridge and transmits the vertical forces directly to piles driven down to a stiff marl, the second, 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 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, 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

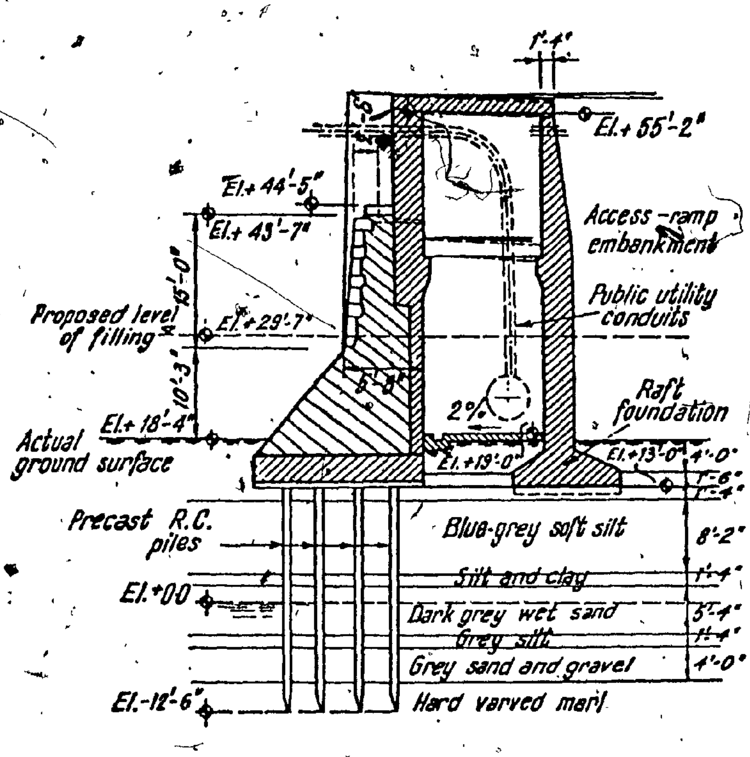

Fig. 21

Cross section of abutment at Arpord bridge in Budapest. Backward tilting and differential settlement occurred. Shown above are the raft foundation built to prevent failure.

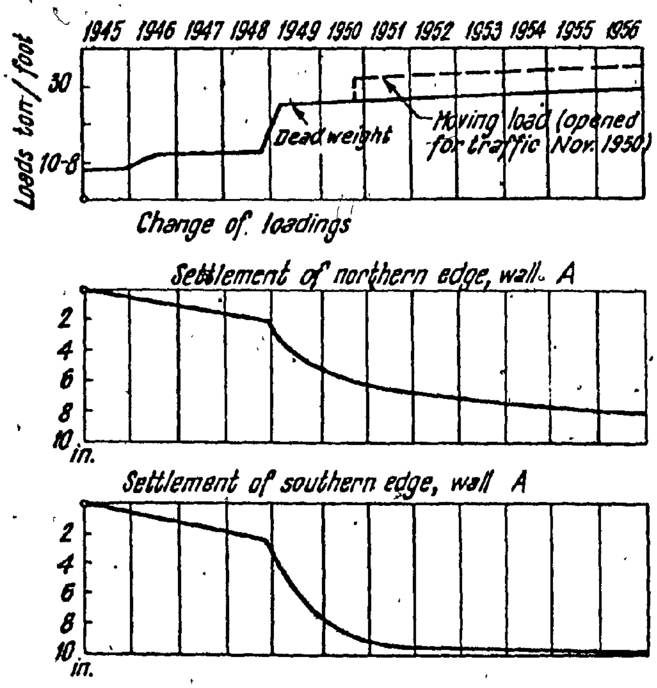

Fig. 22

These graphs describe the movements and settlements of the abutment in Fig. 21, from the beginning of the settlements until they have stopped eleven years after.

was not susceptible to movement.

The curves A in Fig. 22 show the rate the 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 ½ in. between the edges of the foundation of the wall. This tilting was made harmless by the joint, and the flexible nature of the abutment prevented the occurance of cracks.

2.3 CONSTRUCTION STAGE

2.3.1 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 and, as a result the ground water is subjected to considerable pressure, The engine house of Fig. 23 had a concrete raft foundation. Because the bottom of the sump was on a fairly impervious layer, no dewatering was necessary at the time of construction, 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, and through which the water seeped. To prevent this seepage, short timber sheet piles were driven along the end wall.

2.3.2 Ineffective Bracing of Lining of Excavation

The case which follows is an example of the collapse of the lining of an excavation due to the ineffective bracing. The tunnels in Fig. 24

were constructed by a cut-and-cover method which had been standardized after many years of experience. 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. At one part of the work, some changes were made, which caused the absence of both three-dimensional bracing between the vertical struts and the absence of any connection between the upper structs and bracing with the bottom of the excavation. The joists in the inner wall of the eastern part tilted some 20 ft at the top along a length of 200 ft and the wall collapsed. This accident emphasises the need for horizontal stiffening of bracings and structs across large distances.

2.3.3 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.

2.4 AFTER CONSTRUCTION STAGE

Failures also are 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 organic effects.

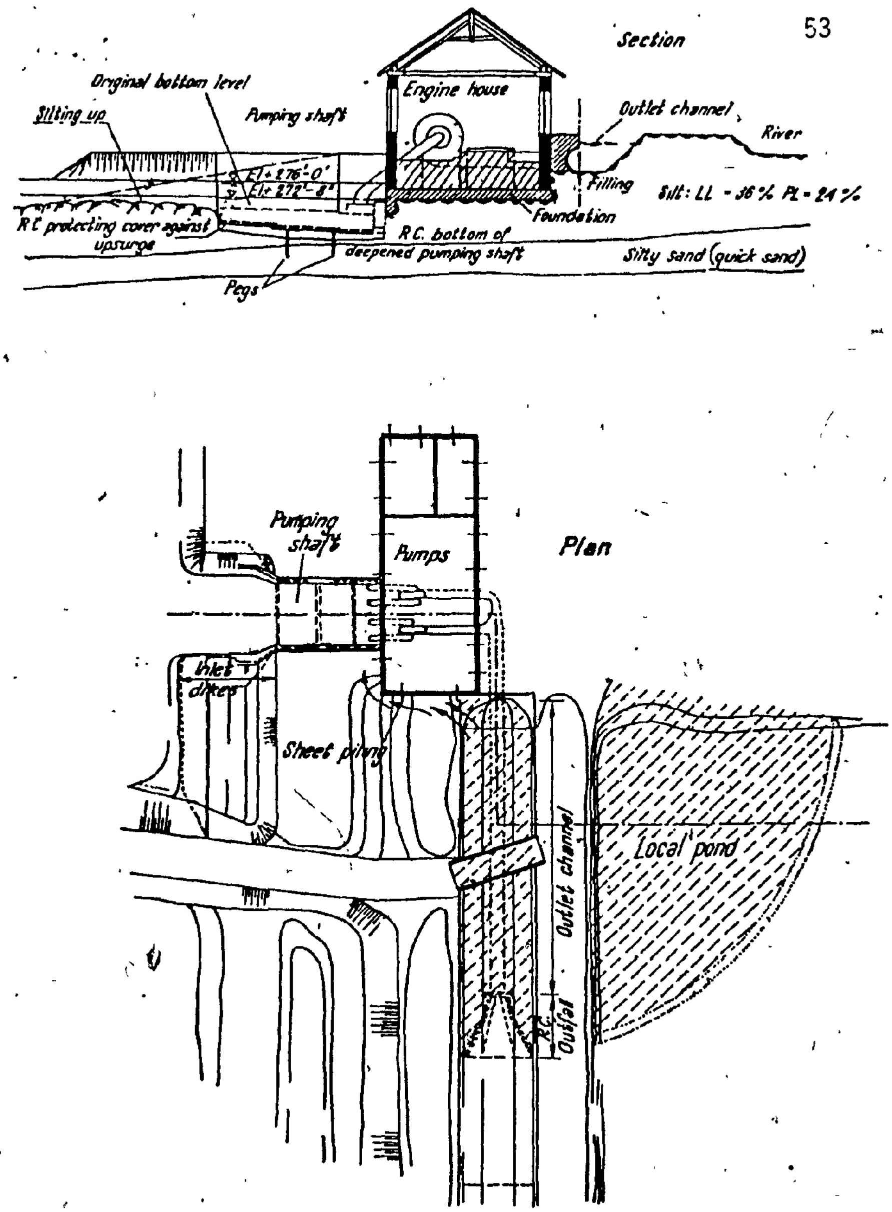

Fig. 23

Shown above are the plan and cross section of the pump house, protected with sheet piling from seepage. This pumping station is located at Karapanesa, Rumania.

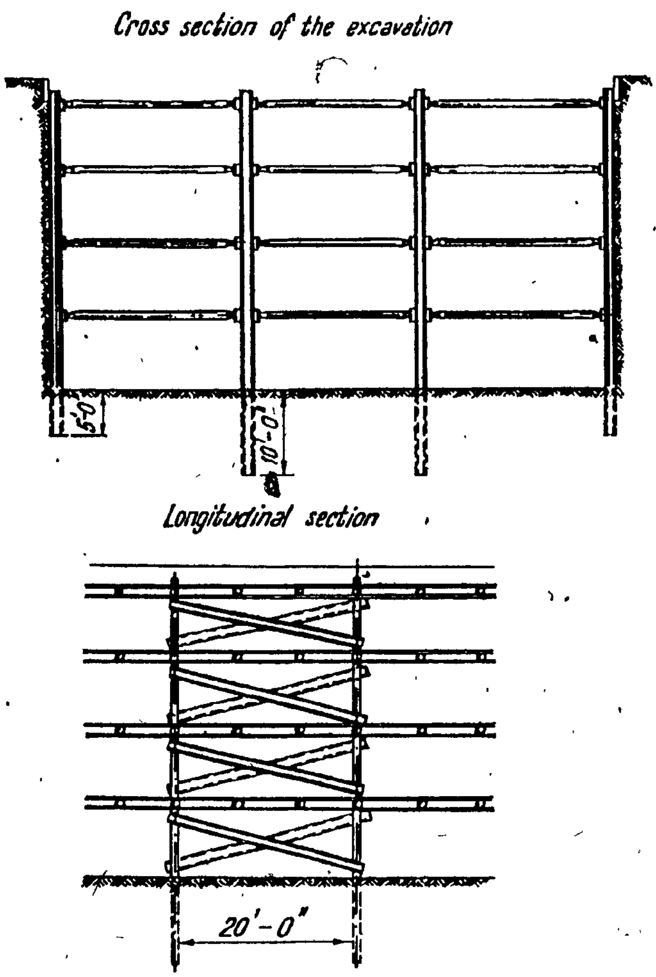

Fig. 24

Bracing for the support of the side of an excavation for part of the underground railway in Berlin, Germany.

2.4.1 Defects Due to Ground Water

Water, as a constituent of the soil, has an important influence on soil 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.

2.4.2 Scouring Due to Seepage of Ground Water

Fig. 25 illustrates an example of damage occurring in the foundation due to seepage. It was found that scouring of the soil due to the ground water 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 water course, and which followed the changes in the level of the river. The soil under the foundation is shown in Fig. 25. Although the velocity of the ground water could have been great enough to wash away the silty sand and loose sand, the differences in the flow of the ground water with changes of the level of the river were the cause of the differential settlement.

2.4.3 Fluctuation of Ground Water

Not only gross movements, but even minor oscillation of ground water levels may cause considerable compression of a cohesionless soil supporting a load. The magnitude of the compression soil supporting a

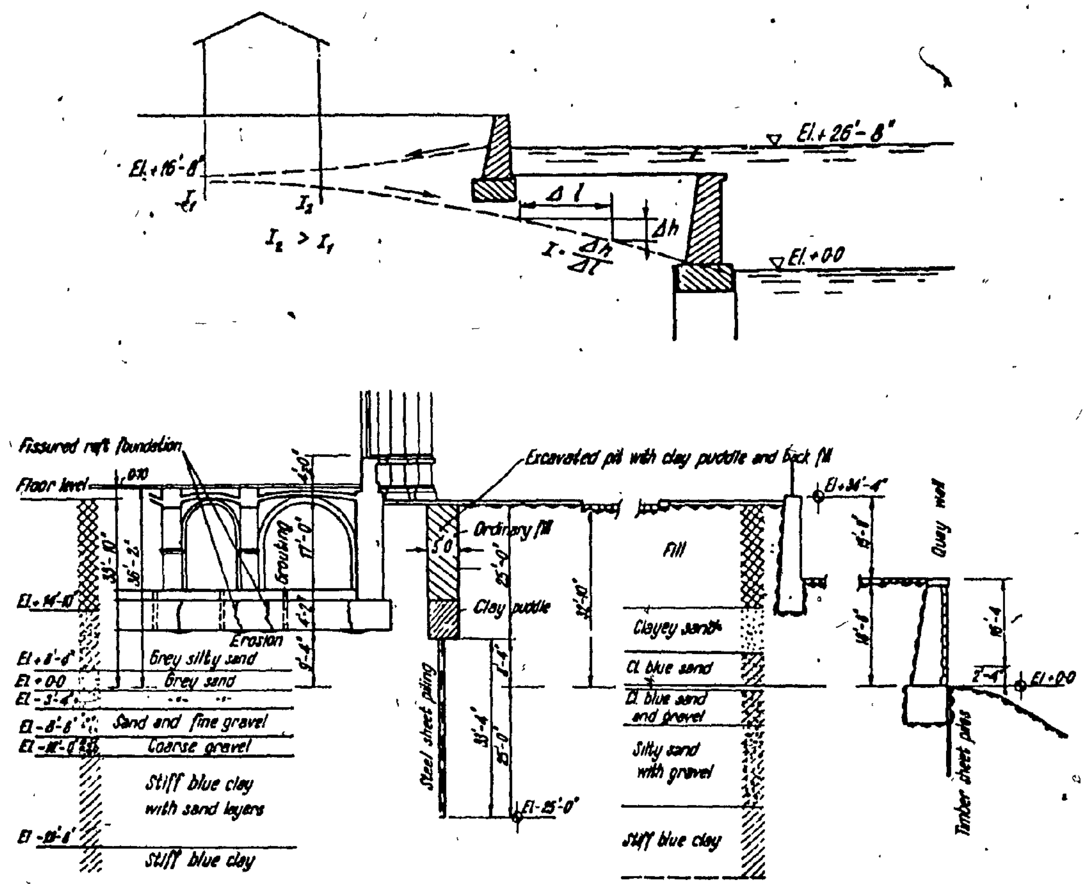

Fig. 26

Customs House at Budapest, Hungary; in which the cracks and settlements increased so that after 20 years it could not be used. Scouring of the soil due to seepage of the ground-water was the cause.

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 resulting will be related to the amount of consolidation.

2.4.4 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.

2.4.5 Scouring Due to Floods

Scouring depends on the effect of flow of large volumes of water and the erosion thus induced.

2.4.6 Combined Flood and Seepage

Combined flood and seepage damage is caused by a drastic change in ground water level and the resulting seepage which occurs.

2.4.7 Changes of Water Content of Soil

An increase of the water content of a soil will reduce the angle of internal friction, and hence reduce the shearing strength and the cohesion of the soil. Increasing water content results in a reduction of frictional resistance and lead to sliding failure of the soil.

2.4.8 Frost Action

It is generally the case that soils of more or less uniform grain size and containing more than 3% of particles smaller than 0.02 mm, and soils of mixed grain sizes containing more than 10% 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 refridgeration or otherwise exposed to the effects of freezing.

2.4.9 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 foundation, particularly when the soil is shrinkable clay. The reduction of water content can result in a reduction of the void content of the clay and cause 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.

3. CONCLUSION

The ability of the foundation to support its superimposed load without causing failure or rupture of the supporting soil is the carrying capacity. Failure of the soil is always accompanied by excessive settlement and after causes collapse of the structure. Failures of soils under foundations have been classified as local shear failures and general shear failures.

Although foundations are ofter designed on the basis of carrying capacity, probably, in most cases, the criterion for design should be settlement. Often excess settlements, especially differential settlements, occur under conditions providing a fairly high factor of safety against failure just as in the case of deflection of beams and other members of a structure. Failures may be produced fairly rapidly by overcoming the ultimate shear strength of the soil along a surface of failure, or very slowly by creep of very soft clays from under a loaded area without a defined failure surface.

Considering all the possible factors which can result in foundation failures, and consequently structural failures, every attempt should be made to avoid or minimize any unsatisfactory consequences.

During excavation and construction, the Engineer should be at hand to check and confirm the assumptions and information on which he based his design. If unexpected subgrade conditions are found, he should be ready to modify the design, to ensure the expected performance of the foundation.

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