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REPORT ON SITE INVESTIGATION

PROPOSED OIL REFINERY SITE

NEAR O'SULLIVAN'S BEACH

HD. NOARLUNGA

SOUTH AUSTRALIA

PART C

CHARACTERISTICS OF THE SOILS AND THE DESIGN OF FOOTINGS FOR INSTALLATIONS

by

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CONTENTS

TABLES

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PART C

CHARACTERISTICS OF THE SOILS AND THE DESIGN OF FOOTINGS FOR INSTALLATIONS

INTRODUCTION

The topography, geology, bore data and soil mechanics data were given in Parts A and B of this report.

As far as the geology of the site is concerned this was comprehensively and adequately described in Part A of the report and in summary form in the preamble to Part B. It is not considered necessary to repeat it here.

Only the interpretative aspects of the geology as they pertain to foundation engineering are presented in this part of the report, interpolated in appropriate places in the general text.

Likewise the soil mechanics data were fully covered in Part B of the report and only those derived parameters which are necessary to the interpretation of foundation conditions and to foundation design are introduced here.

In short Part C represents the joint conclusions and opinions of the authors regarding the foundation conditions and suitable design practices at the site, based on the data presented in Parts A and B.

SEASONAL MOVEMENT OF THE SOIL PROFILE

Speaking generally, the soils of the refinery site area are well drained, externally and internally. No free water was encountered in any of the bores during boring operations. Some water remained in the bores in which percussion drilling was carried out and some bores accumulated a little water after heavy rain, mainly due to the interception of run-off. The sand layers within the clays, the extensive cliff exposures, the re-entrant gullies and the deep stream valley to the south-east all contribute to this well drained condition.

The general soil profile consists of 12 inches of fine loam overlying to a depth of 6 or 7 feet a marl earth and a sandy marl. It has been shown(1) that these materials are relatively free draining and would be expected to wet up fairly quickly under winter rainfall conditions and dry out due to sub-surface drainage and other causes in the summer time.

Immediately underlying the marl earth and sandy marl is a sandy clay whose sand content increases with depth. Below a depth of some 20 feet there is often an increase in the clay content (See Table IX, page 20, reference 1) and then the sand content again increases.

It has been postulated(1) that the depth of seasonal moisture changes is likely to be of the order of 15 feet with a corresponding potential surface movement, under extreme drying and wetting conditions, of approximately $1\frac{1}{2}$ inches. The well drained conditions encountered in the test bores, particularly down to the first sand layer, indicate that in normal seasons a substantial proportion of this potential movement is likely to be realised. However because of a layer of some 7 feet of inert material overlying the expansive sandy clay the damaging effect to such structures as office buildings, supported with the normal strip footing on the surface of the ground, is expected to be only very minor and comparable to other defects produced in building, such as the shrinkage of plaster or the creep in brick-work and nailed trusses.

The effect on larger structures with a framework of steel columns supported by surface pad footings, is also expected to be of a minor nature.

It is not considered that paving around the outside of the structures would reduce the magnitude of the surface seasonal movement to a significant degree, but it will be shown later that such paving, if properly carried out, will have the effect of increasing the shear strength of the soil underlying the footings.

With respect to footings such as piers or rectangular pads at a contemplated depth of 7 feet, whose bases would rest on the surface of the sandy clay, the predicted seasonal shrinking and swelling should cause greater differential movements between adjacent footings. To reduce this effect it would be advisable, if deeper footings are being considered, to locate them not at 7 feet but at 10 feet below natural ground surface. At this depth (see Figure 2, reference 1) the potential seasonal movement should be reduced to approximately 0.6 inches and the effects of loads will reduce this figure still further.

It must be emphasized that the foregoing recommendations apply only to the average soil profile as described in the second and third paragraphs of this section. Different soil profiles will undoubtedly occur and some of these are described by Gibson[2]. To these profiles information contained in Part B[1] should be applied. Also before any final design is attempted it is strongly recommended that more detailed site investigations and testing be carried out.

STRESS HISTORY OF THE SOILS

From a series of Liquid and Plastic Limit tests (see Table VIII, page 19, reference 1) it can be seen that the field water content for the sandy marls lies below the plastic limit whilst the field water contents for the sandy clays are very close to their plastic limits. This would indicate according to Peck et al.[3] that the sandy clays at some stage had been subjected to a consolidating force much greater than is given by the present overburden load. As the depositional history of the area is one of accumulation it is thought that this over-consolidating force was due to desiccation.

Confirmation that the soils have been pre-consolidated is obtained by calculating the values of $\frac{Kv}{c}$ for samples at various depths. The results are shown in Table 1 and have been obtained using data from reference 1.

TABLE 1

Values of $\frac{Kv}{c}$ for Samples from Various Depths

Bore No.	Depth of Sample	$\frac{Kv}{c} = \frac{1}{c.m_v}$
1	5'0"	43
1	9'0"	72
1	15'0"	78
1	19'0"	780
7	4'6"	112
7	9'9"	108
7	14'0"	160
7	19'0"	226
7	29'0"	260

According to Skempton[4] for normally consolidated clays $25 < \frac{Kv}{c} < 75$ and for over-consolidated clays $75 < \frac{Kv}{c} < 200$.

DESIGN OF FOOTINGS

1. Introduction

At the Oil Refinery Site it has been found[2] that down to the first slightly clayey sand layer, unless hard sandstone or bedrock intervenes, the sequence of strata intersected is essentially the same for all bores. However the depth to this horizon varies widely from place to place, as do the thicknesses of the individual strata.

It will be this sequence of strata which is expected to principally carry the applied stresses from footing loads and for the purpose of attempting a logical design of various sized footings at different depths an 'average profile' as defined by Gibson(2) will be considered. This average profile is -

"Surface to 1'0"	Dark brown, fine, somewhat clayey loam.
1'0" - 1'9"	Kunkar limestone rubble horizon. Very variable in character laterally. Might be cemented into sheet limestone in places, or be absent in others.
1'9" - 7'4"	Finely sandy marl-earth. Pale to light colours predominate. Usually contains numerous small limestone nodules in the upper part, diminishing in number and size with depth.
7'4" - 19'9"	Light to pale greenish-grey, finely sandy clay with red-brown, yellow-brown, yellow or deep red mottling occurring in irregular abundance. Often develops very sandy layers. Moist and firm to very firm.
19'9" - 25'5"	Pale yellowish-grey, slightly clayey fine sand, with coarse yellow-brown and deep red ferruginous patches. Damp. Very compact, but friable."

The term marl-earth was coined to describe a very calcareous material of widespread occurrence in the southern part of South Australia and western Victoria and for which, as far as can be determined, there is no equivalent in existing literature. The material so described is composed mainly of very finely divided calcium carbonate containing a small proportion of magnesium carbonate and having the crystal structure of dolomite. The remainder of the material is finely divided silica and unidentified amorphous material. Typically it is intimately mixed with varying proportions of fine sand. There is much evidence to suggest that this material originated on the continental shelf, from whence it was carried inland by strong winds during the Late Pleistocene low sea levels. In its characteristic mode of occurrence it is covered by about a foot of dark loam, at the base of which is usually a discontinuous layer of concretionary limestone known as kunkar (locally called "travertine"). The kunkar ranges in character from small nodules with interstitial sandy marl-earth, through coarse and fine nodules closely packed, thin hard crusts with nodules, close-fitting coarse slabby lumps to hard sheet limestone. It is also very variable in thickness. The marl-earth occurs beneath the kunkar horizon and commonly contains numerous small limestone (kunkar) nodules immediately beneath the kunkar layer, but diminishing in number and size with depth.

Experience with this material in Adelaide and elsewhere shows that within the normal range of moisture conditions the marl earth has a moderate bearing capacity and tends to drain and dry out quickly. However, it loses strength with increase in moisture content and should it become saturated due to an excessive application of water, impeded drainage or confined exposure (as in a pit or trench) it can collapse under load and flow from beneath the point of application of the load. In Adelaide this condition is successfully accommodated in housing and similar structures by the use of wide strip footings seated on the soil surface.

It should be borne in mind that in some bores, such as Bore 11 or Bore 16[2], hard sandstone or bedrock is encountered before the full upper sequence is developed. The depth of this hard material will naturally modify the behaviour of footings designed for the 'average profile'.

It is also not intended to cover every possible footing solution for different structures but rather to treat only normal footing practice which should be satisfactory within the Refinery Site. Emphasis must be placed on the relative small number of tests, (for a project of this size) which have been carried out and as mentioned in 'Seasonal Movement of the Soil Profile' it is strongly recommended that further exploratory work and testing be undertaken before the detailed design of individual structures is attempted.

With respect to the material below a depth of 25 feet there might occur alternating mottled sandy clays and clayey sands as in Bore 1, or fat clays, gradually becoming sandy with depth, as in Bore 23. If footings are considered through these layers they would probably be some type of piled foundation which must rely on skin friction and bearing for their stability. To satisfactorily design such a foundation much more information is needed than is at the authors' disposal, so the behaviour of deep piled foundations will not be considered in this report, except, in so far as they present a satisfactory solution, if properly designed, for the more heavily loaded footings in the deeper soil profiles.

2. Surface Footings

(a) Unprotected and Sheet Limestone not Considered

In the design of surface footings it will be considered first that there exists from 1'0" to 1'9" a travertine rubble horizon and that no protection is provided against the penetration of surface water.

For the design of any footing there are two criteria that must be satisfied. First the footing must have an adequate factor of safety against shear failure and secondly the settlement of the footing must be such that the superstructure can resist differential settlements between adjacent footings. To the different types of footings dealt with in this report these two criteria will be applied.

If no protection is afforded then the underlying material would be expected to behave towards applied stresses as purely cohesive non-frictional material and it is considered sufficiently accurate to calculate the bearing capacity on the assumption that $\phi u = 0$. The equation for the ultimate bearing capacity at foundation level then becomes

$$q \text{ (ultimate)} = c\,Nc + p \quad \ldots\ldots \quad (1)$$

where Nc is a bearing capacity factor depending, for surface footings, on the dimensions of the footing

Nc for surface strips = 5.14

Nc for circular or square = 6.20

p = overburden pressure at foundation level, which for a surface footing = 0.

For a 15 inch strip footing

$$q \text{ (ultimate)} = 5.14\, c \text{ ...... } (2)$$

Now the value of "c" must be selected from [1] and should be an average for a depth of $^2/_3$ B beneath the base of the footing[4]. Provided that the maximum or minimum value of "c" does not vary more than 50 per cent from this average then equation (2) is considered valid.

From Figure 3[1] $c = 3$ p.s.i. and

$$q \text{ (ultimate)} = \frac{5.14 \times 3 \times 144}{2240} \text{ t.s.f.}$$

$$= 1 \text{ t.s.f. approx.}$$

Adopting then a factor of safety of 3, which is customary for a plastic failure.

$$q_a \text{ (allowable)} = .33 \text{ t.s.f.}$$

The allowable bearing capacity for a 15 inch strip on the surface of the ground = .33 t.s.f.

The next calculation that should be carried out for the 15 inch strip is that of settlement. If the material is assumed saturated then the total settlement[5] may be divided into two components, elastic settlement and consolidation settlement, and is commonly expressed by the equation

$$\rho \text{ total} = \rho i + \mu \rho \text{ oed ...... } (3)$$

ρi = initial or elastic settlement

$\mu \rho$ oed = the consolidation part of the total settlement.

$$\rho \text{oed} = \int_0^z m_v \cdot \Delta\sigma \; dz$$

and $$\mu = A + \alpha (1 - A)$$

where A is a pore pressure parameter which depends on the state of consolidation of the clay and α varies as the nature and dimensions of the footings.

The elastic settlement under loads applied to the surface of the ground may be calculated from the formulae

$$\rho_1 = B.\, q \frac{1 - \nu^2}{Es} .\ Ip \qquad (4)$$

where B = breadth of the footing

q = net loading intensity

Ip = influence factor depending on the shape and rigidity of the footing.

ν = Poisson's ratio

and Es = secant modulus calculated at half failure stress.

Applying the above formulae to calculate total settlement

$$\rho\text{(total)} = .22 \text{ inches}$$

Having arrived then at total settlement figures some estimate must be made of the differential settlements likely to occur along the footing or between two separate strips. Very little experimental data is available on this phase of settlement studies but some information on local conditions is now coming forward from work proceeding in Adelaide. However for the Oil Refinery Site, unless the conditions are markedly different across a site, it would be as well to proceed along the lines of making the differential settlement half the total settlement.

Proceeding next to the case of a circular or square footing on the surface of the ground

$$q \text{ (ultimate)} = 6.2\ c \ldots\ldots\ldots (5)$$

$$= \frac{6.2 \times 3 \times 144}{2240} \text{ t.s.f.}$$

$$= 1.2 \text{ t.s.f.}$$

$$\text{or } q_a \text{ (allowable)} = .4 \text{ t.s.f.}$$

Now as the value of "c" in equation (5) is an average "c" some limit must obviously be placed on the maximum dimension of the footing and for the purpose of this report the limit adopted will be a maximum dimension of 12 feet.

The total settlement of square or circular footings on the surface of the ground can be calculated using settlement formulae given earlier in this sub-section. For any one type of footing, if the load intensity is constant, the settlement increases in direct proportion to the dimension B (diameter or side of square) of the footing.

In Table 2 are shown the safe loading intensities and the settlements at these safe loading intensities for various types of footings sitting on the ground surface.

TABLE 2

Footings considered unprotected and sheet limestone not considered

Type of Footing	Safe Loading Intensity t.s.f.	Total Settlement at Safe Loading Intensity "ins"
15 inch strip	.33	.22
22 inch strip	.33	.32
24 inch (circle or square)	.40	.42
48 inch (circle or square)	.40	.84
96 inch (" " ")	.40	1.14
144 inch(" " ")	.40	1.20

(b) Protected and Sheet Limestone not Considered

If the surface footing is protected by some means, such as building a concrete apron around the outside the footing to prevent the direct entry of surface water, then considerably higher average values of cohesion can be used in calculating safe loading intensities. A suitable width for a concrete or other impermeable apron is considered to be equal to the minimum dimension of the footing but in no case less than 3 feet.

A study of the results in (1) indicates that a cohesion of 10 p.s.i. would be a suitable figure for the calculation of ultimate and allowable bearing capacities down to a depth limit of 8 feet.

$$q_a \text{ (allowable strip)} = \frac{5.14 \times 10 \times 144}{3 \times 2240} \text{ t.s.f.}$$

$$= 1.1 \text{ t.s.f.}$$

$$q_a \text{ (allowable square or circle)} = \frac{6.2 \times 10 \times 144}{3 \times 2240} \text{ t.s.f.}$$

$$= 1.3 \text{ t.s.f.}$$

However these safe loading intensities can only be used if water is not allowed access to the base of the footings. This is often a difficult thing to do, despite very careful surface protection, as there is a danger of a burst water pipe beneath the footing or cracks in the concrete apron causing saturation. Another source of danger would be the possibility of a perched water table developing under the footing due to the configuration of the marl earth and the sandy clay interface.

The settlement calculation now becomes uncertain as the materials being dealt with are unsaturated in terms of pore space. However in Table 3 calculations are shown of total settlements, based on saturation, using a general formula by Skempton[5].

Because of pore space unsaturation in the field the figures of total settlements in Table 3 must be regarded as the maximum likely, but they could overestimate settlements considerably if unsaturated conditions could be maintained. On the other hand, with saturation, there is a danger of a collapse mechanism occurring which involves inter-particle forces and the structural arrangement of the soil grains. More information will be available on this mechanism from tests now being carried out. Such collapse could cause immediate settlements which would have a greater damaging effect on the building that settlements from long term consolidation effects.

TABLE 3

Footings considered protected and sheet limestone not considered

Type of Footing	Safe Loading Intensity t.s.f.	Total Settlement at Safe Loading Intensity. "ins"
15 inch strip	1.1	.40
22 inch strip	1.1	.55
24 inch (circle or square)	1.3	.75
48 inch (circle or square)	1.3	.80
96 inch (circle or square)	1.3	1.10
144 inch (circle or square)	1.3	1.20

It should be re-emphasized that in Table 3 no allowance has been made for the frictional behaviour of the material. The ultimate bearing capacity has been defined by Terzaghi[6] in the following terms

$$q = c\,Nc + p_0\,Nq + \frac{\gamma B N_\gamma}{2} \quad \ldots\ldots\ldots \quad (6)$$

where q denotes the ultimate bearing capacity of a shallow foundation.

po denotes overburden pressure at base level

γ denotes density of the soil

B denotes width of foundation

Nc, Nq and N_γ are factors dependent on Ø and upon the dimensions of the foundations.

However the values of Ø in Soil Mechanics Data[1] vary widely and in addition there are a great number of low values of Ø. This justifies treating the materials as behaving as purely cohesive materials with respect to applied stresses but it should be recognised that the values of safe loading intensities in Table 3 are probably on the conservative side if unsaturated conditions are maintained.

(c) Sheet Limestone Considered

If on a building site it is proved that the limestone from a depth of approximately 1'0" to 1'9" is continuous over the area then it is considered that an increase in safe loading intensity could be obtained if the surface loam layer was removed and the footing was made to rest directly on the sheet limestone.

Any calculations attempted of safe loading intensity or settlements would be unreal and the best method of estimating these quantities would be by field loading tests. However when performing the loading tests sufficient time should be allowed for equilibrium to be reached under each load increment.

3. Shallow Footings

Under this heading will be treated footings down to a depth of 3 feet below natural ground level.

(a) Surface Unprotected

There is thought to be some virtue in placing the footings below natural ground surface, because of the increase in the bearing capacity factor Nc, the effect of overburden pressure and the general decrease shown in the values of "m_v" with depth. It is clearly impossible to treat in this report all the depths below ground surface so an average depth of 3 feet will be chosen for the calculations.

If what is known as a deep beam type of foundation is used, that is, in this case a beam of depth 3 feet below the ground surface, rectangular in section and 15 inches wide, then

$$q_a \text{ (allowable bearing capacity) } = \frac{cNc}{F} + p_o$$

Nc for a depth/breadth ratio of 2.4 = 7.2

and $$q_a = (.46 + .16) \text{ t.s.f.}$$
$$= .62 \text{ t.s.f.}$$

so that the safe bearing capacity at foundation level = .62 t.s.f.

In the case of a pad footing 2 feet square at a depth of 3 feet.

$$q_a = .7 \text{ t.s.f.}$$

Some results on safe loading intensities and estimated settlements are shown in Table 4.

TABLE 4

Footings at a depth of 3 feet, surface unprotected

Type of Footing	Safe loading Intensity t.s.f.	Total Settlement at Safe Loading Intensity "ins"
Deep beam 15 ins. wide	.62	.11
24 inch (circle or square)	.70	.20
48 inch (circle or square)	.64	.28
72 inch (circle or square)	.62	.30

(b) Surface Protected

If the surface of the ground is protected to prevent the direct entry of surface water then, as before, increases in safe loading intensities and total settlements will occur. These quantities have been calculated and are shown in Table 5.

TABLE 5

Footings at a depth of 3 feet, surface protected

Type of Footing	Safe loading Intensity t.s.f.	Total Settlement at Safe Loading Intensity "ins"
Deep beam (15 ins wide)	1.70	.46
24 inch (circle or square)	1.90	.80
48 inch (circle or square)	1.74	1.00
72 inch (circle or square)	1.70	1.10

The calculations in Table 5 have been carried out on the understanding that the marl earth material remains unsaturated. This is expected to be the condition to prevail if the surface is completely protected. For footings seated at a depth of 3 feet a greater area of surface protection is required than for the same type of footing on the surface.

Even with this complete surface protection accidental saturation or the development of a perched water table could still lower the values of cohesion and considerably decrease the time for settlement to occur.

4. Deep Footings

Under this heading footings in the sandy clay layer, from 7' - 4" to 19' - 9" will be considered. It has already been shown, (See Section on Seasonal Movement of the Soil Profile), that if deep footings are being considered then to reduce the effect of seasonal movement a suitable depth would be at least 10 feet below natural surface for the idealised soil profile or 2 feet 6 inches into the sandy clay layer for the in-situ soil profile.

(a) Bearing Capacity

Due to the extreme variability, also the presence of many low values of "ϕu" in Reference 1, the soils at depth will be treated as behaving as $\phi u = 0$ materials with respect to applied stresses.

Peck et al.[3] have considered equilibrium conditions underneath a smooth loaded footing with the simplified assumption that failure takes place on two planes at 45 degrees. They arrive at the expression for ultimate bearing capacity -

$$q = 4_c + p_o$$

To take into account the fact that the surface of failure is curved and that the base of a real footing is rough, they suggest using a semi-empirical formulae

$$q = 5.70c \quad (1 + 0.3 \frac{B}{L})$$

which applies to a rectangular footing with a rough base having a width B and a breadth L.

However as a result of model tests carried out[4] in which careful allowances were made for the effects of small decreases of water content in the clay beneath the footings, due to the diffusion of the high pore pressures set up by the load, and for the effects of different rates of strain in the loading tests and the compression tests, bearing capacity factors have been derived for footings of various shapes and at various depths below the surface. These factors are shown in Table 6.

TABLE 6

Bearing Capacity Factors for Foundations in Clay ($\phi u = 0$)

Depth/Width Ratio D/B	Nc	
	Circle or Square	Strip
0	6.2	5.14
0.25	6.7	5.6
0.5	7.1	5.9
0.75	7.4	6.2
1.0	7.7	6.4
1.5	8.1	6.8
2.0	8.4	7.0
3.0	8.8	7.4
4.0 or greater	9.0	7.5

It is recommended that these bearing capacity factors in Table 6 be used in the formula

$$q = cNo + po \quad \ldots\ldots (7)$$

for a preliminary calculation of ultimate bearing capacity. For rectangular footings seated below the surface

$$No = (0.84 + 0.16 \frac{B}{L}) \; Nc \quad \text{(square)}$$

The allowable bearing capacity

$$qa = \frac{cNc}{F} + po$$ where F is a factor of safety

depending on the mode of failure. As the failure was mainly plastic in the compression tests a factor of safety of 3 will be chosen. In Table 7 are shown safe bearing capacities or loading intensities for circular or square footings at a depth of 10 feet below the surface.

TABLE 7

Safe Bearing Capacities at 10 Feet Below Natural Ground Surface for Circular or Square Footings

Size of Footing (Circle or Square)	Safe Loading Intensity t.s.f.
2'	4
3'	4
4'	3.8
5'	3.7
6'	3.6
7'	3.6
8'	3.5
9'	3.5
10'	3.5

In Table 7 the safe loading intensities are the allowable pressures at the bottom of the footing. If for example a hole is excavated for a pier and then backfilled with concrete the allowable pressure at ground level would be the relevant figure in Table 7 minus the weight of the backfilled concrete. Also the safe loading intensities have been calculated on an average cohesion of 18 p.s.i.

Skempton[4] observes that the values of Nc in Table 6 are probably on the conservative side for clays whose $\frac{Kv}{c}$ values fall in the overconsolidated range.

(b) Settlement

The total settlement of any footing can be divided into two components, elastic and consolidation settlement.

As shown previously the elastic settlement of a footing on the surface

$$\rho_1 = B.q \frac{1 - \nu^2}{E_s} I_p \quad \ldots\ldots\ldots (8)$$

Poisson's ratio for a saturated soil is 0.5. Ip = 1.12 for a square flexible area, and Ip = $\frac{\pi}{4}$ for a rigid circular area on the surface. For footings at some distance below the surface the influence value Ip decreases (Fox)[7] and the value of load intensity to use in formula (8) is the net load intensity.

The consolidation part of the total settlement can be calculated from the formulae

$$\mu \int_0^z m_v . \Delta\sigma \; dz$$

An upper value for μ would be = 1 and the value of $\Delta\sigma$ to use immediately underneath the footing would be again the net load intensity on the underside of the footing. The net load intensity is the actual intensity of load minus the original overburden pressure.

In Table 8 total settlements have been calculated for various sized footings at depths of 10 feet below natural ground surface. The settlements are for net load intensities of 2 and 3 tons/sq. foot and use has been made of values of "m_v" from Soil Mechanics Data [1]. The footings have been considered rigid and the same influence factor Ip has been used for square and circular footings.

TABLE 8

Total Settlement at 10 Feet Below Natural Ground Surface

for Circular or Square Footings.

Type of Footing (Circle or Square)	Net Load Intensity t.s.f.	Total Settlement at Net Load Intensity "ins"
2'	2 3	.42 .63
4'	2 3	.84 1.26
6'	2 3	1.26 1.89
8'	2 3	1.70 2.55
10'	2 3	2.12 3.18

However with respect to the damage likely to be caused to the superstructure of a building it is not the total settlements but the differential settlements between adjacent footings that is the main concern.

The differential settlements can best be evaluated on the merits of each site. For example if the sub-soil conditions vary from bedrock to soft clay then the maximum possible differential settlement can be taken as the total settlement on the most compressible strata, that is if we assume equal sized footings loaded with the same intensity. If the sub-soil conditions are homogeneous the differential settlement is often taken as half of the total settlement.

It is also recognised that different types of buildings can withstand different amounts of differential settlement and a guide in this direction is shown in Table 9.

TABLE 9

Differential Settlements for Various Types of Superstructure

Rigid Frames	Differential Settlement $\leq \frac{1}{2}$ inch
Semi-Rigid Frames	Differential Settlement ≤ 1 inch
Flexible Frames	Differential Settlement ≤ 2 inches

PREFERRED AREA FOR HEAVY CONSTRUCTION

Insufficient detail has been obtained regarding bedrock irregularities in the primary construction area, particularly in Pt. Section 588. However, on present indications the area to be preferred from the geological point of view for heavy construction requiring a high degree of stability can be delimited as follows. With Bore No. 12 as centre, describe an arc with radius 690' to meet the eastern boundaries of the site. This arc, together with the boundary intercepts encloses the preferred area. Within this area firm to hard slates or sandstone should occur within 20 ft. of the surface, or less. Here, as elsewhere all foundations for heavy structures should be seated below the marl-earth for maximum safety, in this case at least 6 ft. below the surface and preferably 10 ft. below it. The topography in this area would allow most, if not all of the marl-earth to be stripped off before building. However, this would lower the base of the zone of seasonal soil moisture variation, which would then accentuate seasonal shrinking and swelling movements and still render deep foundations necessary.

CONCLUSIONS

In this report only the more common type of footings have been considered. There are probably other types which would be equally suitable for the area. For example for a surface footing the concrete raft would appear to have distinct advantages, whilst for the more heavily loaded footings deep piles must be considered.

With the deep footings, (those at a depth of 10 feet) since seasonal moisture changes have been postulated to a depth of 15 feet, no account has been taken of skin friction. However with piles at a greater depth than 15 feet skin friction must be taken into account and possibly also a chemical bonding action between the clay and the concrete.

If it is desired to use rectangular footings then the bearing capacities and settlements can be calculated using the minimum dimension 'B' for the calculation of bearing capacity factors and in the formulae for total settlement.

For the settlement formula to apply to any type of footing the spacing between should be not less than 2B centre to centre. After excavation for a footing the minimum possible delay is recommended before backfilling with concrete.

The authors have been given very little information about the types of structures to be supported and since it is not practicable to consider all possibilities in specific detail, what were considered to be the most probable cases were selected and treated in this report.

Even though an analysis of the data indicates that heavy structures, with low intensities of loading, can be safely supported on or below the natural soil surface, experience has shown that carelessness in the use and disposal of water during and after construction can have far reaching consequences.

It is clear that much more sampling and testing needs to be carried out before detailed foundation designs are attempted and this should be done as soon as the location and lay-out of the plant is decided upon. Details of loadings and stability requirements for the various units should then be submitted to the testing authority as soon as they are available.

The appointment of a structural engineer with foundation design experience for close liaison in the field and laboratory would greatly facilitate this work.

P O Morris .... Alan Gibson ....

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24/10/60

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