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Professor A. J. Sutton Pippard, M.B.E., D.Sc., M.I.C.E., Chairman of the
Division, in the Chair.

The following Paper was presented for discussion and, on the motion of
the Chairman, the thanks of the Division were accorded to the Authors.

Structural Paper No. 31

“Buoyant Foundations in Soft Clay for Oil-Refinery Structures at Grangemouth”

by

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SYNOPSIS

In the design of the foundations for heavy load concentrations, maximum use was made of earth buoyancy to reduce the net soil bearing-pressure.

Each foundation consists of a hollow reinforced-concrete box below ground level. Longitudinal and transverse walls divide the foundation into a series of 10-foot-square cells. The walls were built to the full height above ground and sunk to the required levels as monoliths. The cell bottoms were laid during and immediately after the final stages of sinking. Subsequently, structural decks were formed over the cells.

Reinforced-concrete surface-rafts were used for founding light load concentrations.

It was anticipated that the settlements of the surface rafts would be greater than those of the buoyant foundations and allowance was made for this in the initial foundation levels. Records of settlements are being kept, and details for the first four buoyant foundations and the first surface raft to be built are set out in the Paper.

INTRODUCTION

THE original Grangemouth refinery of Scottish Oils Ltd was sited on the bank of the River Forth immediately east of the Port of Grangemouth. When this site was chosen it was known that the soil consisted of a considerable but unexplored depth of soft clay with a somewhat firmer stony layer at about 30 feet below the surface. The plant was carried on reinforced-concrete rafts which were laid practically on the surface of the ground because the site was below the highest recorded tide level. The design was based on a maximum soil bearing-pressure of 0.5 ton per square foot although the average pressure did not exceed 0.42 ton per square foot. Since the rafts were built at ground level these pressures may be regarded as net.

About a year after construction, some slight settlements were observed at the more heavily-loaded points, and by 1948 up to 12 inches of settlement had occurred. The uneven loading of the boiler house raft had resulted in a tilt which brought the chimney 2 feet out of plumb in a height of 110 feet.

In 1930, 1937, and 1939 additional plant and boilers were installed on foundations consisting of bulb-ended cast-in-situ piles, the toes of which were driven into the stony layer existing at a depth of about 30 feet, the load per pile being about 15 tons. Some small settlement of this additional plant, with consequent slight distortion of the superstructure, has occurred.

The throughput capacity of the original refinery had remained unchanged from when it was opened in 1924; but in 1946, as part of the scheme for extension of home refining activities, the parent company, Anglo-Iranian Oil Co. Ltd, decided to increase the throughput capacity to five times the previous figure. This involved the construction of several heavy refining units together with associated treatment plants, tankage and services, the latter including a new power station of 20,000 kilovolt-amperes capacity. Each of the refining units consists of a group of main elements linked together by pipework. The characteristic main elements are furnaces for heating the oil, large reaction vessels, fractionating towers, banks of heat-exchangers, pump-houses, and chimney-stacks.

Flat land suitable in area and position for such an extension was available immediately east of the existing refinery and, although its soil bearing-capacity was low, the site was chosen as being the best available from the operating and commercial points of view.

FOUNDATIONS FOR THE NEW WORKS

The expansion programme demanded foundations capable of carrying a wide range of load concentrations, many of which were much heavier than had been necessary for the earlier works. The foundations were

divided into two categories, the heavier load concentrations being carried on buoyancy rafts and the lighter ones on surface rafts. The Paper deals mainly with the buoyancy rafts, the surface rafts being referred to only in connexion with the choice of types of foundation and with settlements.

It is established practice in foundation design to base calculations on the net soil bearing-pressure, which is obtained by deducting the weight of soil displaced by the foundation from the total load to be supported. In cases where the net bearing pressure cannot be reduced to the safe value for the site by spread surface-foundations, relief of pressure may be obtained by taking advantage of earth buoyancy. A buoyant foundation may be defined as one in which the net soil bearing-pressure is reduced to a practicable value by the use of hollow construction below ground level, so that the combined weight of superstructure and foundation equals the weight of the soil displaced by the foundation plus the soil bearing-capacity at the founding level.

At Grangemouth, buoyancy was obtained by use of reinforced-concrete cellular rafts. Monolith construction was adopted to obviate the problems which would have arisen in the large open excavations necessary for in-situ construction. Monoliths in various stages of sinking are shown in *Figs 1* (facing p. 308). Some settlement of both the surface and the buoyant foundations was anticipated, and allowance was made in the initial founding levels. On completion of the foundation works, arrangements were made for periodical recording of levels at a series of chosen points.

SOIL INVESTIGATION

The soils information available being meagre, a comprehensive survey was carried out. This showed a surface layer formed by reclamation lying on estuarine deposits over boulder clay. Borings, from which samples for analysis were taken, were sunk to depths of 100 to 145 feet. A separate series of wash borings was put down to determine the contours of the boulder clay.

The reclaimed surface layer, which is of clay and about 5 feet in thickness, is firmer than the estuarine deposits and has a shear strength of about 1,000 lb. per square foot.

The estuarine deposits consist mainly of soft silty clay with occasional sandy layers. The thickness varies from about 80 feet in the south-east to more than 240 feet in the north-west corner of the site. Approximate contours of depths below ground level to the top of the boulder clay are shown in *Fig. 2*. The positions of the power station and refinery units, and that of borehole No. 1, have been superimposed on the contours. The soil properties of samples taken from this borehole are shown in *Figs 3*.

The soil properties shown in *Figs 3* are considered to be fairly typical of the whole site. Generally, the liquid limit is only a little higher than

Fig. 2

CONTOURS SHOWING DEPTH OF TOP OF BOULDER CLAY BELOW GROUND LEVEL

Note : Ground level is approximately +11.50 O.D.

the moisture content, which has an average value of about 35 per cent. The estimated shear strength ranges from 100 to 400 lb. per square foot with an average value of 250 lb. per square foot. To obtain undisturbed samples of the soils presented considerable difficulties, so a

Figs 3

TYPICAL SOIL PROPERTIES
(Borehole No. 1)

series of tests was made on the site with the vane instrument, newly-developed at that time. At depths of more than about 45 feet these tests showed greater shear strength, the increase being more marked with increasing depth; at 100 feet the vane gave a reading of about 800 lb. per square foot. A more detailed account has been given by Dr A. W. Skempton.¹

Immediately below the reclaimed surface layer there is a permeable layer containing shells. The moisture content is much higher than at other levels.

A fairly firm stony layer varying from 2 to 10 feet in thickness is found at levels between 23 and 33 feet. The matrix between the stones is comparatively coarse and the moisture content low.

¹ "Vane Tests in the Alluvial Plain of the River Forth, near Grangemouth." *Géotechnique*, vol. 1, No. 2, p. 111 (Dec. 1948).

CHOICE OF TYPES OF FOUNDATION

Foundations for load concentrations ranging from a few hundred lb. per square foot to about a ton per square foot were required in the new works. Past experience on the site and information from the soils survey clearly indicated that no single type of foundation would be suitable for such a wide range of loadings. From the viewpoint of construction economy it was desirable to limit the number of types used, and it was consequently decided to divide the works into heavy and light categories. Available methods of founding included the following :—

- (1) Steel or concrete piles driven into the boulder clay.
- (2) Steel or concrete cylinders sunk to the boulder clay.
- (3) Reinforced-concrete buoyancy rafts.
- (4) Short piles driven to the stony layer at about 30 feet depth.
- (5) Reinforced-concrete surface-rafts.

The first three of these methods were envisaged for the heavy foundations and the last two for the light ones.

On an adjoining site immediately to the east of the power station, a series of steel box-piles had been driven to depths of 90 to 100 feet to found in the boulder clay. Test-loading of these showed that individual piles were capable of carrying a working load of 60 tons. However, much greater lengths would have been necessary for the majority of the foundations and both steel and concrete piles would have proved costly. Steel piles can be easily lengthened during driving but concrete piles more than 100 feet long would have been difficult to handle. In many cases of heavy load-concentrations the effects of close grouping and consequent ground disturbance would have required careful consideration.

Steel or concrete cylinders were not considered, because they would have involved slow construction and high cost.

Reinforced-concrete buoyancy rafts could be designed, for the heaviest loadings, irrespective of the thickness of soft strata. Preliminary estimates indicated that these would be less costly than piled foundations. With the exception of timber for form-work, the materials required for their construction were, early in 1949, more readily available than steel piles. By maximum standardization of detail and the use of pre-cast concrete where possible, it was anticipated that the quantity of timber required for form-work could be kept to the minimum and that available supplies would meet the needs of the job. It was therefore decided that buoyancy rafts should be used throughout the works for founding all the heavy-category loadings.

Short piles founded in the stony layer at a depth of about 30 feet and reinforced-concrete surface-rafts had been used in the original refinery, but, as previously mentioned, neither had been entirely successful. It was considered, however, that by limiting the net bearing-pressure to 500 lb. per square foot, instead of 0.5 ton per square foot as previously used, large

settlements would be avoided. Short piles could be relied on for about 15 tons each, but these possessed no technical advantage over surface rafts and would have been more costly. Consequently, surface rafts were adopted for the lightly-loaded structures.

As a result of the division of the foundation works into two categories, buoyancy rafts were used for the main building of the power station, the furnaces, reaction vessels, fractionating towers, banks of heat-exchangers, and larger pump-houses in the refinery areas ; surface rafts were employed for ancillary items in the power station, and for the control-houses, switch-houses, and smaller pump-houses.

DESIGN

The Consulting Engineers for the Grangemouth foundations had previously made use of earth buoyancy in the design of a reinforced-concrete cellular raft foundation for a large power station in India. In this case some positive net soil bearing-pressure was permissible and the foundations were built in situ without undue difficulties in retaining the sides or risk of upheaval of the bottom of large excavations. At Grangemouth the bearing capacity of the soils was so low that it was essential to make maximum use of earth buoyancy ; that is to say, the net soil bearing-pressure was reduced to zero by arranging the foundation dimensions so as to provide a balance between weight of displaced soil and weight of superstructure plus foundation. Careful positioning of the foundations beneath the loads was necessary to ensure practically uniform net bearing-pressure, for otherwise there would have been risk of tilting.

The monolith construction adopted ensured some weight on the bottom of the excavations at all times, thus reducing the tendency to upheaval of the bottom owing to removal of earth loading. The cell bottoms were plugged with plain concrete as the monoliths were reaching the final levels, so that the soil at the founding level was exposed to the action of the weather for the shortest possible time. The sides of the monoliths retained the surrounding ground, rendering side slopes to the excavations unnecessary and thus reducing soil disturbance outside the monoliths to the minimum.

Before the detail designs were commenced, a trial shaft, 8 feet in diameter and 21 feet deep, lined with pre-cast concrete pipes, was sunk with the aim of amplifying the information obtained from the soils survey. Sinking was done in 3- to 4-foot stages by grabbing below the bottom of the pipes, their downward movement being assisted by kentledge. Inflow of water occurred when sinking through the permeable layer at a depth of about 7 feet, and pumping was necessary until more pipes were placed and had formed a seal. During the remainder of the work, pumping was not required although the bottom was wet at all times.

The difficulties involved in accurately assessing the amount and probable variation of loading made it impossible to ensure perfectly uniform

soil bearing-pressure. However, with the types of structure for which the foundations were designed, load variation was comparatively small. In the power station, the load was mainly the dead weight of structure and plant, and in the refinery the dead weight of plant and liquid loading, so that the proportion of incidental loading was small.

Normally, the operating load condition was used as a basis for design, a check being applied for the effect of wind. A further check was carried out for erection loading conditions with and without the effect of wind. During the erection of the superstructure, the net effective pressure was usually negative and in a material having little shear strength the foundation might have tended to rise. Though small, however, the shear value added to the friction between the outside faces of the foundations and the soils in contact with them was sufficient to counter the tendency to uplift and tilting.

The vessels, towers, and pipework systems were, in the main, test-loaded with water before the plant was put into operation. The load from these tests was usually somewhat greater than the operating load and owing to eccentricity the net bearing-pressure was not quite uniform. The tests took only a few days to carry out and some positive net bearing-pressure was permitted. Owing to the short period of application of the test loads the effect of wind was not considered. In a few cases the full test-loading was almost double the normal operating load and arrangements were made for the testing to be done in stages to avoid large eccentricities of loading and unduly high net bearing-pressures.

In all but the smallest monoliths the wall construction was checked for stresses likely to arise during sinking of the monoliths. Two cases were considered :—

- (1) In which the monoliths were assumed to be supported by friction between the outer walls and the surrounding soil.
- (2) In which the monoliths were assumed to be supported by the soil under the walls of a few cells in the centre.

It is probable that in practice neither of these conditions was fully realized, even in the largest monoliths.

From the viewpoints of both design and construction a rectangular plan shape was desirable, and for this reason cantilevers incorporated with the structural decks were frequently employed to support outlying items. In a few foundations the size was, to some extent, restricted by the proximity of underground pipework. Such a case was monolith No. 6 (see *Figs 4*), where thirty cells in a six-by-five arrangement would have been preferable to the six-by-four adopted. Sufficient buoyancy was, however, obtained by increase in depth. Where the plant arrangement was not suitable, departure from the rectangular shape was necessary. Several such cases occurred in the topping and vacuum units, and these are shown in *Figs 4*. Cellular foundation No. 7 was built in situ after completion

Fig. 1 (a)

Fig. 1 (b)

POWER-STATION MONOLITHS DURING SINKING PERIOD

Fig. 5

COMPLETED STRUCTURAL DECK, MONOLITH NO. 5

Fig. 6 (a)

PRE-CAST WALL PANELS DURING ERECTION

Fig. 6 (b)

PRE-CAST WALL PANELS DURING ERECTION

Fig. 7

DETAILS OF PRE-CAST WALL PANELS

Fig. 8

OPEN EXCAVATION AND CONCRETE BEARING PADS BEFORE ERECTION OF MONOLITH
No. 6

Fig. 4

GENERAL ARRANGEMENTS OF FOUNDATIONS
(The units are not shown in their true relative positions)

of monolith No. 6, but in all other cases the projecting portions were integral with the monolith construction.

The power-station foundation, about 170 feet square in plan, was considered too large for construction as one unit. Sinking stresses might have been high and control of the excavation difficult. Consequently, this foundation was built as four separate monoliths, arranged with north-to-south and east-to-west alleyways about 20 feet wide between them. After sinking, these monoliths were joined throughout their depths by in-situ cellular construction to form a single foundation.

Throughout the design, the aim was to produce light-weight foundation structures so that the superstructure-loading supported per ton of earth excavated was the maximum. In practice it was found that roughly 3 tons of displaced earth provided sufficient buoyancy to carry 2 tons of superstructure. More accurate figures are given in Table 1.

TABLE 1.—CHARACTERISTICS OF BUOYANCY FOUNDATIONS

Unit	Monolith number	Number of cells	Overall depth of foundation : ft. in.	Weight of foundation : tons	Weight of displaced earth : tons	Weight of foundation per unit vol. : lb./cu. ft.
Power station	1, 2, 3 & 4 Joined by cells of similar construction.	240	15 6	8,150	23,100	40.6
Topping unit	5	23	16 0	840	2,000	48.4
	6	24	20 3	970	2,980	37.4
	7	4	12 6	260	570	52.5
	8	19	16 3	820	1,810	52.1
Vacuum unit	9	23	17 3	880	2,370	42.7
	10	55	13 0	1,700	4,300	45.5
Catalytic Cracking unit	11	30	18 0	1,130	3,250	40.0
	12	36	20 0	1,400	4,300	37.4
	13	16	20 0	670	1,990	38.7
	14	18	19 0	730	2,060	40.8
	15	45	19 0	1,640	5,080	37.1
	16	12	16 0	460	1,190	44.5
Phosphoric Acid unit	17	6	17 0	270	650	47.7
	18	15	21 0	660	1,920	39.5
	19	6	16 3	260	620	48.2

The walls forming each monolith were erected on a prepared surface above ground. The internal walls were 6-inch-thick pre-cast reinforced-concrete panels joined at the intersections by in-situ concrete. Continuity

of the horizontal reinforcement was obtained by full-strength welding of the bar ends which projected from the sides of the pre-cast panels. The 12-inch-thick external walls were of in-situ construction and were poured after erection of the internal walls.

The cell bottoms were designed for construction in two stages, first a plain concrete plug and, secondly, a reinforced-concrete seal. During the final stages of monolith sinking the plugs were employed to assist in arresting the downward movement of the monoliths and later formed a dry bottom on which the seals were laid. The seals were cross-reinforced and continuity of reinforcement in adjacent seals was obtained by splice-bars inserted through holes cored in the pre-cast walls during manufacture.

The monoliths were capped with structural decks, consisting of a 12-inch thickness of in-situ reinforced concrete laid on top of 3-inch-thick pre-cast pre-stressed concrete planks. These planks, spanning between cell walls, served as formwork for the in-situ concrete, thus rendering unnecessary the use of large quantities of timber or other shuttering which would have been difficult to remove from inside the cells. The top surface of the pre-cast planks was corrugated to assist in bonding them to the in-situ concrete, the design having provided for their combined action in resisting the stresses. The structural decks were stiffened as necessary by spreader beams and plinths to carry local loading from brickwork, structural frameworks, and plant. *Fig. 5** shows the completed deck of monolith No. 5.

The walls, bottoms, and structural decks forming the foundations were designed to resist (in combination) the main raft stresses, in addition to the stresses set up by local loading.

In the refinery-plant foundations it was necessary to take precautions against the accumulation of explosive gases in the cells. With this end in view, airtight access manholes were used in the structural decks, these being mounted on upstanding kerbs so that any oil which might be spilled would not leak into the cells.

CONSTRUCTION

Programme

To prevent risk of flooding, all areas to be developed were raised about 2 feet to bring the ground level up to + 13.75 O.D., which is the highest recorded tide-level at Grangemouth. The material used was spent oil-shale from the Oil Company's Scottish workings. This packed down to form a surface over which construction plant was able to move freely throughout the work.

The power station and topping unit were required to be in operation by the end of 1950, with other refinery units following later. Since the power station involved the greater amount of work in erection of super-structure and plant, work on its foundations was started as early as

* *Figs 5-8 inclusive are all photographs and appear between pp. 308 and 309.*

possible, that is, in April 1949, the foundations for the topping unit being started 3 months later. The remaining units were brought into the programme subsequently; work on the phosphoric acid unit started in August 1949, on the catalytic cracking unit in June 1950, and on the vacuum unit in January 1951. Other works, not described in this Paper, but which were incorporated in the programme, included a series of surface-raft foundations in the units mentioned and various treatment plants and tankage. This programme was eminently suitable for stage-by-stage construction and the Contractors were able to keep a steady labour force on the foundations until the work was all but completed.

Internal Pre-cast Walls

Details of the pre-cast wall panels are shown in *Fig. 7*. The panels were cast lying flat, in stacks of about ten, separated by layers of building paper, thus reducing the shuttering to that required for the edges. The bottoms of the panels were splayed to form cutting edges during sinking and the splays later served as skewback supports for the cell bottoms. Splays were formed along the vertical edges of the panels to permit easy access of tools for welding the horizontal reinforcement.

Access through the cells for purposes of examination and cleaning out was provided by crawl-ways formed through the wall panels. During manufacture the crawl-ways were blanked off by plain concrete diaphragms, so that individual cells could, if necessary, be filled with water during sinking operations. The diaphragms were cut away after completion of sealing the cell bottoms.

Monolith Erection

The monolith walls were erected on plain concrete bearing-pads laid directly on the ground. Small square pads were used at the intersections of the pre-cast walls and continuous strip pads under the in-situ-poured external walls. Folding wedges and packing were employed to adjust initial levels of the walls and to counteract the small settlements of the pads which occurred during erection of the monoliths. The first monoliths to be erected were Nos 1, 2, 3, and 4 (see *Figs 4*), which form the power-station foundation. These were built on top of the general site filling. To reduce excavation costs and time of sinking, the remainder were built on the floor of shallow open excavations. The procedure adopted was to excavate the general site filling and about 3 feet of depth of the surface crust, then to build the monoliths on pads laid on a 6-inch-thickness of granular filling placed in the bottom of the excavation. Drainage of surface water was effected by rubble drains laid below the filling and connected to a pumping sump in one corner of the excavation. The open excavation and concrete bearing pads for monolith No. 6 are shown in *Fig. 8*.

Erection of the pre-cast walls was commenced at one of the central

cells, and panels were added, one at a time, until all internal walls of a monolith were erected. The panels forming the first cell to be erected were assembled around a timber templet frame which served temporarily to hold the panels vertical. The completed assembly of pre-cast walls for monolith No. 4 is shown in *Fig. 6 (a)*. The methods adopted for securing the walls during erection can be seen in *Figs 6 (b)* and 7.

The horizontal reinforcement projecting from the pre-cast walls was interlocked on erection and butt-welded. The welds were of the moulded and backed type using non-recoverable mild-steel saddle-pieces. The gap left between the bar ends to be filled by weld metal was $\frac{1}{2}$ inch wide and the bars used were $\frac{1}{2}$, $\frac{5}{8}$, and $\frac{3}{4}$ inch in diameter. The junctions between the pre-cast wall panels were concreted after completion of the welding. The 12-inch-thick external walls were then built in situ in lifts, a cutting-edge to match the splays on the pre-cast walls being formed at the bottom of the inside faces.

The whole of the outside of the peripheral walls was heavily coated with bitumen to improve watertightness and to afford protection against attack from sulphates, the presence of which in the soils had been shown by chemical analysis.

Sinking

Sinking operations were commenced about a week after completion of monolith construction, when the concrete of the in-situ-poured walls and junctions between pre-cast wall panels was sufficiently matured to withstand the sinking stresses.

The concrete bearing-pads were broken away in stages to a pre-determined pattern which provided uniform support of the monolith while the bearing area was gradually reduced. Downward movements commenced before the pads were completely removed. *Fig. 9* shows diagrammatically the final stage of cutting away the pads and the excavation stages for monolith No. 15. To avoid undercutting the walls, the excavation in each cell was worked to a dish shape, the bottom of which was not more than 1 foot below the cutting-edges except for a deeper grab hole at the centre. Approximately 2-foot stages of excavation were aimed at, commencing at the corner cells and working sideways and inwards. As sinking progressed, a further lead was developed at the corners and midway along the sides so that the excavation level in the central cells was about 2 feet above that of the peripheral cells at the end of a stage and up to 4 feet at its commencement. The excavation was controlled to keep a balance about the longitudinal and transverse axes so as to reduce the tendency to tilt. Throughout the sinking operations, a continuous check on line and level was kept and to correct any tilting which developed the excavation procedure was revised, as necessary, by deeper digging on the higher side to bring the monolith to an even keel. In all cases the weight of the monoliths was sufficient to cause sinking without the use of kentledge.

Some disturbance during excavation of soft soils is unavoidable, and to minimize this the last 3 feet of excavation was done by hand.

When nearing the final level, the downward motion was arrested by plugging the peripheral cell bottoms. The excavation in each cell was cleaned out and formed to an inverted dome shape. The plain concrete plug was then poured immediately, the top surface being formed to a more shallow inverted dome shape. As early strength was essential, the concrete for the plugs was made with rapid-hardening cement. Plugging of the bottom was continued, working from the peripheral cells towards the more

MONOLITH EXCAVATION STAGES

- I. Plain concrete bearing pads cut away to pre-arranged pattern
- II. Excavation carried out in stages. Each stage commencing at 1 and working towards 4

central cells, the rate being controlled so that only a few central cells remained to be plugged when downward movement had ceased and the final level was reached.

Sealing of the cell bottoms was carried out after plugging. Protection of the reinforced-concrete seals against sulphate attack was provided in this case also by heavily coating the top surface of the plugs with bitumen.

The top surface of the seals was formed to a slight slope and holes cut through the internal walls, level with the top of the seals, afforded drainage paths to a conveniently placed sump formed in the bottom of one cell on each monolith. Ejector pumps were provided to clear periodically any water collecting in the sumps.

Structural Decks

The structural decks, consisting of pre-stressed concrete planks and in-situ concrete, were laid after sealing of the cell bottoms had been completed.

Cellular Foundation No. 7

This foundation was built in situ, inside a sheet-pile cofferdam, after completion of foundation No. 6. The piles were driven to the stony layer at about 30 feet depth. The overall width of the foundation was 12 feet. Proceeding from one end, the excavation was made and the plain concrete ground-mat and reinforced-concrete bottom-slab were poured in stages so that the bottom-slab provided a strut between the sheet-piling before excavation for the next stage was completed. The walls and structural deck were also completed within the cofferdam.

Notes on Sinking

Control of the line, level, and rate of sinking were found to be easier in the large monoliths than in the small ones. On an average the final positions were within 2 inches of alignment and 1 inch of level. Adjustments were made in the structural decks so that superstructures were set out correctly for alignment and level.

The excavation procedure shown in *Fig. 9* was adhered to for the majority of the monoliths, with minor adjustments to suit the different shapes and sizes. In small monoliths such as Nos 17 and 19, which have only six cells each, the four corner cells were excavated in advance of the two central ones.

The rate of sinking was to a large extent governed by the number of cells to be excavated and the construction plant available. In the power station unit, four large derricks capable of reaching the central area of the foundation were used; these were situated at the ends of the junctions between monoliths. Mobile plant was used in the refinery units since the monoliths were smaller and the plant was able to operate on more than one foundation simultaneously. *Fig. 10* shows the sinking record for monolith No. 15. This is typical of the refinery monoliths and indicates the rapid sinking-rate obtained during the period of grabbing, and the gradual slowing down of movement during the plugging of the cells. The fastest rate was 6 feet in one day, which was attained during the sinking of monolith No. 18. In the power-station monoliths, the average rate until commencement of plugging was about 1 foot per day. In some cases, where the sinking rate was high, excavation was carried well below the cutting-edges of the corner cells so that the plugs could be placed and the monolith allowed to sink on to them. This enabled the concrete of the plugs partially to mature before being subjected to stress. Even so, there was a number of cases where the first poured plugs fractured and had to be replaced after the remainder had been laid. This procedure was

Fig. 10

SINKING RECORD FOR MONOLITH NO. 15

normally accompanied by a certain amount of soil squeezing through between the cutting edges of the outside walls and the plugs.

No actual sequence of sinking operations as between adjacent monoliths was laid down. Generally, the larger ones were sunk first and a close watch on the line and level of these was maintained during the sinking of others nearby. Sinking of monoliths was completed prior to commencement of work on adjacent surface-rafts.

In the power station, the monoliths were sunk in the order 4-3-2-1, with the work arranged to give an interval of about 2 weeks between

a stage on any monolith and the same stage on the next one. *Figs 1 (a) and 1 (b)* show two stages during the sinking of the monoliths. The joining of these monoliths to form one foundation was started immediately after completion of bottoming operations. Sheet-piles, with walings packed off the monolith walls, were driven to close the outer ends of the junctions. Excavation proceeded from the sheet-piling towards the centre for the junctions between monoliths Nos 4-3, 4-2, and 2-1, and from the centre towards the sheet-piling for the junction between monoliths Nos 3-1, which was the last section to be done. Each junction was carried out in stages, the object being to complete the in-situ construction of the outer cell quickly and to continue excavation only when this concrete had matured. Subsequently, the work was staged in 10-foot bays so that the partially completed cellular work and the soil remaining to be excavated provided struts between the monoliths being joined, so as to lessen any tendency to unbalanced soil-pressure causing horizontal movement, and to minimize risk of upheaval of the bottom of the excavations.

In view of the experience gained during the sinking of the trial shaft previously mentioned, it was anticipated that there might be some tendency to inflow of water or soil under the cutting-edges of the external walls, particularly when sinking through the permeable shelly layer and when nearing the final levels. Serious cases of such inflow would have been controlled by immediate filling of the affected cells with water to balance the pressures, and excavation would then have continued in the wet until further sinking of the walls formed an effective seal. Inflow during plugging could have been controlled in similar manner, the plugs being placed under water with pressure-relieving standpipes passing through them. Although some inflow did occur, the amount was comparatively small and control was maintained without resort to flooding of cells.

That so little trouble was experienced from inflow of water is explained by the low permeability of the soils, and by the control exercised on the excavation level in relation to the cutting edges. Generally, the soils in the refinery sites were firmer and appeared to have a lower water-content than those in the power-station site.

When the sinking of No. 3 monolith was nearly completed there was an inflow of water to the peripheral cells on the north side. These cells were pumped out and plugged immediately. Water inflow was again experienced in the north-west corner cell of monolith No. 2 when nearing the completion of sinking. A well-point tube put down outside the monolith was sufficient to clear the water while the north-west corner and adjacent peripheral cells were plugged.

The following cases of soil inflow are worthy of note :—

(1) A small inflow of soil into the cells occurred at the south-west corner of monolith No. 2 when within 1 inch of the final level. This was stopped by immediately plugging the affected cells.

(2) Monolith No. 6 was about 6 inches above the final level and cleaning-out of the corner cells on the south side in preparation for plugging was proceeding when the monolith began to sink rapidly. Plugs were placed immediately in the corner cells on the south-side, which was sinking more rapidly than the north. Although these cracked they were beneficial in helping to arrest movement. The south side sank 11 inches and the north side 4 inches in 11 hours. The movement continued more slowly and the monolith became stable again after 4 days, by which time the south side had sunk 15 inches and the north 9 inches. The cells on the south side were then plugged, the fractured corner cells were replaced and the monolith was allowed to sink under its own weight until the top was practically level, after which plugging of the remaining cells was completed. The monolith was then $7\frac{1}{2}$ inches below the designed level and the walls were subsequently built up to correct this. The movement of this monolith probably started as a result of soil inflow on the south side, where excavation in the corner cells was dished downwards from the cutting edges of the outer walls in preparation for plugging. The inflow was quickly sealed off as the cutting-edges sank into the soil.

(3) Local patches of softer soil occasionally caused temporary loss of alignment. A typical case was monolith No. 11, which tilted 2 feet 9 inches in 27 hours owing to a soft patch of soil under the north-east part of the monolith. The monolith was about 5 feet 6 inches above final level when the tilting started and by faster digging on the higher side it was brought to within 1 inch of level during the next 3 feet 6 inches of sinking.

Some leakage of water occurred through construction joints between the outside walls and bottom seals, but pressure grouting in the joints ensured that all foundations were made reasonably watertight.

SETTLEMENTS

Settlement of the whole site resulting from natural time consolidation, which must be taking place in the Grangemouth soils, is not of practical importance when considering the foundations, since in this respect both site and foundations will settle equally. The question of relative settlement between individual foundations and adjacent ground does, however, require more careful examination.

In an ideal case, a buoyancy raft should not settle relative to the surrounding ground, because the equilibrium conditions of the founding layer are theoretically the same on completion as before commencement of the construction. As previously mentioned, however, some disturbance and possibly upheaval of the bottom of the excavation are unavoidable during construction owing to the low shear strength of the soil. Hand excavation during the last 3 feet of sinking minimized the soil disturbance, and the tendency to heaving was hindered by the cellular form of the monoliths.

Fig. 11

FOUNDATION SETTLEMENT CURVES

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The equilibrium conditions of the founding plane cannot be restored until the working load has been added to that of the foundation, that is, until the construction of the superstructure or plant supported is completed. Some settlements are to be expected during this construction period and will continue afterwards until the soil has recovered its capacity for passive resistance, which must have been to some extent destroyed by the disturbance during excavation.

Previous mention has been made of a series of reinforced-concrete surface-rafts that was used for founding the lighter load-concentrations. These were designed for a uniform soil bearing-pressure and were founded as near the surface as plant requirements permitted, so as to take maximum advantage of load-spread through the general site filling and the tougher surface crust. These foundations will, of course, be subject to some settlement owing to compression of the ground.

It was estimated that probable settlements during the lifetime of the plant would be about 1 inch for the buoyancy rafts and about 3 inches for the surface rafts. Allowance for this was made in the initial foundation levels by siting the surface rafts 1 inch higher than the buoyancy rafts, so that eventually the surface rafts may be 1 inch or so below the buoyancy rafts.

Fig. 11 shows settlement curves to date for some of the first completed foundations. Curve E is for a surface raft and the others are for buoyancy rafts. Settlements of the remaining foundations, not shown in *Fig. 11*, appear to be following the same general trend.

CONCLUSIONS

Although the principle of using net soil bearing-pressures is firmly established, its use in the limiting case, where net pressure is reduced to zero and the foundation becomes earth buoyant, is rare. The Authors believe that it has not been previously developed on the scale employed at Grangemouth. A total superstructure and plant load of about 37,000 tons has been carried on a series of fifteen separate foundations, which range in size from about 170 feet by 170 feet down to 33 feet by 22 feet 6 inches. The maximum depth of excavation was 24 feet 6 inches, for monoliths Nos 12 and 13, the top of these being 4 feet 6 inches below ground level. This is considerably in excess of the depth to which open excavation could have been made. Some settlements were envisaged from the outset and a close watch will continue to be kept on these.

ACKNOWLEDGEMENTS

For permission to produce this Paper the Authors are indebted to the Anglo-Iranian Oil Co. Ltd, Scottish Oils Ltd, and Messrs Rendel, Palmer and Tritton, the Consulting Engineers for the works described.

The principal Contractors were George Wimpey and Co. Ltd, and the soil investigations were carried out by Soil Mechanics Ltd.

The Paper is accompanied by seven photographs and seven sheets of drawings, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

The Authors introduced the Paper with the aid of a series of lantern slides.

Mr A. C. Hartley observed that when any large enterprise had to be planned, the site should be chosen from very many aspects. The commercial points of view, which would be operative throughout the whole existence of the enterprise, were perhaps fairly predominant. In the case of an oil refinery, the site had to be suitable for bringing in the crude oil and have assured availability of power, water, and the labour required to operate the plant. It had to be well placed from the point of view of road, rail, and sea transport, to export the products of the refinery. As each enterprise was built, it might be assumed that the best available sites were chosen, so that in later years it became more difficult to find a suitable site and more and more impossible to find all the conditions that were wanted. Moreover, the unseen conditions—those below the ground—were liable to be forgotten at first.

In the case in question it had been known that the site was a very difficult one. As the Authors had explained, the refinery had been there for nearly 30 years, but the loads due to each new plant had tended to increase. Settlement could no longer be tolerated between parts of modern units. There were very large high-pressure high-temperature pipes which connected the units and sections of units, and there was a definite limit to the settlement which was tolerable. The Paper indicated, therefore, one of the greatest services that civil engineers could render to industry—finding ways of doing the almost impossible and providing a reasonably economic foundation in such difficult conditions.

In the case in question, a very great service has been done by the soil mechanics experts and by the early investigation of the site. The Anglo-Iranian Company had made use of their geo-physical exploration facilities and had been able to get a very good underground contour plan, as shown in *Fig. 2* of the Paper. It gave an opportunity of assessing, in the near future, by watching settlements and so on, at what contour it paid to change, other things being equal, from piles to buoyant rafts. He did not think that there was any doubt that it had been right to use, in the associated Company, piles when there was boulder clay at a depth of about 80 feet,