Model Studies for Shaft Raising through Cohesionless Soils

Etudes sur modèle du soulèvement des puits dans des sols pulvérulents

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SUMMARY

At Sizewell Nuclear Power Station, eight vertical shafts were constructed from the cooling water tunnels by jacking cylindrical linings vertically from the tunnel to sea bed level. This shaft raising was carried out in cohesionless soil through a depth of about 20 ft. The problem of determining the required jacking loads is similar to that of the tensile loads required to pull out pylon foundations. As no satisfactory theory exists for the calculation of these loads, the problem was resolved by the use of model experiments. The jacking loads required in the field fell within the range of those predicted from the model studies.

SOMMAIRE

On a construit huit puits verticaux à la station nucléaire de Sizewell en soulevant verticalement des caissons cylindriques à partir du tunnel d'eau de refroidissement jusqu'au lit de la mer. La construction de ces puits fut effectuée dans un sol sans cohésion sur une distance de 6.50 mètres environ. Le problème du calcul des charges de soulèvement ressemble à celui des charges de traction nécessaires pour arracher les assises d'un pylône. Aucune théorie adéquate n'existe pour permettre le calcul de ces charges; le problème fut résolu au moyen d'essais sur modèle. On a constaté que les charges de soulèvement requises demeuraient effectivement dans les limites prévues par les essais sur modèle.

ON THE COAST OF SUFFOLK, at Sizewell, which lies about 90 miles northeast of London, a nuclear power station of 580-megawatt output is being constructed. The circulating water system for the station is now complete, and involved the construction of four tunnels driven from the shore under the North Sea. The tunnels are 11 ft in diameter and are formed from cast iron segments. They were driven through cohesionless soil, using compressed air to exclude the entry of sea water during construction.

Two of the tunnels are for the cooling water intake and two are for the discharge of the cooling water. At the seaward end of each tunnel, two shafts extend vertically upwards from the tunnels, through the sea bed, to the water. The shafts are 7 ft 10 in. in outside diameter. The conventional method of constructing these shafts would be by sinking caissons through the sea bed to tunnel level, pumping them dry, and connecting them to the tunnels. At Sizewell, however, a novel procedure was used and the shafts were raised from the tunnels by thrusting vertical shafts, with solid ends, up to and through the sea bed. The shaft-raising operations took place at distances of up to 1200 ft from the shore. There was about 20 ft of sand cover to each tunnel and a 20 ft depth of water.

This paper describes the methods used to determine the jacking forces required to raise the shafts. Model studies were made since there is no reliable theoretical solution to this problem. The jacking forces measured in the field are compared with the values obtained from the model investigations.

STATEMENT OF THE PROBLEM

The Sizewell shafts were raised through a cohesionless soil and the forces which had to be considered in the jacking operations were: (1) the force mobilized on the closed end of the vertical shaft because of the effective weight of the overlying material and the friction developed along the rupture surface in the sand mass; (2) the force mobilized by skin friction around the perimeter of the vertical shaft after jacking commenced; (3) the force resulting from the pressure of the water on the closed end of the shaft; (4) the weight of the shaft lining; (5) the force resulting from the pressure of the compressed air within the tunnel and shaft. For any given case, factors 3, 4, and 5 can be calculated. As the vertical shaft rises, the skin friction, force 2, builds up at a lesser rate than force 1 decreases.

The problem therefore reduces to the determination of the force required to push the closed end of the vertical shaft away from the tunnel lining. It is a similar problem to that of the design of pylon foundations where the foundation consists of a vertical shaft with a circular base slab of greater diameter than the shaft. This base slab provides resistance against failure caused by a vertical tensile load.

Given the density and shear-strength characteristics of a soil, the jacking force can be calculated if the form of the failure surface is known. Various proposals have been made as to the form of failure surface and these will be discussed later. A number of preliminary tests were made in the University of Glasgow to investigate the failure surface, but the results obtained showed no pattern of failure surface which could be applied to all states of density for the sand used in the tests. The various theoretical forms of failure surface give a wide range of results for the jacking force required on the closed end of the shaft. Since the shaft-raising operation required loading to failure in the field, it was considered that a theoretical determination of the jacking force would not provide a satisfactory solution to the problem.

Rocha (1957) drew attention to the potential advantages of the use of models in soil mechanics problems and pointed out the limited use that has been made of them to date. It was decided to investigate the Sizewell shaft-raising problem using models. The similarity between model and prototype was established from dimensional analysis, and for the case of the cohesionless soil at Sizewell, the following relationship between dimensionless products was obtained:

$$p/\gamma D = f[(D/B)\ \phi], \tag{1}$$

where p = jacking pressure required at the closed end of the vertical shaft, γ = effective unit weight of the cohesionless soil, B = diameter of vertical shaft, D = depth of soil overburden above the closed end of the shaft, ϕ = angle of shearing resistance of soil, P = total jacking force = $p \times \pi/4 \times B^2$. It can be concluded from Eq 1 that for any cohesionless soil of known ϕ: (a) $p/\gamma D$ depends on D/B alone; (b) for a given D/B, $p/\gamma D$ is constant irrespective of the values of D and B, i.e. $p/\gamma D$ from the model applies equally well to the prototype; (c) a plot of $p/\gamma D$ against D/B gives a solution for a soil of known ϕ. The complete solution is a family of curves each corresponding to a particular ϕ.

The programme of model tests was planned from these conclusions. The jacking pressure was measured for various combinations of D and B using sand of known γ and ϕ. The effective unit weight of the sand was varied by preparing it in dense and loose conditions, and also by carrying out tests on inundated sand.

APPARATUS AND TESTS

The shaft raising in the field was carried out by jacking the closed end of the shaft from within the tunnel. In the laboratory tests, the jacking operation was simulated by pull-out tests. These consisted of measuring the load required to pull out discs of different diameters, B, from different depths, D, from sand which was placed in different states of density.

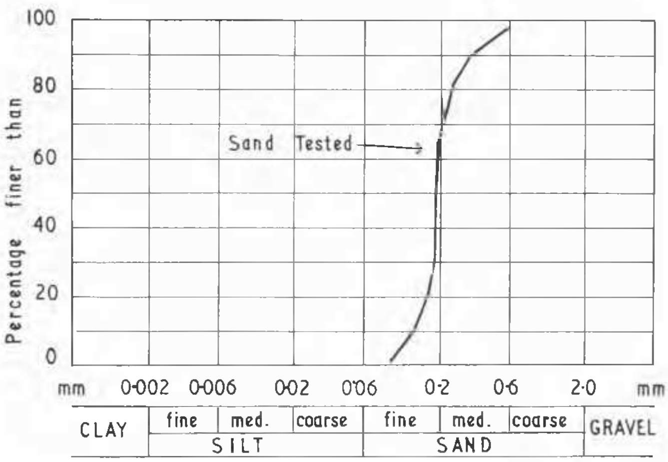

FIG. 1. Particle size distribution of soil tested.

Two sand containers of different sizes were used. The first container was 30 in. in diameter and 14 in. deep. The load was applied by a jack with a uniform rate of load application. The load was measured using a calibrated spring balance. Discs with diameters, B, ranging from 1½ in. to 6 in. were used. These discs were placed at known depths below the surface of the sand and D/B values from 1 to 5 were investigated. This container was used for tests on oven-dried sand which had the grading characteristics shown in Fig. 1. This grading was similar to that of the sand at the tunnel face. The sand was investigated in a very dense and a very loose condition so as to cover the range of jacking pressures which could be developed. Techniques were developed whereby these densities could satisfactorily be reproduced.

Triaxial compression tests, made on sand samples prepared at these densities, gave $\phi = 45°$ for the dense state and $\phi = 31°$ for the loose state. Twenty-three pull-out tests were made on the dense sand and twenty-two on the loose sand using the first container. Each test was made on freshly

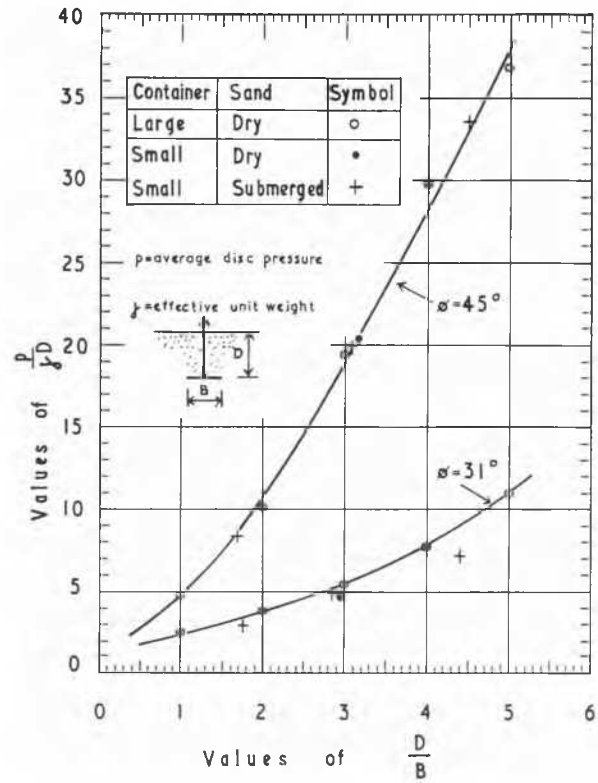

FIG. 2. Model test results.

placed sand. The average value of $p/\gamma D$ for particular values of D/B (based on tests with different D and B values) are given in Table I. The range of $p/\gamma D$ values contributing to the average $p/\gamma D$ for a particular D/B was small. The results are plotted on Fig. 2.

TABLE I. AVERAGE VALUES OF $p/\gamma D$ FOR PARTICULAR VALUES OF D/B AS DETERMINED BY TESTS USING DIFFERENT D AND B VALUES

		Values of $p/\gamma D$ for				
Sand	ϕ	$D/B = 1$	$D/B = 2$	$D/B = 3$	$D/B = 4$	$D/B = 5$
Dense	45°	4.8	10.1	19.3	29.8	36.9
Loose	31°	2.5	3.8	5.4	7.7	11.0

The second container was smaller than the first, being 18 in. long, 9 in. wide, and 12 in. deep. Since the jacking operation in the field was to be carried out in inundated sand, laboratory tests were made to investigate the effects of submergence on the sand. The use of a smaller container for these tests cut down the work of placing and drying the wet sand.

Before any tests were made on inundated sand, a number of check tests were run in this container using oven-dried sand, placed at the same densities used in the large container. These test results are shown on Fig. 2. They agreed with those obtained in the larger container. A number of pull-out tests were made for the sand in an inundated state and the results are also shown on Fig. 2. The $p/\gamma D$ values plotted are for effective unit weight values for the sand and confirm the theoretical reduction of pull-out force due to submergence.

The capacity of the jacking system used to raise the shafts was determined from the model tests and provision was made for jacking the shafts through a very dense sand.

THEORETICAL SOLUTIONS

Reference has been made to theoretical methods of obtaining the pull-out force required to cause failure of pylon foundations. Balla (1961) reviewed and compared these methods and in turn proposed a method of his own. Each method (including Balla's) makes an assumption as to the form of failure surface developed.

Balla carried out a limited number of model experiments on the pull-out loads developed in an air-dried sand with $\phi =$ 36° to 38°. From these tests, he concluded that the failure surface was a circular curve starting with a vertical tangent to the base slab and curving outwards from the vertical axis to meet the ground surface at an angle of $(\pi/4 - \phi/2)$. He concluded that this form of failure surface applied to all values of ϕ.

The limitations of the Balla approach become apparent when his formula is reduced to the form of Eq 1. Values of $p/\gamma D$ from his formula can then be plotted against D/B. This has been done for $\phi = 31°$ and 45°, the ϕ values found for the sand in the Glasgow tests, and the results are plotted on Fig. 3. These show that the jacking pressures obtained from Balla for $\phi = 31°$ are about 90 per cent of those obtained for $\phi = 45°$. This small range implies that the frictional properties of the sand have little effect on the jacking pressures and his theoretical results are not consistent with the values obtained from the tests described in this paper and which are also shown in Fig. 3.

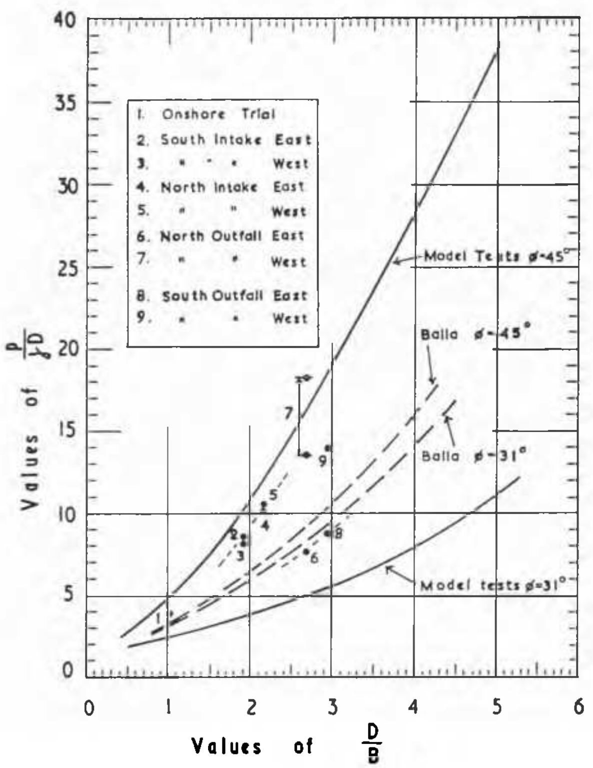

FIG. 3. Comparison of theoretical, model test, and site results.

Balla justified his theory by applying it to field test results. In one set of field tests where ϕ was 40°, he obtained reasonable agreement. This could be expected since the ϕ value was close to his laboratory test value. In the other field test, where ϕ was 30° the agreement was not good and, in fact, two of the three tests where ϕ was 30° gave $p/\gamma D$ values close to the $\phi = 31°$ curve found in the Glasgow tests.

A further series of tests using a variety of methods have been made in the University of Glasgow to investigate the form of failure surface in this problem. These tests have covered a range of ϕ values and are still continuing, but so far no one failure surface has been defined which will apply to all states of density of a cohesionless soil.

SHAFT-RAISING OPERATIONS AT SITE

A total of eight shafts were raised from the tunnels. Before these operations, a trial shaft was raised onshore to develop the necessary techniques. Four of the eight shafts were grouped at the seaward end of the intake tunnels and four at the seaward end of the outfall tunnels. The jacking loads, height of sand cover, and the level of the sea were measured at each of the eight shafts. Samples of sand were taken from the face of the outfall tunnel during driving. The effective unit weight was found to be 66 lb/cu.ft. The compressed air pressure in the tunnels during the shaft raising equalled approximately the pressure of the water outside the tunnels. The initial jacking loads applied had therefore to overcome the resistance caused by the effective weight of the overburden and the shear resistance mobilized along the failure surfaces. Table II gives a summary of the data regarding the shaft raising for the eight shafts and also for the onshore trial. The results have also been expressed in Table II in terms of the dimensionless factors $p/\gamma D$ and D/B and the nine points obtained are plotted on Fig. 3.

Shafts 2, 3, 4, and 5 were grouped together in the intake area. The results from these four shafts were consistent and indicated a ϕ value for the overburden in this area of about 42° based on the model test results.

Shafts 6, 7, 8, and 9 were grouped together in the outfall area. During the tunnel driving, a "blow" occurred and about 20 tons of cement had to be injected in the area of the west shaft at the north outfall. It was anticipated that this grouting would affect the jacking loads required at this shaft, shaft 7, and also at shaft 9, the west shaft on the south outfall tunnel. The effects of the grouting can be seen in Table II and Fig. 3. The variation in jacking load of 400 to 540 tons for shaft 7 corresponded to the loads required for spasmodic

TABLE II. SUMMARY OF DATA AND DIMENSIONLESS FACTORS $p/\gamma D$ AND D/B, FOR NINE SHAFT RAISING OPERATIONS

Shaft No.	Location	Shaft	Diameter, B (ft.)	Sand cover, D (ft.)	Sand resistance, $p(\pi/4)B^2$ (tons)	$p/\gamma D$ (γ = 66 lb./cu.ft.)	D/B
1	Onshore trial	—	7.83	8	45	3.97	1.02
2	South intake	East	"	15	184	8.65	1.91
3	South intake	West	"	15	176	8.29	1.91
4	North intake	East	"	17	256	10.62	2.17
5	North intake	West	"	17	250	10.38	2.17
6	North outfall	East	"	21	232	7.8	2.68
7	North outfall	West	"	21	400 to 540	13.4 to 18.2	2.68
8	South outfall	East	"	23	288	8.85	2.94
9	South outfall	West	"	23	450	13.85	2.94

initial movements before the shaft actually raised. The other two outfall shafts, 6 and 8, were unaffected by the grouting and gave similar results which correspond to a ϕ value of about 35° based on the model test results.

The loads required to raise shafts 2, 3, 4, and 5 were about 50 per cent greater than those given by the Balla theory for the same conditions, i.e. $\phi = 42°$. For shafts 6 and 8, where ϕ appeared to be about 35°, the actual jacking loads were 88 per cent of those obtained by Balla. This theory for the shaft-raising problem gives errors on the unsafe side for denser sands and errors on the safe side for looser sands.

The jacking loads required for the shafts in the intake area indicated that the overburden in this area was denser than that at the outfall area. Boreholes had been sunk at both locations during the site investigation and showed, on average, dense sand at both the areas. However, in the intake area, gravel and cobbles were reported in the upper layers, and some iron-cementation of the sand was also found. These factors probably contributed to the higher frictional properties at the intake area, but it was impossible to investigate with any accuracy the frictional properties of the sand throughout the depth from sea bed to tunnel level.

CONCLUSIONS

1. No satisfactory theory is available for the calculation of jacking pressures in shaft-raising problems in cohesionless soils or for the vertical pull-out loads required to cause failure of pylon foundations in such soils. Balla's theory is that most recently proposed, but model studies and site tests indicate that this theory would underestimate the loads mobilized in dense cohesionless soils. This would constitute an error on the unsafe side for shaft raising and an error on the safe side for pylon foundations. For loose cohesionless soils, the theoretical loads would be overestimated, giving safe errors for shaft raising and unsafe errors for pylon foundations.

2. Dimensionally similar models can be extremely useful in soil mechanics for solving problems where theoretical solutions are not available.

3. Further work is required to evaluate the forms of failure surface to enable a more satisfactory theory to be developed. Until this is done, the experimental curves presented in this paper offer a more reasonable basis than theoretical solutions for the design of pylon foundations in cohesionless soils.

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