ting expansion joints, the changes in temperature cause secondary stresses in the concrete slabs, which can be of the same magnitude as the stresses resulting from wheelloads. A conclusive solution for the construction of expansion joints not yet having been invented, these joints are more especially troublesome in the case of soft subgrades. The flexible pavement on the other hand does not spread the load so well but it constitutes a continuous, impervious carpet, largely unaffected by temperature changes.

However, if a concrete slab is laid at a depth below ground surface, sufficient to neglect the influence of temperature changes, the expansion joints may be omitted. To transmit the pressures of the airplane wheels, it is then required to cover the concrete slab with materials able to withstand the resulting stresses, which provides the additional advantage that the pressure on the concrete slab is spread over a larger area.

The new N.W. to S.E. runway has now been constructed on these lines as follows (fig.15). Under the 5 cm asphaltic concrete toplayer lies first a 25 cm rolled stone base, next a 35 cm sand layer and lastly a 30 cm unreinforced concrete slab, for which the concrete mix is 1 cement on 2½ sand on 8 gravel (volume measurements), corresponding to 175 kg cement per m3 concrete. After placing the concrete, the slab is compacted by rollers and at once covered with moist sand.

The asphaltic concrete toplayer consists of 65% aggregate of 5-20 mm (porphyr), 23% sand, 6% filler and 6% bitumen (60-70) and is surface-treated. In addition to the rolled stone base of 25 cm thickness the depth of the sand layer has been taken at 35 cm in order to keep the concrete slab below frost depth and also to keep sufficient depth in hand for the crossing of cable conduits and pipes. The waterproofing of the surface by the asphaltic wearing coarse together with an efficient drainage at the edges, see to it that rainwater is carried off before it has time to penetrate into the sand layer, and this adds to the stability of the construction.

A subsidiary advantage of this construction is that the laying of cables and conduits is facilitated. They can be laid directly on the concrete slab and the surface carpet with its base can here, in contrast to a concrete surface, easily be broken up and repaired again. Also, alterations to the runway profile can be achieved fairly easily, and in such cases, the base material can be used again. If strengthening of the runways in the future might become necessary this may be obtained by substitution of part of the sand by an additional concrete layer on top of the existing one. Furthermore, the presence of the rigid concrete slab will tend to reduce any deformations of the asphaltic toplayer and its stone base, with considerable benefit to the life time of the construction.

If a crack should occur in the concrete slab a failure will not ensue as the slab will still continue to spread the pressures. In contrast cracks in a bare concrete runway cause rapid desintegration of the slabs.

Remained the question whether the construction would indeed prove to withstand high loads and whether the dimensions of its parts, as designed, would constitute a well balanced total. To solve this question, test loads were made, which will be described and discussed in a separate article. 5) When judging the results of the test loads attention should be paid to the fact that the bearing capacity of the described "sandwich" construction may be considerably augmented by using concrete of better quality.

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VIII d 5 DETERMINATION OF THE BEARING CAPACITY OF THE NEW COMBINED RIGID-FLEXIBLE RUNWAY CONSTRUCTION AT SCHIPHOL AIRPORT WITH THE AID OF LOADING TESTS

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INTRODUCTION

The tests at Schiphol Airport have been performed by the Construction Department of the Amsterdam Public Works in collaboration with the Delft Soil Mechanics Laboratory. Moreover assistance was given by the Laboratory "Bataafsche Petroleum Maatschappy".

The object of the here described tests was to check the bearing capacity of the combined rigid-flexible runway construction at Schiphol Airport 1) for aircraft with a maximum gross weight of 135 tons (300,000 lbs) and tyre pressures of 8.5 kg/cm^2 (120 lbs/sq.in). The P.I.C.A.O. in 1945 recommended for class A air-

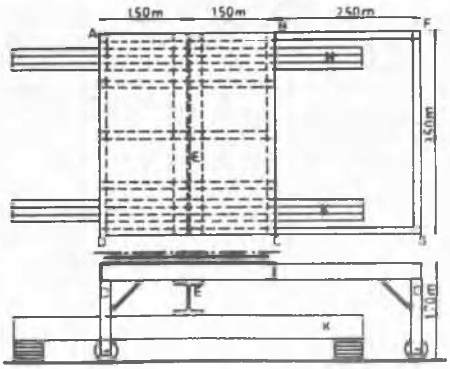

FIG. 1

FIG. 2

FIG. 3

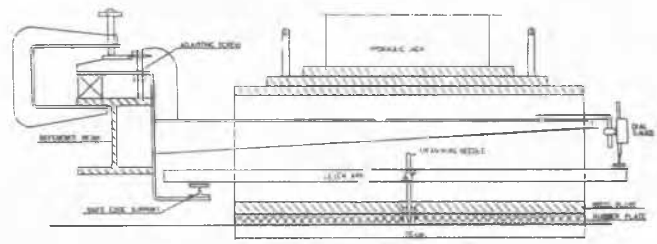

FIG. 4

ports single wheel loads of 67.5 ton, in 1946 altered to dual wheel loads of 67.5 ton and in 1947 to single wheel loads of 45 ton.

First, the tensile stresses which may be expected in the concrete slab have been calculated with the aid of the Westergaard formulas, and these stresses have been compared with the flexural strength. For this procedure it was required to know the value of the modulus of subgrade reaction on the one hand and of the modulus of elasticity, Poisson's ratio and the flexural strength of the slab in question on the other hand. Also required were the factors in the "revised" formulas of Westergaard 3) 4), determining the adjustments of the deflections and stresses, caused by a redistribution of the subgrade reactions. Finally the load spreading capacity of the layers above the concrete slab had to be known. In order to obtain these informations, in test sections loading tests have been performed on the natural foundation as well as on the concrete slab and on the construction as a whole.

Secondly, the maximum bearing capacity of the layers above the concrete slab has been determined with the aid of the information obtained at the loading test on the construction as a whole.

The loading tests were performed with a specially built loading platform on wheels, as shown schematically in fig. 1. The ballast consisted of cast iron plates of more than 100 kg each, which were placed by means of a crane in piles of 10 on part ABCD of the platform. Under the middle of beam E a hydraulic jack was placed, as shown in fig. 2. By raising this jack the weight of platform and ballast was transmitted on a rigid bearing plate (dia. 75 cm), under which a 2½ cm rubber plate in order to obtain a more uniform pressure distribution.

It appeared feasible to place the ballast in such a way, that supports A, F, G and D were standing free and the platform was resting in perfect balance on the jack. Lowering the jack the weight came to rest on the cross beams H and K and the bearing plate was released. Fig. 3 gives a view of the loading platform with ballast (total 65 tons), and shows clearly the crane, the cross beams, the reference beam and the rain- and windcreens above the reference beam.

Two different methods of loading were possible with this arrangement, each with its separate use:

1) Gradual loading with the piston of the jack pumped up, the cross beams H and K not functioning.
2) With one definite load pumping the piston up and releasing it again, allowing quick repetitions of the same load.

MEASURING APPARATUS.

Deflections of the surface were measured at several distances from the centre of loading by means of dial gauges, fixed on a 15 m steel reference beam protected against influences of the weather. In order to measure also deflections at the centre of loading, a special construction was provided as shown in fig. 2 and 4. The dial gauge was fixed to the reference beam, while movements of the measuring needle in the centre were transmitted by means of a lever arm.

Moreover, at the combined construction deflections of the underlying concrete slab were measured with the aid of steel rods embedded in the concrete (fig. 5, left). These were shielded against the influence of friction of the compressible layers above by telescopic tubes with an outside rubber coating.

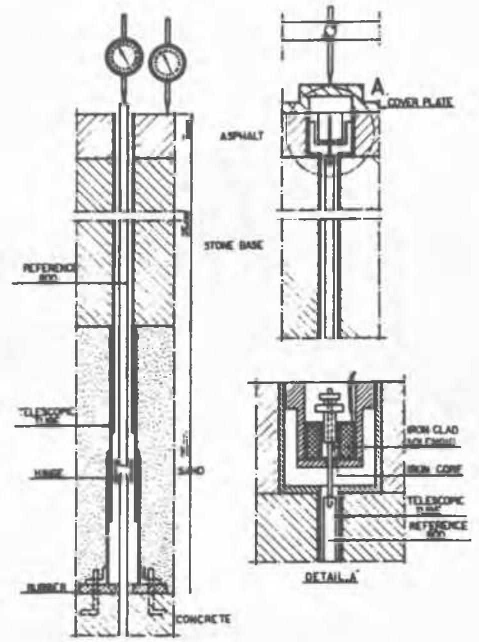

FIG. 5

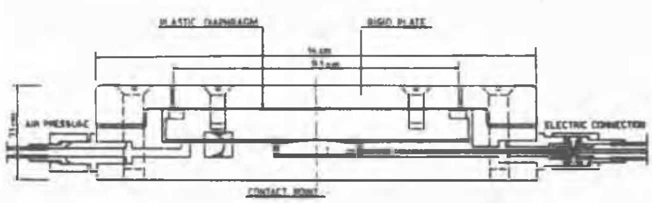

FIG. 6

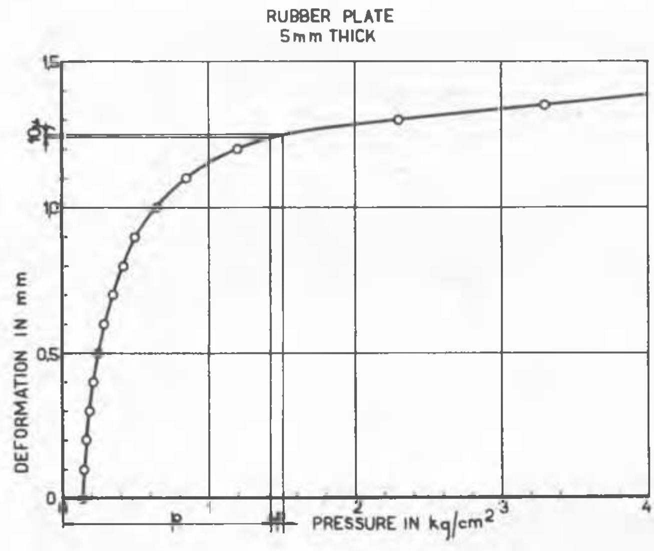

FIG. 7

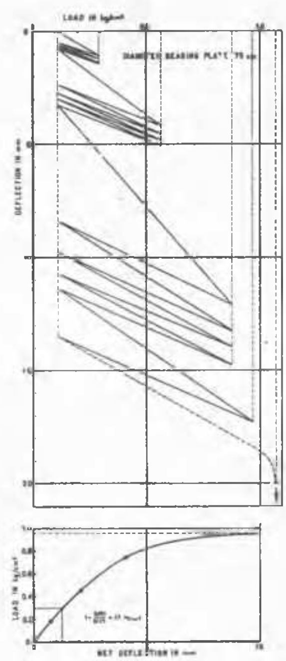

FIG. 8

Under the centre of loading the difference of the deflections of concrete slab and loaded surface were measured by means of an electric gauge 5) (fig. 5, right).

In order to measure pressure distribution in the layers above the concrete slab Goldbeck pressure cells 6) were used because of their simple construction and the small movements of the diaphragm. The cells were embedded in the concrete, their tops being flush with the slab surface. After preliminary tests some improvements were made in the construction (fig. 6) e.g. the electric and the compressed air systems were separated, while in connection with the desired water and airtightness the copper diaphragm was replaced by a plastic one (thick 0.2 mm). Breaking and closing of the contact were registered by a relais, so that sparks were prevented and deterioration of the contact point in the pressure cell was not to be feared.

These cells require a movement of the rigid plate against the earth pressure to break the circuit at the contact point, so that an excessive pressure is measured. These movements have been measured in the laboratory and amounted to 10 micron at most for pressures higher than 1 kg/cm^2, when after each measurement the air pressure was allowed to drop to atmospheric. For lower pressured they might go up to 35 micron. Closing the circuit movements were about 5 micron less. The difference at breaking and closing the circuit is due to the compression of the contact point. In order to minimize the influence of movements of the rigid plate on the measured pressures, a 5 mm thick rubber plate was laid over the pressure cell, which at the same time enabled one to know the limit of the excess of pressure. The influence of the compression of this rubber plate caused by the overburden, on the pressure distribution above the concrete slab, was forestalled by giving the rubber plate a sufficient size.

Deformations of the rubber plate at increasing loads are given in fig. 7. A definite deformation corresponds with a given pressure p on the rubber plate. Now the excess of pressure at breaking the contact will always be less than the pressure Δp, required in case the total movement of the rigid plate had to be taken up in the rubber plate alone. For it is clear that, in order to enable the excess of pressure Δp to come about, the earth above the rubber plate must make a movement, which in its turn reduces the deformation of the rubber plate. Hence the real excess of pressure lies between 0 and Δp. This limiting value Δp has been measured in the laboratory and amounted to an average of 5% (maximum 7½%) of the applied pressure. This obtains also for the lower range of loads, because the larger movements of the diaphragm are then compensated by the lower modulus of elasticity of the rubber.

MODULUS OF SUBGRADE REACTION

To determine the modulus of subgrade reaction field bearing tests have been made. At these tests 7) a rigid bearing plate with diameter 75 cm (30") was loaded with several loads, each successive load being repeated at least 3 times, so that the deflection of the foundation soil at the last repetition of the same load approached to a constant value (fig. 8). Each loading and unloading was maintained during 5 minutes, after which the rate of deflection proved to be very small. The modulus of subgrade reaction k has been calculated for deflections of 1.25 mm (0.05"). At greater deflections smaller k-values were found, which may be explained by the influence of plastic deformations, which however will not occur to a great extent under the concrete slab.

It was evident from results of loading tests on the bare concrete slab, that the actual modulus of subgrade reaction was less. From additional field bearing tests with plate diameters of 100, 75 and 50 cm the average k-values appeared to be almost inversely proportional to the diameter d of the bearing plate (fig. 9). The values for diameters of 100 cm were in fair agreement with the values found at the loading tests.

Since at Schiphol the k-value varied from 2.0 - 3.0 kg/cm³ for diameters of 75 cm 1), it follows from the above mentioned linear relationship that k can vary from 1.5 - 2.25 kg/cm³ for diameters of 100 cm.

MODULUS OF ELASTICITY AND POISSON'S RATIO OF THE CONCRETE SLAB.

The modulus of elasticity of the concrete slab has been determined at several places by the Laboratory B.P.M. by means of vibration measurements 8), average values over a distance of about 50 m being measured. There was only little spreading in the results and the average was found to be 76,000 kg/cm². Moreover, the modulus of elasticity has been determined by means of bending tests on beams sawn out of the slab. More spreading occured here, but the average worked out at about the same, Since for computations concerning the concrete slab it is necessary to make use of the average modulus of elasticity over a certain distance, the value of 76,000 kg/cm² seems reliable. This relatively low value is due to the low cement content of the concrete.

Poisson's ratio does not have a great influence on the results of the calculations. In keeping with the information available in the literature, a value of 0.15 was chosen.

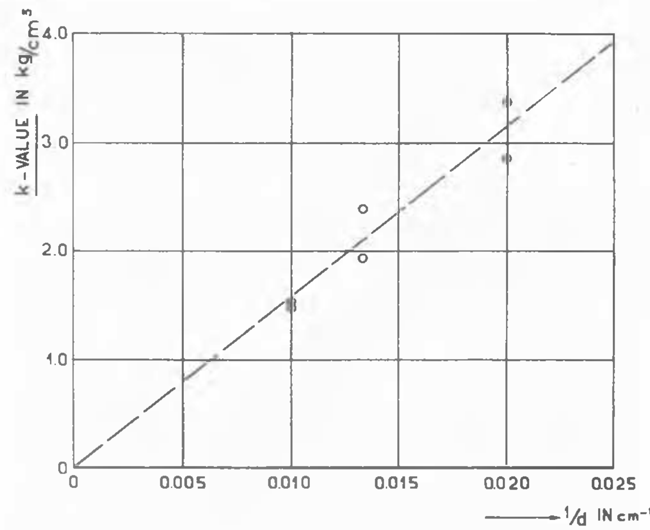

FIG. 9

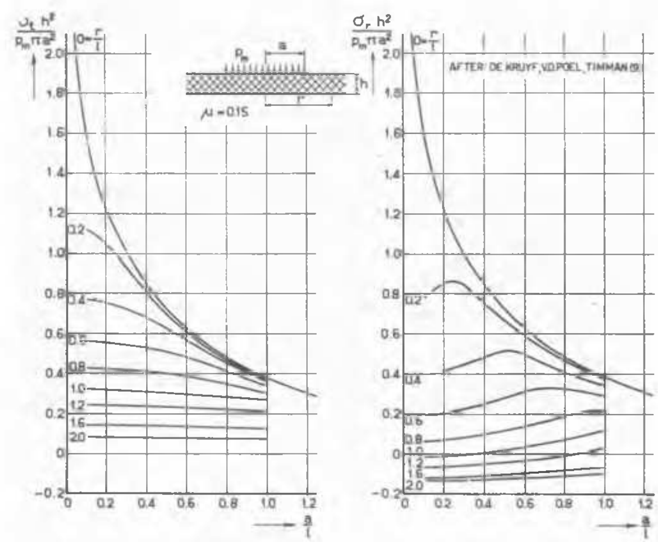

FIG. 10

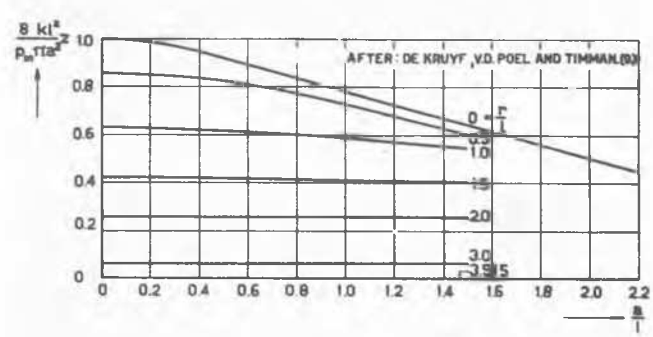

FIG. 11

LOADING TESTS ON THE BARE CONCRETE SLAB.

For a uniformly distributed load p_m on a circular area in the centre of a sufficiently large elastic concrete slab, the maximum tangential and radial tensile stresses σ_t and σ_r and the corresponding vertical deflections z are given as a function of the radius a of the loaded area and the distance r to the load centre in fig. 10 and 11, according to a paper by de Kruyf, v.d. Poel and Timman 9). These graphs are based on the same assumptions as the "original" and "corrected" formulas of Westergaard, which, however, give only the stresses and deflections in the centre of the loaded area, resp. for a point load 2) and for a uniformly distributed circular load 4). In the graphs a and r are expressed in the radius of relative stiffness l, which depends on the thickness h, the modulus of elasticity E and Poisson's ratio μ of the concrete slab and on the modulus of subgrade reaction k:

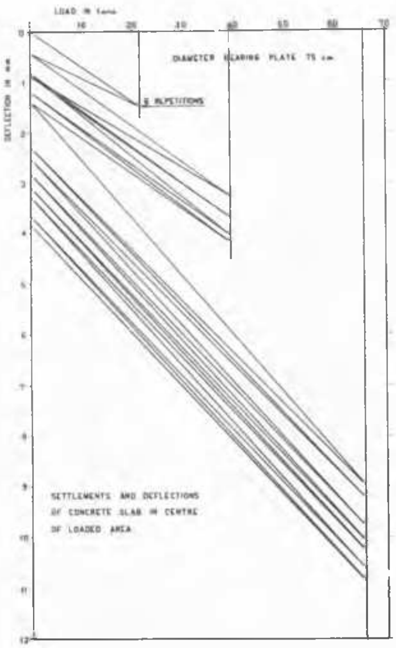

FIG.12

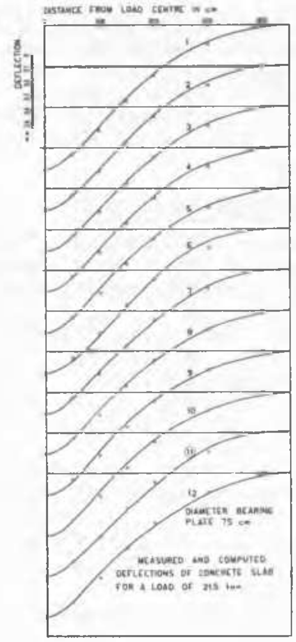

FIG.13

$$l^4 = \frac{E h^3}{12(1-\mu^2)k} \qquad (1)$$

However, the reaction of the subgrade per unit of area at each point is assumed to be proportional to the deflection of the slab at that point. But according to the "revised" theory of Westergaard 3) a redistribution of subgrade reactions is possible, more closely concentrated around the load than are the deflections. This corresponds with a reduction of the deflection z (z_1 in the centre of the load) with z' (z_1' in the centre of the load). Then

$$C = \frac{z_1'}{z_1} \quad (2) \qquad z' = z_1'\left(1 - \frac{r^2}{L^2}\right)^{10} \quad \text{when} \quad r < L \quad (3),$$

$$z' = 0 \quad \text{when} \quad r > L \quad (4)$$

C is the ratio of reduction of the maximum deflection; L is the value of r limiting the range within which the adjustments are made. The corresponding reduction of the maximum tensile stress under the centre of the load is

$$\sigma_1' = 120 \frac{k l^4}{h^2 L^2}(1+\mu) C z_1 \qquad (5)$$

Hence the influence of the redistribution of soil reactions on deflections and stresses is determined by the factors C and L. According to Westergaard the maximum value of C for a point load is 0.39, while a plausible value for L may be 5 l (from an approximate analysis under the assumption that the subgrade has a constant modulus of elasticity in compression and a constant Poisson's ratio). One of the objects of the loading tests was to determine the values of C and L for the concrete slab as constructed at Schiphol, in order to use these in the calculation of the combined rigid-flexible runway construction.

At the loading tests on the bare concrete slab the first cracks started to develop at a load of 20 to 25 ton on the 75 cm dia. bearing plate. Since the highest tensile stresses occurred at the bottom of the slab, the existence of these cracks could only be established indirectly by irregularities in the deflection curve and by an increasing deflection at repetitional loading. The latter is evidenced by fig. 12, which shows successive deflections at the centre of the load for one of the tests. At the lowest load (21.5 ton, 6 repetitions) the slab had not yet cracked, which is in accordance with the completely elastic deflections after the first settlement. Hence these elastic deflections have been used for determination of C and L.

Based on a provisonally estimated and lateron corrected value of l, the theoretical deflection curves for various combinations of C and L have been plotted with the aid of fig. 11 and formulas (2), (3) and (4), approximating as far as possible the measured deflections by varying the value of I. The results are shown in fig. 13. The deviations of all these curves from the measured points are within the order of magnitude of the observation errors, so that, only based on the shape of the curve, it is impossible to take a choice. However, from the values for l thus found, corresponding values for k may be calculated with the aid of fig. 11, next corresponding values for E with the aid of formula (1), and lastly corresponding maximum tensile stresses σ under the centre of the load with the aid of fig.10 and formula (5). The combined results are given in table 1.

Hence it follows from the experimentally determined modulus of elasticity of the concrete slab of 76,000 kg/cm^2, that the values C = 0.3 and L = 4 l are the most probable ones for a loaded area of 75 cm dia. The "revised" tensile stress of 14.1 kg/cm^2, calculated for this load of 21.5 ton, amounts to 66% of the stress of 21.4 kg/cm^2, calculated with the aid of the same values for E, μ and k, but without redistribution of the subgrade reactions.

It should be noted here that these calculations are based on elastic deformations at repeated loading, since it may be expected that the weights of the aircraft will increase gradually and the heaviest types can only be expected in the long run. At first application

TABLE I

Curve	1	2	3	4	5	6	7	8	9	10	11	12
C	0	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.3	0.3
L	-	3 1	4 1	5 1	6 1	3 1	4 1	5 1	6 1	7 1	4 1	5 1
l(cm)	113	107	110	112	115	100	106	112	117	122	102	111
k(kg/cm^3)	1.90	1.90	1.80	1.74	1.66	1.92	1.72	1.55	1.42	1.31	1.62	1.38
E(kg/cm^2)	135,000	109.000	114,000	119,000	127,000	84,000	94,000	106,000	116,000	126,000	76,000	91,000
σ(kg/cm^2)	22.5	17.6	19.7	20.9	21.7	12.2	16.9	19.2	20.7	21.8	14.1	17.7

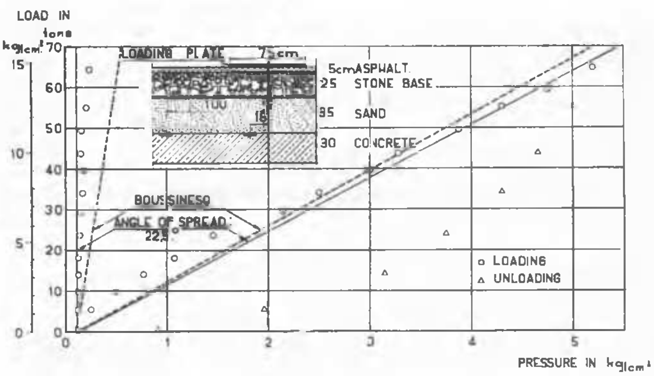

FIG.14

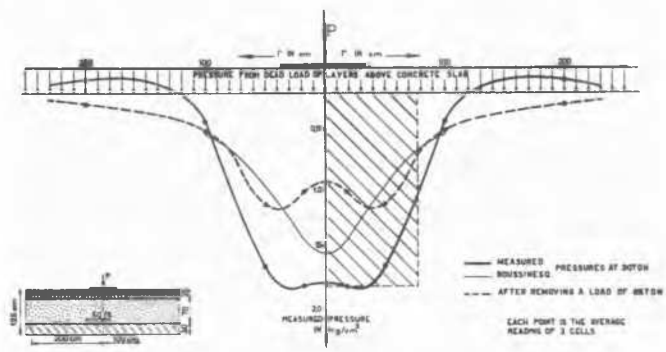

FIG.15

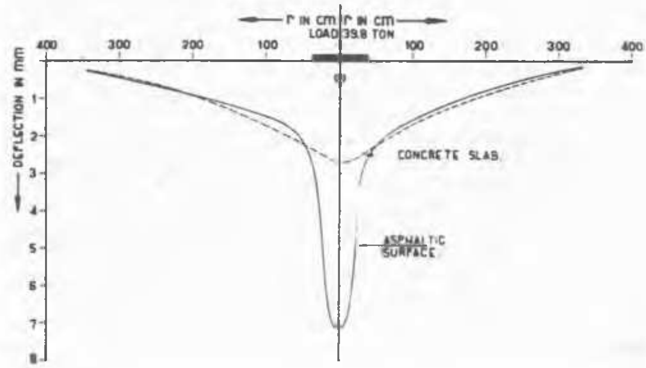

FIG.16

of the load the k-value was smaller and the computed maximum tensile stress was in this case 15.5 kg/cm^2.

FLEXURAL STRENGTH OF THE CONCRETE SLAB.

Since for several concrete slabs the first cracks appeared at a load of 20 to 25 ton, it follows from the above calculations that the flexural strength of this concrete is at least 15 kg/cm^2. It must be emphasized that even with higher loads failure of the slab does not occur, as is evidenced by fig. 12. Although irregularities in the deflection curves and increasing settlements at repetitional loading occurred, a load of 65 ton could be repeated several times without apparent desintegration of the slab, so that even after exceeding the flexural strength an ample margin of safety with regard to complete failure still exists.

The flexural strength has also been determined from bending tests on test beams, sawn out of the concrete slab. Values varying from 15 to 23 kg/cm^2 were found (only for one test beam a value of 11 kg/cm^2). This agrees fairly well with the value found above.

LOADING TESTS ON THE COMBINED RIGID-FLEXIBLE CONSTRUCTION.

The object of these tests is to obtain information on load spreading capacity and bearing power of the layers above the concrete slab, and also to find out whether the measured deflections of the concrete slab check with the theoretical deflection curves.

In a test section with built-in pressure cells and deflection gauges one test loading has been performed to date by building up a load of 64.6 ton in successive stages of about 5 ton on a bearing plate of 75 cm diameter. This test was not used to check the theoretical deflection curve of the concrete slab, since for this purpose the elastic deflections at repetitional loading should have been determined.

Fig. 14 shows the readings of two pressure cells, located at 18 cm and 100 cm from the centre of loading, together with the corresponding theoretical pressures according to Boussinesq at a depth of 65 cm. No appreciable deviations from Boussinesq's theory were found at 18 cm from the centre of loading; however, at unloading no corresponding pressure decreases were measured, probably because of arching in the layers above at the rebounding of the concrete slab. At 100 cm from the centre of loading pressure increase appeared to be only very slight, which points to a stress distribution different from Boussinesq's. This was also indicated by the results of another test on a similar construction, where the thickness of the sand layer had been increased to 70 cm. The pressures in the latter case (fig. 15) could be approximated fairly well by assuming a uniformly distributed load and an angle of spread of 22.5°. Based on these preliminary tests stresses in the concrete slab are given below, first for pressures following from Boussinesq and next for uniformly distributed pressures, the load being spread according to the above mentioned angle. Further measurements will be required to establish the load-spreading properties of the upper layers with greater accuracy under different circumstances.

The fact that the load-spreading is less

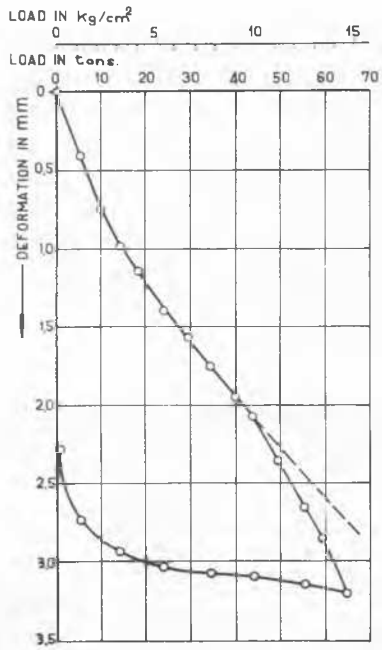

FIG. 17

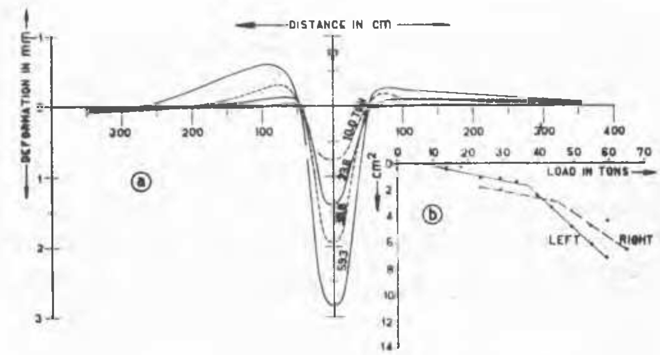

FIG.18

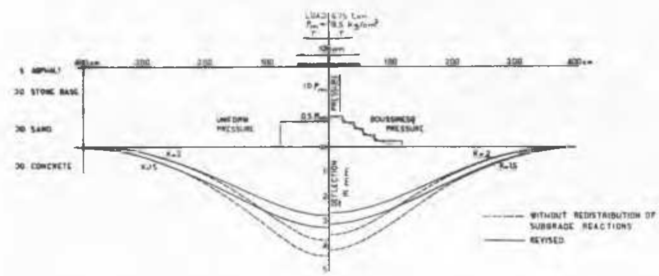

FIG.19

than expected is also supported by fig. 16, which shows the deflection of asphaltic surface and underlying concrete slab for a load of 39.8 ton. Outside the bearing plate the deflection of the concrete slab was more than the deflection of the surface of the construction, because the upper layers, especially stone base and asphaltic toplayer, probably had difficulty in following the curvature of the concrete slab; however, the differences were only slight (for this load max. 0.25 mm).

The deformations of the layers above the concrete slab, without the asphaltic toplayer, occurring at this test are shown in fig. 17 and 18. In fig. 17 the vertical deformations of these layers under the centre of loading are given; these show higher increments for a load of 40 to 45 ton, which were probably caused by plastic deformation. Fig. 18a shows the vertical deformations at increasing distances from the load centre for different loads. The measured areas above the zero-axis in these sections are plotted in fig. 18b. The increasing thickness of these layers may be caused by the influence of the relatively greater deflection of the concrete slab, and also by plastic deformation. However, it seems likely that the higher increment of the areas for resp. 37 ton (left) and 46 ton (right) should be caused by plastic deformation, as the settlement of the concrete slab did not show a sudden increase for these loads. Hence it appears from the measurements on this test section that probably at a load of about 40 ton on the 75 cm dia. bearing plate (contact pressure 9 kg/cm^2) plastic deformation of the upper layers started.

STRENGTH OF THE COMBINED RIGID-FLEXIBLE RUNWAY CONSTRUCTION.

Stresses in the concrete slab have been calculated for single wheel loads of 45 and 67.5 ton applied on top of the construction. A uniformly distributed contact pressure of 8.5 kg/cm^2 has been assumed on circular areas with diameters resp. of 82 cm and 100 cm, and subsequently pressures on the concrete slab have been determined by using the Boussinesq theory (approximation by circular slices) as well as by using an angle of spread of $22\frac{1}{2}^o$ (fig. 19). Calculations have been based on the "revised" formulas of Westergaard, taking the values C = 0,3 and L = 4 1, as found for a loaded area with diameter 75 cm. Further investigations will be necessary to indicate whether these values hold also for different diameters. Thus found stresses in the concrete slab have been collected in table 2, which for the sake of comparison gives also the stresses that would have occurred without a redistribution of subgrade reactions. The influence of this redistribution is apparently large, since it lowers the stresses to 40-50%.

TABLE 2

Wheel load in ton	Modulus of subgrade reaction in kg/cm3	Stresses in concrete slab in kg/cm2 — Boussinesq pressures: "revised"	Boussinesq pressures: without redistribution	Uniform pressures: "revised"	Uniform pressures: without redistribution
45	1.5	12	26	15	29
45	2.0	11	24	13	28
67.5	1.5	15	36	18	39
67.5	2.0	13	33	15	36

Since the load on the concrete slab is best approximated by the uniform pressures, it follows that for a wheelload of 45 ton the found flexural strength of at least 15 kg/cm^2 is not exceeded. For a wheel load of 67.5 ton this is only the case at places with a low modulus of subgrade reaction.

With regard to the strength of the layers above the concrete slab it followed from loading test on the test section that plastic deformation may only be expected at a contact pressure exceeding about 9 kg/cm^2.

Therefore it may be concluded from this first series of tests, that the bearing capacity of the combined rigid-flexible runway construction as applied at Schiphol Airport is sufficient for the requirements, without cracks occurring in the concrete slab and plastic deformations in the layers above. Since stresses in the concrete slab and in the layers above reach their safe limiting values almost simultaneously the construction as applied constitutes a well balanced system.

ACKNOWLEDGMENTS.

The authors wish to thank ir. C. van der Veen of the Delft Laboratory of Soil Mechanics for his assistance in evaluating the test results, also ir. C. van der Poel and M. de Kruyf of the Laboratory of the Bataafse Petroleum Maatschappij for their assistance at the tests.

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SUB-SECTION VIII e

INVESTIGATIONS ON FAILURES, DRAINAGE AND FROST ACTION

VIII e 5

THE CAUSES AND CONTROL OF SUBGRADE MOISTURE CHANGES

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SUMMARY.

In this paper consideration is first given to the fundamental factors which control the movement of moisture in soil and on which a satisfactory subgrade drainage technique must be based. Distinction is drawn between movements occurring under the action of gravity and those resulting from suction or vapour pressure differences in the soil. The influence of moisture content, grading and temperature on the suction and vapour pressure characteristics of soil is considered in detail.

On the basis of this discussion an analysis is made of the principal ways in which water can enter and leave the road subgrade, viz:-
1) through a pervious or cracked road surface
2) by seepage from surrounding high ground
3) as a result of suction differences (a) between the subgrade and the verge, and (b) between the subgrade and the soil beneath
4) as a result of water vapour movements associated with temperature gradients in the road foundation.

Existing and proposed methods for controlling the moisture changes which arise from these sources are reviewed.