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SOIL INVESTIGATIONS AND RUNWAY CONSTRUCTION AT SCHIPHOL AIRPORT

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INTRODUCTION

Schiphol, the principal airport of the Netherlands, is situated at a distance of 8 kilometers from the centre of Amsterdam, the capital. The situation is very favourable, the field itself and its surroundings are completely flat and there are no obstacles of any importance; there is ample room for expansion on a big scale. Now an important junction in the transoceanic, international and national civil air traffic, Schiphol started as a military airfield at the end of the first world war. From a grass covered field of an area of 640,000 m2 (160 acres), it developed gradually into a modern world-renowned airport with buildings, hangars, aprons and runways in four directions. In 1937 and 1938 the runways were constructed and the lay-out shown in fig. 1 was established, comprising a 1000 m runway in the direction of the prevailing winds S.W. to N.E., a 800 m runway perpendicular on this direction and runways W. to E. and N. to S., each 850 m long, all runways being 40 m wide and covering a total area, inclusive of taxiways, of 160,000 m2. Drainage was provided both for runways and the remaining grass areas. The total area of the airfield was at that time 2,100,000 m2 (540 acres).

The second world war brought serious de vastations, all buildings and hangars were rased to the ground, runways and aprons were systematically destroyed. When at the end of 1943 an Allied air attack had put the runways out of use, the Germans took the reconstruction in hand at once, but hardly was it completed when the Allied forces advanced at such a rate, that the Germans in September 1944 exploded 250 charges in runways and aprons, with the result not only that bomb-holes were created, but also that the storm water drainage along the centre of the runways was destroyed over a length of 5 kilometers.

Immediately after the liberation reconstruction was started; after one month the temporary repairs were so far complated on a 1000 m runway, that Dakota's were able to land. By the end of November 1945 the runways could be put into normal use again. The Germans had enlarged the airfield appreciably; the main runway was extended to 1600 m, the runway perpendicular on this one to 1250 m and the E. to W. runway to 1400 m. A large number of taxi-ways and dispersal tracks was constructed, resulting in a total paved runway area of 320,000 m2, while the airfield area was enlarged to 2,800,000 m2 (720 acres), as shown in fig. 2.

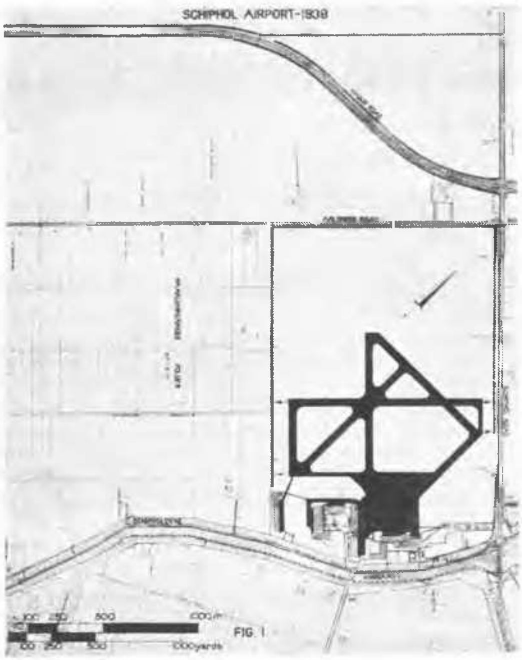
Following the reconstruction of the runways, the extension of the S.W. to N.E. runway, the "instrument-runway", to 2350 m was taken in hand which work was completed in 1947. At the same time a new N.W. to S.E. runway, long 1800 m, was constructed. In 1948 a new N. to S. runway will be made, long 1500 m. These additions are shown in fig. 3. The two last-mentioned runways are designed for the heaviest aircraft that may be expected in the future on the transoceanic airlines according to the classification of the P.I.C.A.O. (maximum gross weight 300,000 lbs; type pressure 120 lbs/sq.in.). Their width is 60 m excluding shoulders on either side, wide 10 m. Both runways are part of the future masterplan of the airport, in which all normal runways ultimately may reach lengths of 2150 m to 2550 m and those in the direction of the prevailing winds of 3000 m (instrument-runways) The plan shown in fig. 3 represents therefore a transition period, during which sufficiently long runways in four directions are available.

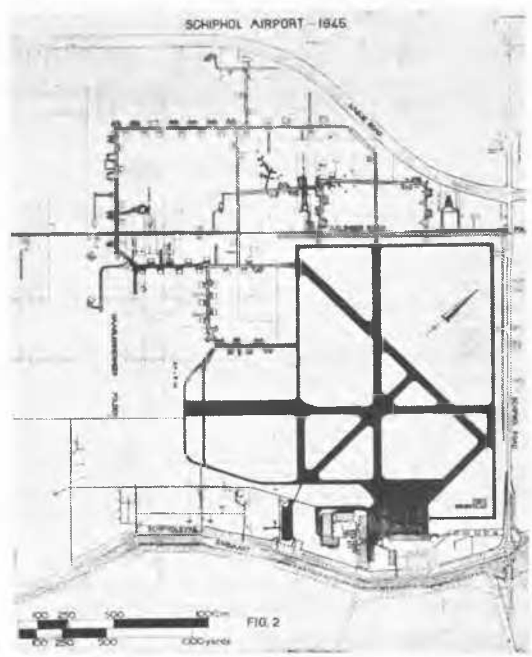
The situation of the Schiphol Airport is unusual in this respect that it lies in a polder, known as the Haarlemmermeer Polder, where the ground surface lies at 4.20 m under the average sea level (designated as N.A.P. in the Netherlands) i.e. at 4,20 m - N.A.P. Originally the present ground surface had been covered by peat layers, but these have been gradually washed away by floods in early times, resulting in a large lake. This lake has been reclaimed about 1850, after constructing an enclosing dike round it together with a canal. At present redundant water from rain and seepage is pumped out by pumping stations into this enclosing canal (waterlevel 0.50 m - N.A.P.), and from this it is evacuated via other canals into the North Sea at ebb tide. The water table of the Haarlemmermeer Polder is at 5.50 m - N.A.P., i.e. about 1.30 m under ground surface. The airport is provided with a drainage system; the drainpipes (6 cm dia.) lie at 0.90 m to 1.20 m under the surface at distances of 7 m and discharge into storm water mains (30 to 120 cm dia.), which carry the water via open trenches to the pumping stations, In order to increase the hydraulic gradient, the watertable in these trenches has been lowered to 6.30 m - N.A.P. and the airport is therefore a separate polder within the Haarlemmermeer Polder; the dewatering is done by means of two pumping stations, situated respectively at the North and the South end of the airport.

SOIL CONDITIONS

Soil conditions have been determined by means of many superficial borings to a depth of 2 m at distances of 200 m, and 12 deep borings to depths between 10 m and 20 m. From these borings a large number of undisturbed and disturbed soil samples were taken, the properties of which have been examined in the laboratory. It appeared that neither the properties nor the elevations of the various soil layers differed very much from place to place, so that it may be said that the airport is constructed on a horizontally homogeneous foundation. To characterize the subsoil three borings are shown in fig. 4 and some typical size distribution curves in fig. 5.

Of the original toplayer of peat only a thin layer is left on a narrow strip alongside the enclosing canal on the S.E. side of the airport. On the remaining area, the toplayer at a few spots still shows a humus content slightly higher than the average. On the whole the following soil profile was found under the existing ground surface:





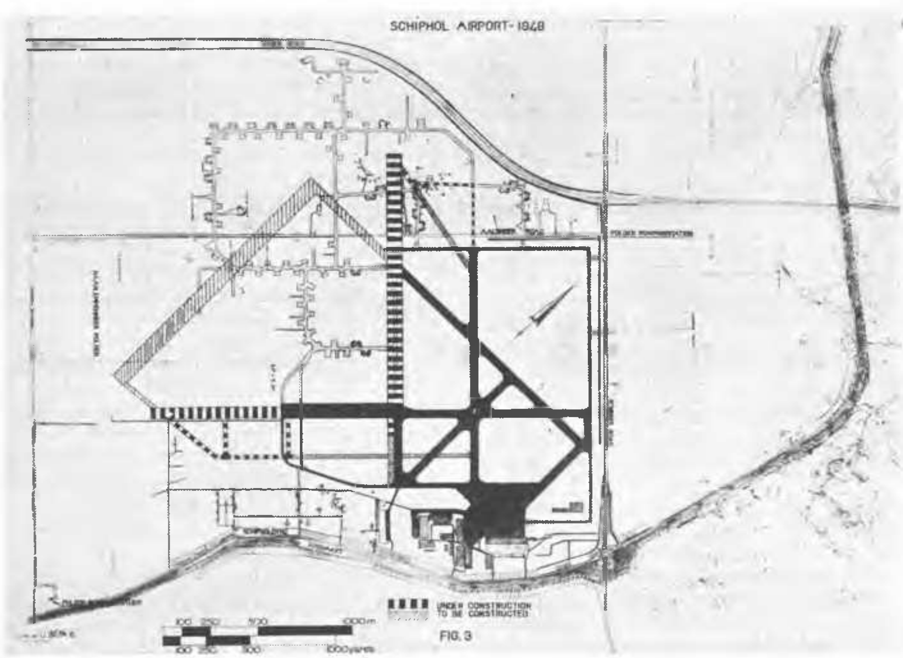


FIG. 1 FIG. 2 FIG. 3

- 0 to 0.5 m: grey clay (e.g. sample 276, fig. 5)
- 0.5 to 1.5 m: brown-grey sandy clay (e.g. sample 278)
- 1.5 to 3.0 m: blue-grey clayey sand (e.g. sample 282)
- 3.0 to 4.5 m: blue-grey sandy clay (e.g. sample 283)
- 4.5 to 6.5 m: blue-grey clay (e.g. sample 284)
- 6.5 to 7.0 m: hard peat. Below this fine sand occurs (e.g. sample 289), which grows coarser with increasing depth (e.g. sample 242) and contains locally some gravel (e.g. sample 244). The piezometric rise of the water in these sandlayers is to 4.40 m - N.A.P., causing an average seepage of 0.5 mm per 24 hours.

Table 1 gives some characteristic properties of the undisturbed samples from borings C8, E8 and G2. The angle of internal friction and the cohesion are determined by quick loading in the cell apparatus. According to the soil classification for airfields by A. Casagrande, the sandy clays and the clayey sands belong to the groups ML, CL and OL, and the clays to the groups CH and OH.

In order to determine the homogeneity of the foundation soil and the bearing capacity of the soil layers at different depths, soundings have been performed at all places where

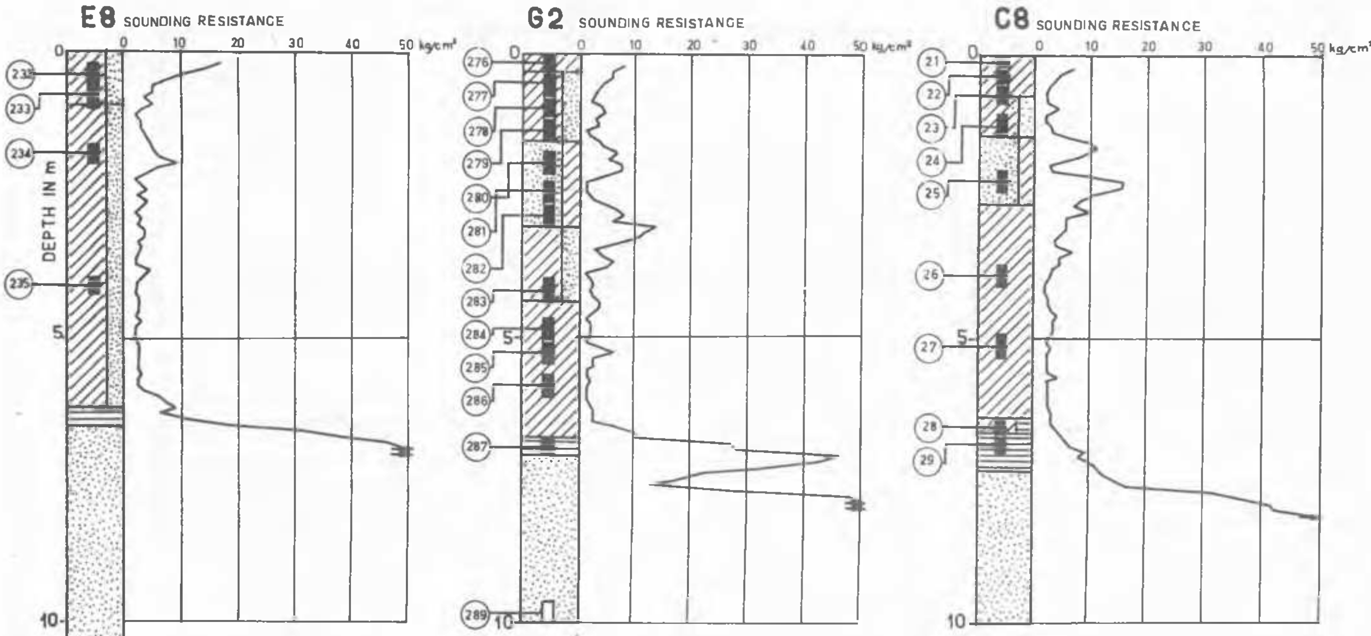
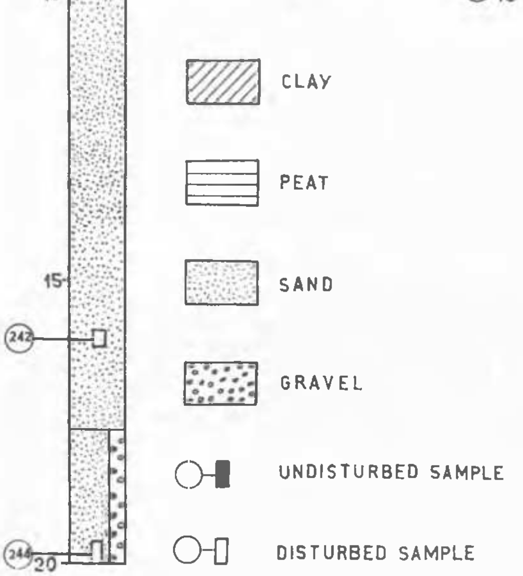



FIG.4

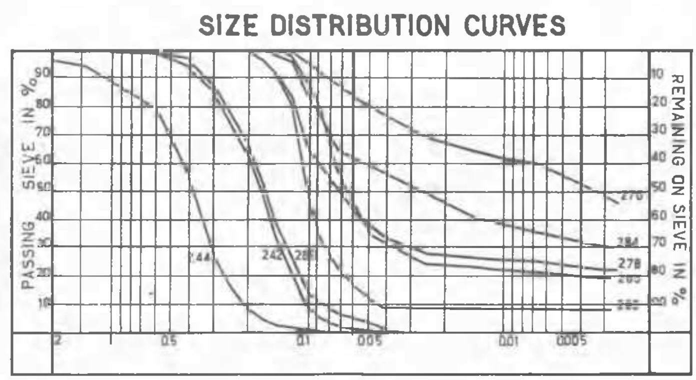


FIG.5

TABLE 1

SAMPLE NO.	DEPTH IN m	VOIDS IN %	DENSITY IN g/cm³ IN SITU	DENSITY IN g/cm³ DRY	MOISTURE CONTENT IN % DRY WEIGHT IN SITU	MOISTURE CONTENT IN % DRY WEIGHT LIQUID LIMIT	PLASTICITY INDEX	ANGLE OF FRICTION	COHESION IN kg/cm²
21	0.0-0.2	49.8	1.77	1.33	33.0	49.4	20.4		
22	0.2-0.5	55.9	1.70	1.17	45.5	79.8	41.2		
23	0.5-0.8	47.6	1.83	1.39$_5$	31.8	53.0	28.0		
24	1.0-1.4	51.5	1.78	1.28^5	38.6			30	0.02
25	2.0-2.4	43.3	1.92	1.50	27.9			35	0.08
26	3.6-4.0	49.0	1.86	1.35	37.8	48.6	26.4	13	0.13
27	4.9-5.3	65.0	1.55	0.93	67.0	64.2	37.9	12	0.16
28	6.3-6.6		1.13	0.35	224.0				
29	6.6-6.9		1.02	0.22	357.0				
232	0.2-0.5	47.2	1.88	1.40	34.2			19	0.06
233	0.5-0.9	45.3	1.89	1.45	30.5	37.2	11.5	21	0.03
234	1.6-1.9	49.2	1.90	1.34	41.2	37.9	16.9	19	0.05
235	3.9-4.2	49.8	1.80	1.33	35.1	41.8	17.5	7	0.06

SAMPLE NO.	DEPTH IN m	VOIDS IN %	DENSITY IN g/cm³ IN SITU	DENSITY IN g/cm³ DRY	MOISTURE CONTENT IN % DRY WEIGHT IN SITU	MOISTURE CONTENT IN % DRY WEIGHT LIQUID LIMIT	PLASTICITY INDEX	ANGLE OF FRICTION	COHESION IN kg/cm²
276	0.0-0.3	53.8	1.72	1.22	40.7	80.4	34.7		
277	0.3-0.7	45.2	1.88	1.45	29.6	36.1	8.1	27	0.11
278	0.7-1.1	49.3	1.78	1.34$_5$	32.6	38.8	19.3	27	0.12
279	1.1-1.5	44.4	1.91	1.47^5	29.4	46.4	18.9	27	0.05
280	1.7-2.1	44.6	1.90	1.47	29.4			34	0.03
281	2.2-2.6	42.0	1.94	1.54	26.2	33.1	11.4	28	0.03
282	2.6-3.0	42.7	1.92	1.52	26.6			34	0.06
283	3.9-4.3	52.8	1.79	1.25$_5$	43.0	41.6	16.1	23	0.10
284	4.6-5.0	69.6	1.46	0.80^5	81.6	70.2	17.5	14	0.24
285	5.0-5.4	67.4	1.52	0.86	76.0	68.0	12.7		
286	5.6-6.0	71.3	1.44	0.76	89.5	118.8	41.4	11	0.26
287	6.7-7.0		1.03	0.29	350.0				

borings have been taken. At these soundings the penetration or sounding resistance of a cone with a basis area of 10 cm2 is measured at increasing depths under the ground surface 1). Fig. 4 shows some examples of the resulting resistance-diagrams. The range of the apparatus does not go further than 50 kg/cm2. It appears that in general the resistance in the layers under the groundwater table down to 4.50 m - ground level varies from 3 to 5 kg/cm2 with local higher values in the sandy layers. In the clay layers below mostly not more than 2 kg/cm2 was found, whereas the resistance in the peat layer increases again and in the under-

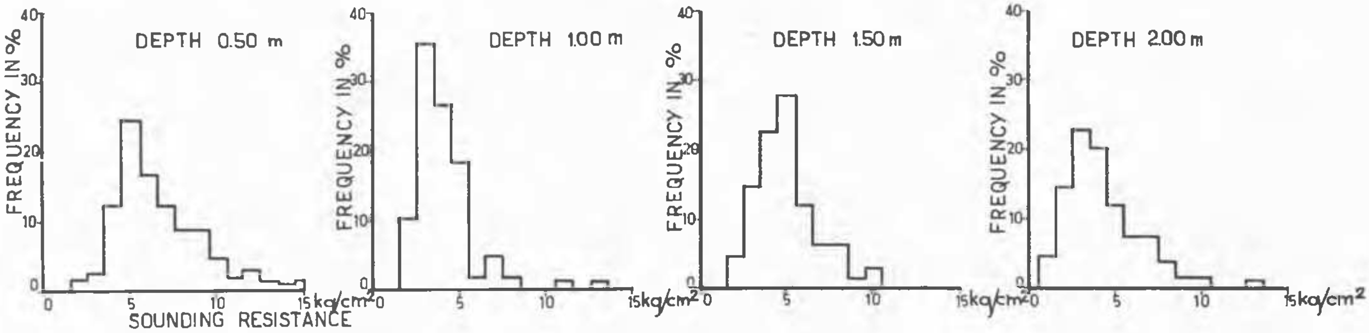


FIG. 6

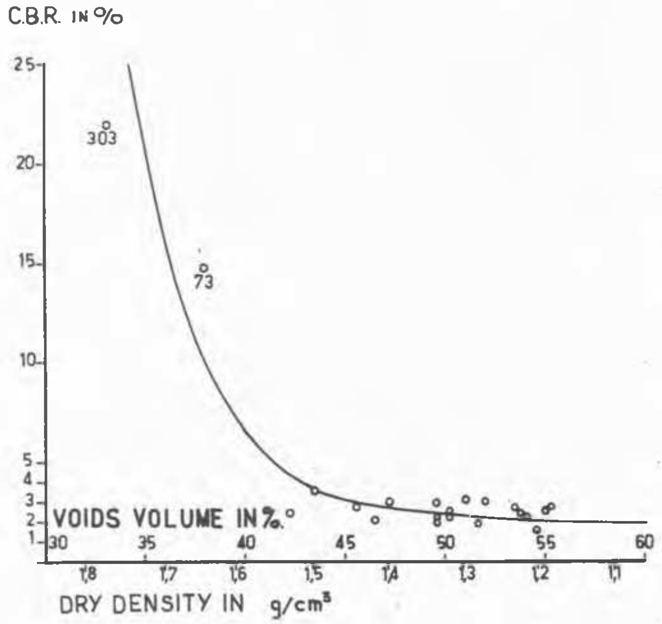


FIG.7

lying sand layers it rises rapidly to more than 50 kg/cm2. Fig. 6 shows the frequency of the resistances in the toplayers, which are essential for the construction of the runways. At a depth of 1 m an average of about 4 kg/cm² was found.

The California Bearing Ratio has been determined of undisturbed soil samples taken from the layers at 1 m below ground surface, at which depth the foundation of the runways was originally designed. In fig. 7 these C.B.R.-values have been plotted against the voids volume and the dry density. From investigations at some twenty airfields in the Netherlands a relation between C.B.R. and voids volume appeared, being valid for sand as well as for clay. This curve is also shown in fig. 7. The soil of Schiphol proves to conform well to this relation. Samples 73 and 303 are compacted in the laboratory according to the "modified A.A.S.H.O.-compaction test" (fig. 8). Whereas for undisturbed samples the C.B.R. had an average value of 2.5% (natural moisture content), for these two compacted samples values were found after soaking of resp. 15 and 22% at 85% and 88% of optimum density.

Moreover, field bearing tests have been carried out to determine the modulus of subgrade reaction k. This modulus has been calculated for deflections of 1.25 mm, obtained with a rigid bearing plate of 75 cm diameter. The k-values for the undisturbed foundation soil at 1 m below ground surface vary between 2.0 and

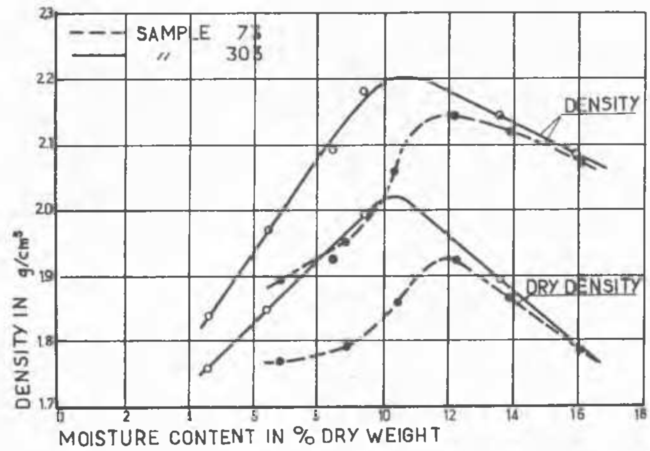


FIG. 8

3.0 kg/cm3, except for the areas with a high humus content, where a value of 1.3 kg/cm3 was found. The modulus of subgrade reaction is also calculated from the results of quick compression tests with repeated loads on undisturbed soil samples in the laboratory; the values thus found conformed very well to the values obtained from field tests.

In view of the favourable results of the compaction of the sandy clay in the laboratory, it was decided to investigate also the possibility of compacting the subgrade in the field. A difficulty however is that the optimum moisture content lies at 10-13%, whereas the average natural moisture content of the toplayers in 35%. For these compaction tests soil from the excavation for a runway was used; the moisture content of the soil stacks was at the surface about 17% during dry periods and 27% during rainy periods. During dry warm weather in a test section excavated to 60 cm below ground surface, first three layers of 10 cm thickness each (average moisture content 21.5%) were compacted with smooth rollers and subsequently four layers of 7.5 cm thickness each (average moisture content 17%) were compacted with empty sheepfootrollers of light standard type. The density increased after 2, 4, 8 and 16 coverages from 77% to 82% of the optimum at modified A.A.S.H.O. compaction for the same soil and proved to be almost equal in the various layers. For the C.B.R. of the toplayers after 16 coverages values of 8% were found with the moisture content as present and of 5.8% after soaking, while the modulus of subgrade reaction was found to be 4.3 kg/cm3. Though these results are not unfavourable, the application of subgrade compaction at the actual construction of the runways was made impracticable by the unstable Dutch climate and the high

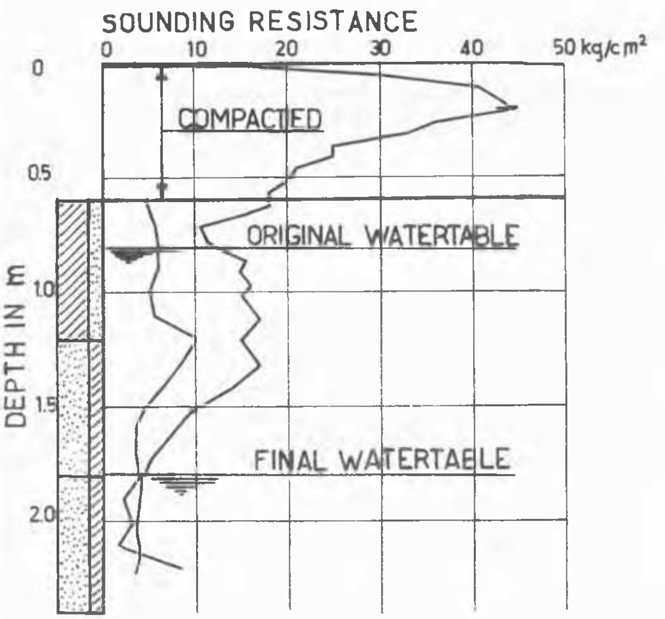


FIG. 9

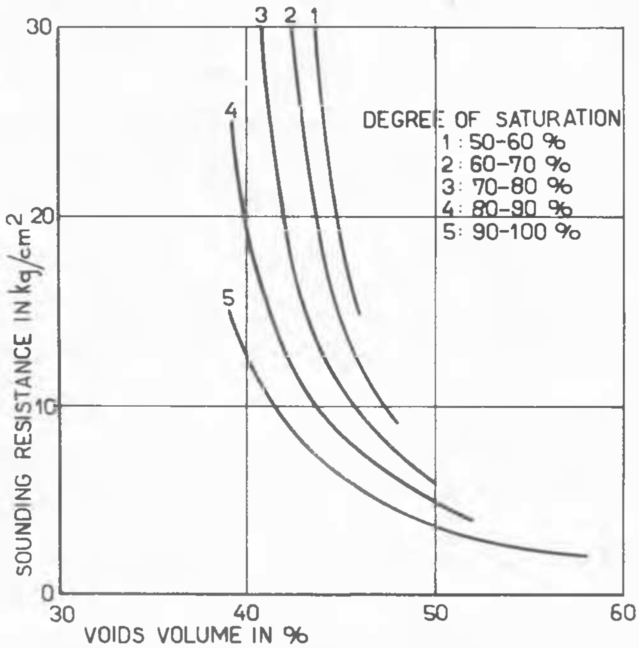


FIG.10

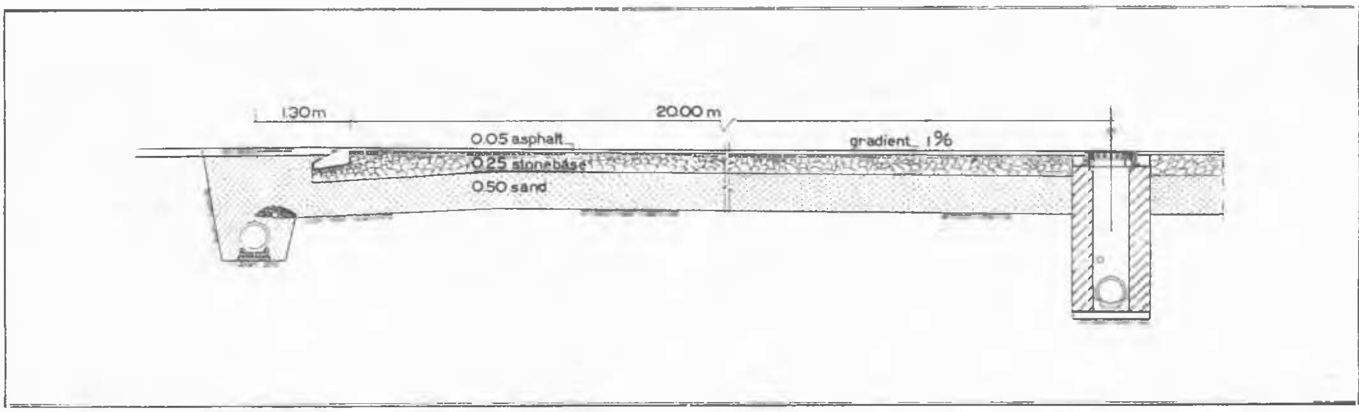


FIG.11

normal moisture content of the soil.

Fig. 9 shows the sounding resistance in the test section before and after the tests. However, during the period of 3 weeks between the two observations the local groundwater table had gone down about 1 m, and this can be clearly seen in the measured resistances in the subgrade above groundwater level. The resistances in the compacted layers also depend largely on the moisture content, From the information obtained it is attempted to find a relation between sounding resistance and voids volume at various degrees of saturation for the sandy clays of Schiphol (fig. 10). From this relation for almost saturated soils and from the C.B.R.-voids volume curve (fig. 7) it is possible to derive a relation between sounding resistance and C.B.R. for these soils at Schiphol.

RUNWAYS WITH FLEXIBLE PAVEMENT

The runways constructed in 1937 and 1938 had a flexible pavement. After the necessary excavation, a 50 cm thick sandlayer was applied on which a rolled stone base (25 cm), covered with 5 cm asphaltic concrete with a surface treatment. as shown in fig. 11. The gradient was 1% to the centre, where a drain channel of mastic asphalt is provided with catch-pits at distances of 25 m. At that time a gradient to the centre was preferred for reasons of flying technique; in practice however, this caused the disadvantage that in winter the catch-pits became blocked by snow and ice, resulting in difficulties when the thaw set in. It appeared moreover afterwards that the alternating longitudinal slopes of 1 : 250 along the centre of the runway, caused troublesome vertical movements in aircraft with a nose wheel. For the rest this construction has stood up very well, and the stretches that have not been purposely destroyed by bombs, are still in good condition under an intensive traffic with aircraft of the types DC 3, DC 4, Skymaster and Constellation.

The Germans, when extending the runways, at first applied the same construction, but for some parts and for the taxiways lack of time obliged them to make a lighter construction. Fortunately, the mainpart of the taxiways will not be used in the long run, and the rest is in process of being reconstructed on a heavier scale.

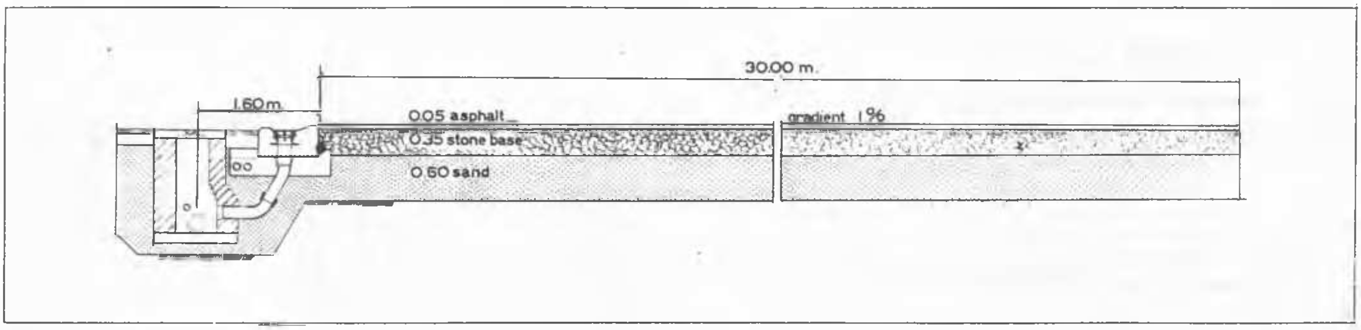


FIG.12

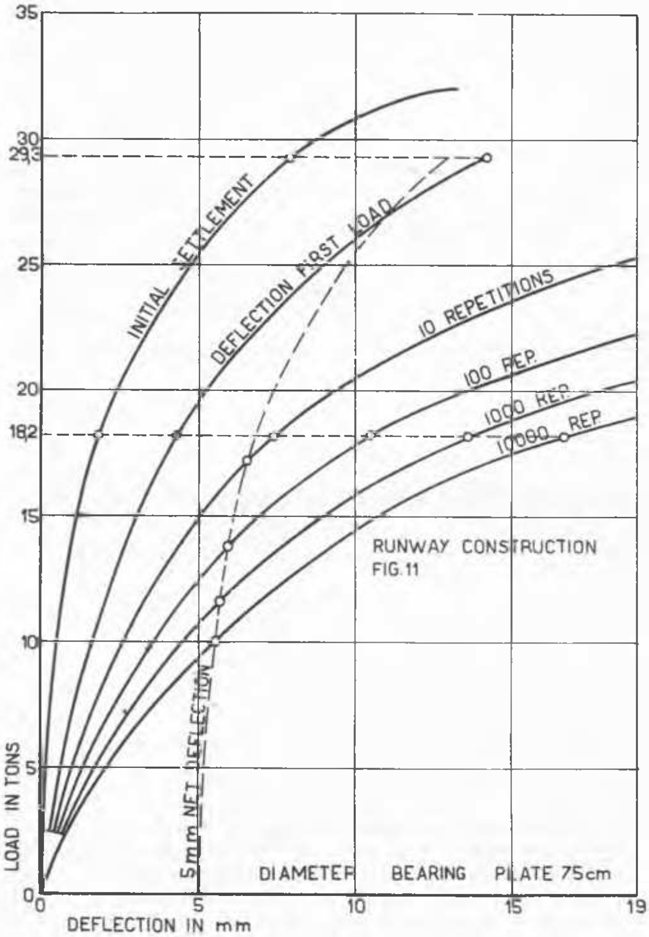


FIG.13

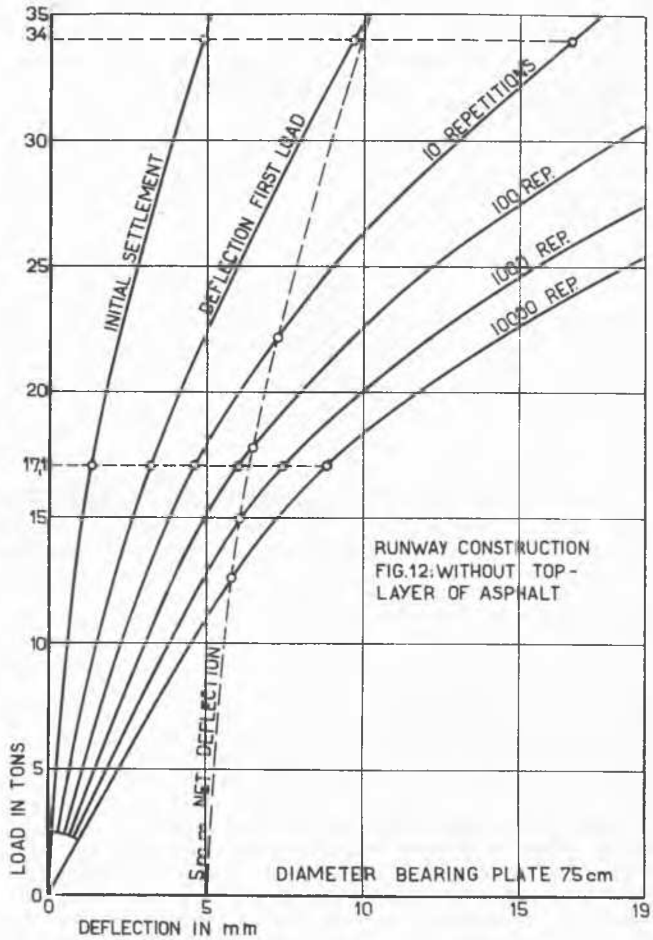


FIG. 14

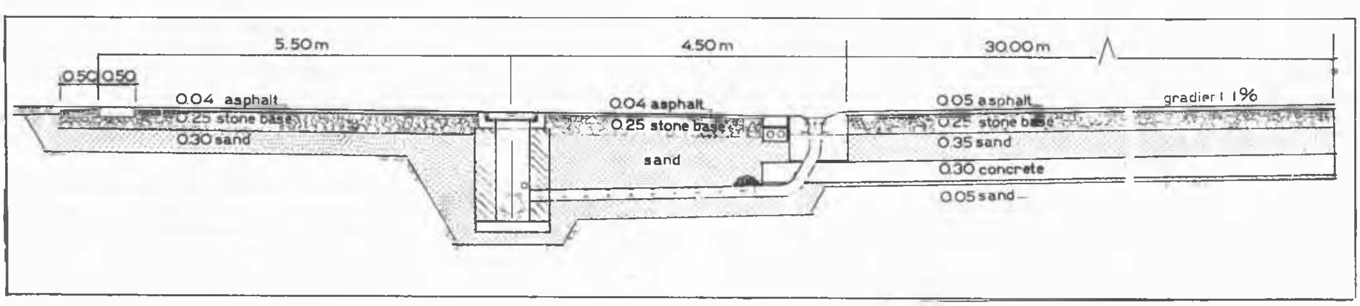


FIG.15

The extension of the S.W. to N.E. runway had to be designed before the present recommandations of the P.I.C.A.O. for class A airefields were known. Based on the favourable experience with the runway construction of 1937 and 1938, the thickness of the sand subbase was increased to 60 cm, the thickness of the rolled stone base to 35 cm, with the same 5 cm asphaltic concrete (fig. 12). The surface was drained with a gradient of 1% from the crown via open concrete edge drains to pipe drains alongside the runways.

The bearing capacity of those runways has been determined by means of the California Bearing Ratio method, based on an average C.B.R. of 2.5% for the subgrade. The safe single wheel loads at "capacity" and "limited" operation are given in table 2, both for runways and for taxiways. According to the specifications of the U.S. Engineering Department 2) the thickness resulting from the design curves may be reduced with 20% at limited operation, while for taxiways the safe loads are 20% lower.

Moreover, load tests have been made on the runways by the method, recommended by the Committee on Flexible Pavement Design of the Highway Research Board. 3)
Thereby each consecutive load is at least four times applied and removed, while at these repetitions the increments of deflections and settlements are assumed to be proportional to the logarithm of the number of repetitions, so that from these observations the deflections may be calculated for any desired number of repetitions (fig. 13 and 14). It is recommended to allow a deflection of 5 mm (0.2") over and above the settlement after the first application of each load, which settlement is mostly attributed to initial compaction. At these tests the load is applied by means of a specially constructed loading platfrom on wheels 5) via a rigid circular bearing plate (dia. 75 cm) under which a 2,5 om thick rubber slab is laid in order to obtain uniform load distribution.

Table 2 gives the resulting safe loads at 5 mm net deflection and the corresponding contact pressures at these tests at 10, 100, 1000 and 10000 repetitions, together with the by means of Boussinesq's formulas calculated maximum vertical stresses and shearing stresses (Poisson's ratio = 0) in the subgrade. These safe loads are subsequently converted for contact pressures of 5 kg/cm2, keeping the same subgrade stresses as previously found. The same method has been applied for the calculation of the safe load for dual wheels at distances centre to centre of 75 cm, which method is justified by the results of tests in the U.S.A. 4) Account is taken of the fact that at the tests of fig. 14 the 5 cm asphaltic toplayer had not been applied yet.

From these tests it follows that the results of both methods agree very well, and it appears that, for conditions as obtain at Schiphol, limited operation on runways and taxiways according to the C.B.R. method would correspond to respectively 10 and 100 repetitions of static loads on the same identical area, and capacity operation on runways and taxiways to resp. 1000 and 10000 repetitions. In this context it is to be noted that in these tests the safe load at 10 repetitions was equal to half of the load, which caused loss of equilibrium of the whole construction at first application.

The bearing capacity of the runway construction of fig. 11 is therefore sufficient for limited operation with aircraft of a maximum gross weight of respectively 35 and 52 tons for single dual wheels (10% of the weight being carried by nose wheel or tail wheel).
The bearing capacity of the runway construction of fig. 12 is sufficient for limited operation with aircraft of a maximum gross weight of respectively 55 and 68 tons for single and dual wheels.

NEW RUNWAYS WITH A COMBINED RIGID-FLEXIBLE PAVEMENT

In the meantime the conference of the P.I.C.A.O. in 1945 at Montreal had recommended, that airfields meeting the highest requirements (class A) had to be designed for "capacity" operation with aircraft with single wheelloads of 68 tons (150.000 lbs), this being reduced in 1946 to dual wheelloads of 68 tons. Based on a C.B.R. of 2.5% for the subgrade, the required combined thickness of pavement and base for a flexible construction would be resp. 195 cm and 180 cm. Since subgrade compaction is practically excluded, a construction of such considerable thickness would implicate the use of larger quantities of sand and broken stone. Moreover, excavation of the soil to a depth below groundwater level would still further increase the construction costs. Consequently the problem was to find a construction of restricted thickness, still capable to spread the load of the heaviest aircraft in such a way that the deformations of the subgrade would be limited to allowable values.

It was then that the first mentioned author of this article conceived the idea of a construction combining the advantages of both the flexible and the rigid pavement, and which as far as known, has not been applied before, This idea was based on the following considerations. The advantage of the rigid pavement lies in its great stress distributing capacity, owing to the ability of concrete to withstand tensile stresses. The drawback of concrete, directly exposed to varying weather conditions, lies in the fact that it is subjected to temperature fluctuations. Apart from necessita-

TABLE 2

number of repetitions	load in tons at 5 mm net deflection	contact-pressure in kg/cm2	max.subgrade stresses in kg/cm2		load in tons at 5 kg/cm2 contactpressure		C.B.R. = 2,5% max.single wheelload in tons	
			vertical pressure	shearing stress	single wheels	dual wheels		
RUNWAY CONSTRUCTION FIG.11. TOTAL THICKNESS PAVEMENT AND BASE 80 CM.								
10	17,2	3,87	1,00	0,54	16,1	23,4	15	limited operation runways
100	13,8	3,11	0,80	0,44	12,4	18,7	12	" " taxiways
1000	11,5	2,60	0,67	0,37	10,1	15,6	9	capacity operation runways
10000	10,0	2,26	0,58	0,32	8,6	13,6	7,3	" " taxiways
RUNWAY CONSTRUCTION FIG.12. TOTAL THICKNESS PAVEMENT AND BASE 100 CM.								
10	22,2	5,03	0,98	0,53	24,9	30,8	25	limited operation runways
100	17,8	4,01	0,78	0,43	18,8	23,9	20	" " taxiways
1000	15,0	3,39	0,66	0,36	15,6	20,0	15	capacity operation runways
10000	12,5	2,83	0,55	0,30	12,7	16,6	12	" " taxiways

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-