# CORRESPONDENCE
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Airport Paper No. 26

" Some Considerations of Airfield Pavement Design " †

by

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### Correspondence

**Mr G. M. J. Williams** observed that when discussing the methods adopted in America for designing pavements the Authors stated in the Paper that " the whole American approach at present is aiming at a much greater standardization than is used in Great Britain—for instance, tire pressures have been standardized by adopting 100 and 200 lb. per square inch, whilst for weight distribution the wheel spacings of the Boeing undercarriage as used on the B.29 and the Stratocruiser have been adopted for the dual wheels, and the B.36 for the four-wheelers."

That implied that the C.B.R. and the Westergaard methods, as used, were restricted to certain arbitrarily chosen wheel spacings when applied to the design of pavements to carry multiple-wheel undercarriages. It was to be regretted that the Authors had not mentioned the development of the C.B.R. method described by Boyd and Foster,[18] and of the Westergaard method described by Pickett and Ray.[19]

The method of Boyd and Foster enabled the thickness of a flexible pavement to be determined directly for any arrangement of wheels in the undercarriage, and was based on a plotting of the C.B.R. curves on a double logarithmic base (see Fig. 11). A point A was then determined by the load of one wheel and a depth equal to half the clear space between the closest wheels, and a point B was determined by the total weight of the undercarriage and twice the spacing between the most distant wheels.

† Proc. Instn Civ. Engrs, Pt II, vol. 4, p. 55 (Feb. 1955).

[18] W. K. Boyd and C. R. Foster, " Design Curves for Very Heavy Multiple Wheel Assemblies." Trans Amer. Soc. Civ. Engrs, vol. 115 (1950), p. 534.

[19] Pickett and Ray, " Influence Charts for Concrete Pavements." Proc. Amer. Soc. Civ. Engrs, separate No. 12, May 1950.

Those two points were joined by a straight line whose intersection with the C.B.R. lines gave the required C.B.R. at any depth. Furthermore, for any existing pavement a line CD could be drawn giving the C.B.R. at any depth below the surface. The line CD shown represented a pavement with a base extending down to 16 inches below its surface on a sub-base 14 inches thick whose minimum C.B.R. was 15, resting in turn on a subgrade whose minimum C.B.R. was 8. It was then a simple matter to determine if the pavement would carry any particular aircraft by whether or not the line AB corresponding to that aircraft was wholly to the right of line CD.

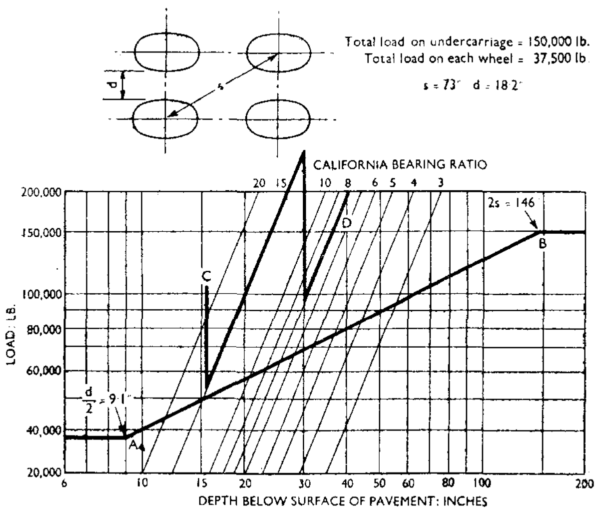


Fig. 11.—C.B.R. METHOD USED TO DETERMINE THICKNESS OF FLEXIBLE PAVEMENT NECESSARY TO SUPPORT A MULTIPLE-WHEEL UNDERCARRIAGE

As compared with that, the method of design described in the Paper was indirect and lengthy because it appeared to involve the following three steps :—

(1) The required pavement thickness must first be guessed and then used to find the equivalent single-wheel load from Fig. 5 of the Paper by Cooper.[13]

(2) The load classification number must then be found from Fig. 2 (see p. 67).

(3) The pavement thickness was then found from Fig. 9 of that Paper.

The thickness calculated must then be checked against the figures guessed in (1) above and if those did not agree the process had to be repeated until agreement was obtained. The method gave similar results to that described by Boyd and Foster.[18] It was true that Boyd and Foster's curves had been published only for tire pressures of 100 and 200 lb. per square inch, but for most undercarriage layouts a variation of tire pressure throughout the whole range would only result in a difference of thickness between 1 inch and 2 inch in any layer of the pavement, so that it was quite simple to interpolate the thickness for any required tire pressure to within $\frac{1}{2}$ inch which was sufficiently accurate for all practical purposes.

Pickett and Ray's method was not a direct method and probably took rather longer than the method described by the Authors for the design of concrete pavements. Both methods seemed to give very similar results.

It was unfortunate that both the Air Ministry and the International Civil Aviation Organization used the term "equivalent single-wheel load" when something completely different was meant by it, as would be seen by comparing the definition in Annex No. 14 to the Convention on International Civil Aviation with that in Cooper's Paper.[13]

The Authors made an important point in discussing the difficulty of allowing for softening of the subgrade and sub-base after the pavement had been constructed, and it was possible that the next significant advance in the design of pavements would lie in the development of practical methods of estimating the worst condition which a subgrade or sub-base was likely to reach. The theoretical basis for doing that had already been laid by the Road Research Laboratory and it remained for practising engineers to develop the practical application of those methods.

**Mr L. W. Brown** stated that he was glad that the Authors had stressed the importance of the economics of airfield pavement construction in the choice of design but he did not think that all engineers would unreservedly accept the second paragraph on p. 56 without some proof being offered. The idea that one particular form of construction was always more suitable for a particular type of soil would die hard and it would probably need to be demonstrated that any form of construction could be used on any type of soil and that the choice was usually determined by availability and price of materials.

Once it was agreed that the cost of airfield pavements was a major factor in deciding the type of construction, a further proposition followed as a corollary—that everything which contributed to the cost should be given consideration in proportion to its contribution.

The most important single factor in comparing the cost of two different types of construction, for instance, flexible and rigid, was the price of aggregates and base materials. Mr Brown was of the opinion that a third proposition followed logically from the first two, that it was a waste of money to spend much time and trouble on a thorough soil survey without spending an equal amount of time and trouble on a thorough survey of local aggregate resources. In that connexion he looked forward to the time when further research suggested on p. 91 would enable the cost of using a local gravel aggregate for concrete to be compared with the cost of going farther afield and paying a few more shillings per ton for crushed stone aggregate in order to take advantage of the increased flexural strength to save an inch or two in the depth of the slab.

He submitted an alternative presentation of the Teller and Sutherland

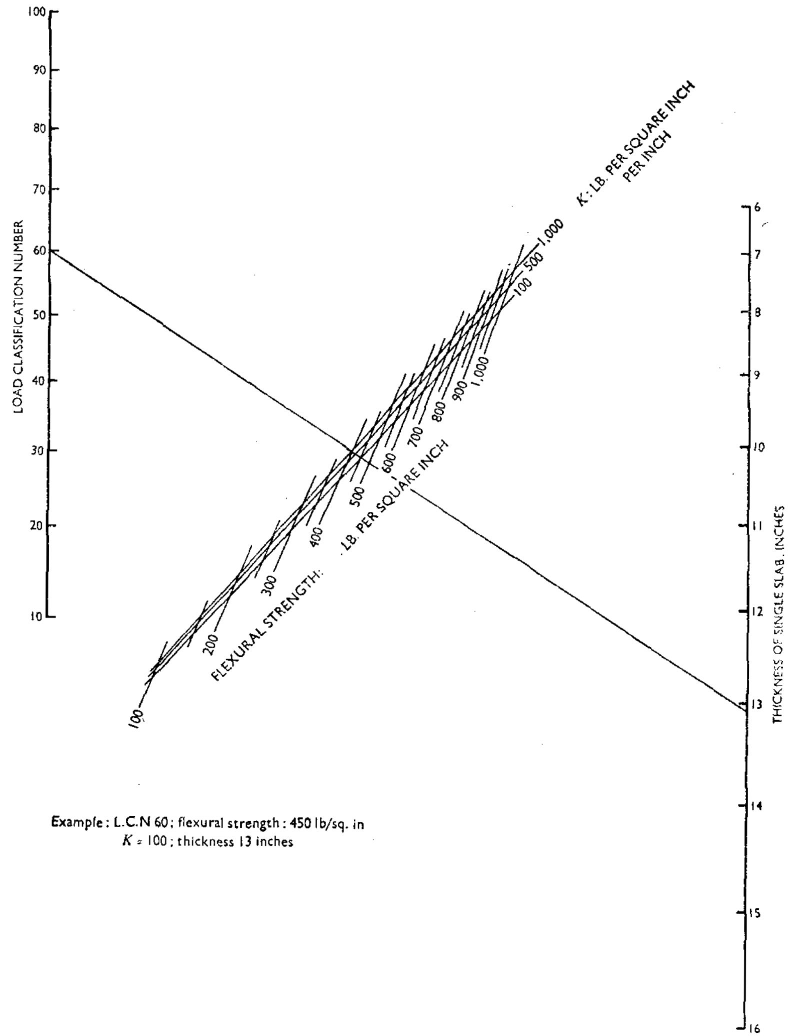


FIG. 12.—DESIGN OF SINGLE-SLAB CONCRETE RIGID PAVEMENTS

formula in the form of a nomogram (Fig. 12), which he thought was a little easier to read than Fig. 8a. It was a pity that the method of calculating Fig. 8b for double-slab concrete construction had not been given in the Paper since it should not be difficult to construct a further nomogram which varied the flexural strength and perhaps also the thickness of the upper as well as the lower slab.

When making the necessary calculations for constructing the nomogram he had noticed that the calculated wheel loads corresponding to L.C.Ns between 10 and 70 did not agree with values read from Fig. 2, assuming on the basis of Fig. 1 that those values should lie between the tentative extensions of the L.C.N. curves for flexible and rigid pavements.

The relevant values were as follows :—

| L.C.N. | 10 | 20 | 30 | 40 | 50 | 60 | 70 |
|---|---|---|---|---|---|---|---|
| From Fig. 1 . . . | 10,500 | 15,500 | 20,000 | 24,500 | 28,000 | 32,500 | 36,500 |
| Calculated . . . | 9,310 | 14,110 | 18,190 | 21,910 | 25,430 | 28,800 | 32,080 |

In view of the small effect of $k$-value on slab thickness it was interesting to re-read the assertions made by the Authors of, and contributors to the discussion on, Airport Paper No. 1,[20] regarding the importance of obtaining a subgrade of high bearing value. He wondered whether the considerable sums which had been paid in the past for excavating clay and replacing it with gravel could not have been considerably reduced by the use of a 4-inch dry lean concrete working course or another inch or two of concrete paving.

He wished to say a few words in favour of deep single-slab rigid construction which appeared to have had rather a raw deal in Britain, although it was, he believed, more popular in America.

The advantages of double-slab construction mentioned at the top of p. 82 and repeated on p. 91 should result in the total depth of double-slab construction being less than that of single slab for equal bearing values. That result was not upheld by comparison of Figs 8a and 8b from which total depths of concrete of flexural strength 450 on a subgrade of $k$-value 300 were as follows :—

| L.C.N. | 30 | 40 | 50 | 60 | 70 | 80 | 90 |
|---|---|---|---|---|---|---|---|
| Single . . . . | 10″ | 11″ | 12″ | 13″ | 13½″ | 14″ | 15″ |
| Double . . . | 11½″ | 12″ | 13″ | 14″ | 14½″ | 15″ | 15½″ |

[20] George Graham and F. R. Martin, "Heathrow: The Construction of High-Grade Quality Concrete Paving for Modern Transport Aircraft." J. Instn Civ. Engrs, vol. 26, p. 117 (April 1946).

The advantages of single slab-construction were:— (1) saving of cost of separation membrane and of surface finish and joints in the lower slab; (2) saving of from $\frac{1}{2}$ to $1\frac{1}{2}$ inch of concrete, if the method of calculating the depth of double-slab construction incorporated in Fig. 8b could be relied upon; (3) fewer joints with consequent saving of expensive sealing compound and maintenance costs, since 8-inch slabs were divided into 10-foot-by-10-foot bays whilst slabs exceeding 12 inches thick only required 20-foot-by-20-foot bays.

The only serious disadvantage of deep concrete pavements which was usually stressed was the difficulty of obtaining thorough compaction for the full depth. In that connexion Mr Brown wished to differ from the Authors by submitting that full compaction at the bottom of the slab was not necessary.

It was explained on p. 79 that the method of designing single-slab pavements took account of the weakest condition, namely, the corners when in a state of temperature stress. Now, when the temperature gradient was such that the corners curled downwards so that the slab tended to rise in the centre and support itself on its four points the single slab supported on a relatively weak foundation (even a dry lean concrete working course) would probably depress the subgrade to an extent which would ensure support over an adequate area. On the other hand, where the corners curled upwards and acted as a cantilever, at first unsupported and later, when under load, supported by the subgrade, the top would always be in tension and the bottom in compression. But it was the top of the slab which above all should be fully compacted and was therefore capable of attaining its full flexural strength. Since that was only about 10 per cent of the compressive strength he maintained that however badly the bottom of the slab was compacted, within reason of course, it would always develop sufficient compressive strength to balance the flexural strength in the top. It might sound revolutionary to speak of deliberately laying concrete which was not fully compacted but it was only extending to pavement construction the principle of using a material to the best advantage and not putting a lot of unnecessary strength where it was not required.

On p. 83 he noted the flexural strength adopted for design purposes was that which should be attained at 6 months. That was probably satisfactory for a big contract for a complete new runway with taxiways and standings but there must be many contracts for small extensions, a single aircraft-servicing platform, etc., which would have to carry a full load within a month of the laying of the last bay. He would like to know whether any account was taken of that when designing the smaller jobs.

Finally, he wished to say a few words on separation membranes between the upper and lower layers of double-slab pavements. The Air Ministry in its latest specification had modified the earlier requirement for a canvas belt finish everywhere and had asked for a slightly roughened surface in order to prevent skidding on concrete runways and taxiways where aircraft travelled at high speeds. That implied that the canvas belt finish which was still required to facilitate drainage where fuel was liable to be spilt could be made very smooth indeed. It appeared, therefore, that it was necessary only to ask for the smoothest possible finish on the lower slab, which might be obtained at very little expense, together with a thin coat of bitumen emulsion to prevent chemical adherence of the two slabs. If the contractor had failed to produce the required degree of smoothness he should be ordered to cut out the faulty concrete and re-lay it, to grind the surface smooth, or to lay building paper at no additional expense. To specify building paper in all circumstances was merely condoning bad workmanship.

**Mr Alfred Goode** expressed full agreement with the Authors in their statement on p. 58 that "the most acceptable design method must be almost wholly based on practical evidence," and on p. 63 that "the best method of testing soil strength must be one primarily related to in situ as opposed to laboratory conditions." Although the Paper was entitled "Some Considerations, etc.," Mr Goode thought that the Authors would have proceeded, in view of the above quoted remarks, to recommend the field testing of a design, especially since, in his opinion, (*a*) designs were in all cases based on assumptions, the major one of which was the state of the ground before superimposed layers were placed on it ; and (*b*) considerable capital expenditure was involved for pavements even in the smallest airfields.

A small trial area of the pavement as designed, or perhaps several areas, should be placed and tested before major construction was embarked upon. That would meet the Authors' views expressed in the second quotation, because in addition to testing the design it should also test the strength of the soil at the same time. It would be of inestimable benefit if the early provision of trial areas could be incorporated in future pavement contracts.

He noted that the Authors considered that where low $k$-values were encountered, it was necessary to provide a working course, with which Mr Goode was in full agreement. He also noted that the Authors proposed that the $k$-value of the base under a rigid pavement should be raised to a value of 300 lb. per square inch per inch and Mr Goode inferred that the Authors would accept the provision of a working course to provide the necessary value. Furthermore, the working course envisaged was one that was now very often used, namely, 4 to 6 inches of rolled dry lean concrete having a compressive strength of about 1,000 lb. per square inch. Mr Goode was extremely dubious whether the $k$-value on top of such a working course would reach anything like 300 lb. per square inch per inch, especially when laid on a moist clay subsoil which was the usual soil needing a working course. Had the Authors conducted any series of tests on different thicknesses of working course on different soils, and, if so, could they give an indication of the results obtained ?

He noted that the Authors did not favour dowels and that that opinion had been generally supported during the discussion. Could the Authors, however, state whether in their opinion it was economically possible to decrease the overall thickness of a rigid pavement by dovetailing or rebating adjacent slabs to transfer loads ? Mr Goode had in mind a method he saw in New Zealand a few years previously, where an apron was constructed in hexagonal slabs rebated into one another. It had the advantage of load transference and a 120-degree instead of a 90-degree corner. He had, however, no knowledge of the effectiveness, or of comparative cost, and had not seen the system used elsewhere.

On p. 81 the Authors recommended separating twin slabs with both bitumen and a building paper. Was there any evidence that both were necessary and would not the bitumen alone suffice ? Furthermore, did the Authors see any disadvantage in using a cheap material, usually at hand, by brushing a clay slurry on to the lower slab ?

Mr Goode was very interested in the Authors' remarks on p. 89 that rolling tests had shown that the omission of stone filling between a concrete base and a bituminous surface gave more stable construction. Could the Authors give some idea of the number of different pavements on which tests had been made which led them to reach such a conclusion ? Mr Goode had consistently advocated the use of composite pavements with the rigid concrete layers low down and stone filling above, since he believed that that was a more economical design and the Authors' conclusion mentioned above was at variance with his own opinion and experience.

Finally, he noted that the Authors were of the opinion that prestressed concrete pavements would provide the answer to many pavement problems. He could see the economy in using a very thin prestressed concrete slab but understood that that would be temporarily supported on dwarf walls prior to stressing and thereafter under-grouted. If those slabs, however, were placed on a soil of low $k$-value, say, less than 100 lb. per square inch per inch, he felt that comparatively high deflexions would occur under each passage of a wheel and that those temporary deflexions would increase owing to cumulative permanent deformation of the soil, and that in time aircraft would have a rough passage over the pavements because of differential deflexions of the various parts.

The Authors' remarks on that aspect would be of interest.

**Mr E. D. Rees** observed that it was refreshing to learn that emphasis was now placed on practical experience in the design of airfield pavements. In the past, design had been more or less based on a theoretical approach. The bearing value of the subgrade was usually taken from results of soil tests carried out in the laboratory and not under practical working conditions.

In practice it was found that, with correct drainage of the subgrade in advance of actual construction of the paved area, in cohesive and non-cohesive soils, the subgrade was able to sustain greater bearing stresses than that shown by results of tests taken in a laboratory. Consequently, the thickness of paving laid was often greater than was necessary.

On one site on which an A.S.P. (aircraft servicing platform) was to be constructed, after the top soil had been removed, the subgrade heaved even when it was trodden on! It was saturated silt containing clay, down to a depth of 4 feet; below that depth the silt was in a liquid state.

In order to proceed with construction the site was isolated with 9-inch French drains laid around the periphery at an average depth of 4 feet and the whole of the site was dewatered by secondary drains, draining into the French drains at suitable intervals; the depth of the secondary drains was 2 feet below formation level. By so doing, a subgrade with an average C.B.R. exceeding 15 per cent was obtained and construction was able to proceed. To facilitate construction, because the subgrade was in non-cohesive soil, concrete beams 12 inches wide by 4 inches deep were laid at 20-foot centres to carry the forms and laying and compacting equipment. A 3-inch sub-base of lean-mix concrete was laid only in the pilot bays to prevent vehicular traffic cutting up the subgrade when delivering concrete to the laying plant.

On another part of the same site where it was proposed to extend the runway, waterlogged sand was encountered at a depth of 6 inches below the surface. Accordingly, before construction could proceed, temporary drains 9 inches below formation level had to be laid on either side of the line of the runway to dewater the subgrade. After that had been done, an average C.B.R. of 16 per cent was obtained and construction proceeded as described above, except that the depth of the concrete beams carrying the forms, etc., on the edges of the runway extended to the invert of the temporary drain in order to contain the sand. As work proceeded the temporary drains were converted to permanent sealed drains, to avoid any possibility of the flow of sand from under the paved area.

Similarly, in cohesive soils, by drainage of the subgrade greater bearing values were obtained than those obtained from laboratory soil tests.

In Mr Rees's experience in all types of soils, with correct drainage of the subgrade, a sub-base was necessary in only the pilot bays to prevent the vehicles that fed the laying and compacting plant from breaking up the subgrade.

The use of a tarmacadam base course with asphalt wearing course for the surfacing and strengthening of concrete runways, Mr Rees thought, was wrong in principle, because a very flexible material was sandwiched between the rigid concrete and the fairly rigid asphalt surface; consequently, under load the tendency was for the flexible material to squeeze out, move, and cause cracking of the asphalt wearing course. The base course should be dense bituminous macadam with an asphalt wearing course.

From 1939 to 1941 tarmacadam had been used for the base course and wearing course of flexible runways. It had been found that the tarmacadam wearing course, even using a maximum-sized stone of $\frac{3}{8}$ inch in an attempt to produce a closed surface, did not stand up to climatic conditions on runways and perimeter tracks, because it fretted and weathered in a comparatively short period. To prevent complete disintegration the surface was dressed with hot tar and cold asphalt. In view of that, up to the time of the first resurfacing programme in 1942, bitumen had been used subsequently as a binder in lieu of tar for the wearing course, and as it had been laid the surface was dressed with cold asphalt at 250 yards to the ton, to give complete seal, and so prevent water entering the subgrade. During that period it was only with difficulty that bitumen was obtained, until the above facts had been reported to the authorities.

Mr Rees was of the opinion that to attempt to design the upper layers of flexible paving to carry high sheer stresses was to destroy the principle of flexible paving and convert the top surface into a rigid surface.

As the Authors had stated the deflexion and settlement of flexible paving progressively decreased under repetition of load, that was to say, provided the depth of construction was such that the subgrade was not overstressed, a stage was reached after sufficient repetition of load, when no further settlement could occur and deflexion of the black top was an absolute minimum; there would be no tendency to sheer under load or high tire pressure. He therefore suggested that when a new flexible paving was being constructed, it should receive a temporary surface of ordinary $2\frac{1}{2}$-inch tarmacadam or bituminous macadam and a wearing course of $\frac{3}{4}$-inch bituminous macadam sealed with cold asphalt at 250 yards to the ton. The temporary surface, being more flexible, would have a smaller tendency to break up than the more rigid dense bituminous macadam during the period of settlement under load. The surface of that temporary black top could be treated at small cost, to prevent deformation under load. In the tropics, even using 80–90 penetration bitumen, that type of paving on runways and aircraft platforms, had stood up to constant use by " Constellations " and " Stratocruisers." When no further settlement could take place then, and only then, should it be surfaced with dense bituminous macadam and an asphalt wearing course.

When subgrades of saturated waterlogged sand were encountered, he suggested that consideration should be given to stabilization by pumping cement into the subgrade.

**Mr Robert Marwick** observed that the analytical approach which was necessary in airfield pavement design for new wheel arrangements, increased wheel loads, and higher tire pressures was bound to produce design practices containing uncertain and contentious aspects which could be finally resolved only by experience of the performance of the new designs under the new loadings.

The Authors' approach for rigid pavement design was, at least as presented in the Paper, purely analytical. It was concerned primarily with the slab itself and, apart from indicating the foundation $k$ as being of comparatively small importance in slab design, it did not cover the overall design of the slab and base in the light of subgrade conditions. Another purely analytical approach, Burminster's three-layer method, was concerned with the pavement as a whole; it, too, had shown a relatively small effect of the foundation on the load/slab-thickness relation but it had indicated that the provision of a reasonable foundation over weaker subgrade might require quite appreciable base work. Burminster's design chart had also indicated slabs thinner than those shown in Fig. 8a for the same loading as being sufficient where a thick base of high rigidity was provided; Fig. 8b showed the same thing for adequate lower slabs and Mr Marwick suggested that a really substantial granular base might, even though it had little or no tensile strength, be stiff enough to give comparable support.

The difference between the indications given by the above two methods of approach appeared to arise from the basic criterion of the Westergaard method, the restriction of tensile stresses in the slab to a safe proportion of the ultimate strength of the concrete. That factor of safety had been related to fatigue failure, but Mr Marwick thought it possible that fatigue failure in rigid pavements might originate in regression of $k$-values through repeated loading. Such regression might tend to result in uneven support across a slab, whereas constant support was assumed in the analysis. It seemed more likely to occur in subgrades of low $k$-value and, assuming that there was very rapid change of the slab thickness/bearing capacity relation as $k$ approached zero, it would then be very dangerous. A lower slab or a substantial granular base would not be subject to such deterioration and higher working stresses might thus be permissible in slabs supported by them. The two types of base differed in being tensile and non-tensile, but Mr Marwick questioned whether the complete avoidance of cracking was the essential criterion for lower slabs.

A comparison between available rigid-pavement loading-test results and the figures given by the slab analysis for the particular conditions of $k$ and tensile strength appeared to be very desirable for confirmation of Figs 8a and 8b, and Mr Marwick considered that the possibilities of high quality granular and soil-cement bases as alternatives to lower slab construction warranted investigation by loading tests. He suggested that investigations on those lines might reveal that the Westergaard approach was, whilst quite correct for the ultimate strength of the slab itself, incapable of indicating all the permissible variations in the overall design of rigid and composite pavements.

Present uncertainties in flexible pavement design had arisen basically as doubts on the validity of the latest U.S. Army C.B.R. design curves for new and heavier loadings since those curves appeared to have been derived analytically from earlier curves which had barely reached the stage of confirmation by tests and by actual performance experience. They could be resolved only by experience and the instance quoted by the Authors of an 80 per cent C.B.R. base course failing under 250-lb.-per-square-inch tires was thus of great value. The usual base-course gradings, however, precluded direct measurement of C.B.R. value and information on how the 80-per-cent value had been ensured or measured would be welcome. Regarding the stability of bituminous surfacing under very high tire pressures, Mr Marwick enquired if the coarse open type of mix generally used by Air Ministry in the past had proved unsatisfactory; denser mixes had been generally used in the U.S.A., but one recent American investigation had favoured the coarser type as being less liable to instability through softening of the bitumen by the heat of jets.

At the other end of the new design curves, the provision of very large overall thicknesses was a serious problem where the lack of suitable superior soils made sub-base construction a quarry-aggregate, instead of an earthworks, task. The permissibility of reducing such thicknesses on the basis of the frequency of loading by the critical types of aircraft concerned warranted consideration in such circumstances.

Mr Marwick considered that the L.C.N. system was not entirely suitable for use in flexible pavement design by the C.B.R. method because the equivalent single-wheel load required for L.C.N. classification of multi-wheel undercarriage loads was a function of the depth to the critical layer and the very different stress distributions under one-, two-, and four-wheel undercarriages were liable to result in a pavement having a different and critical depth for each of the loadings. A pavement might thus have a different L.C.N. for each type of undercarriage and only one of those, that for single-wheel undercarriages, would have a direct significance operationally. He considered that the use of C.B.R. design curves specially drawn to produce constant L.C.Ns for the three main types of undercarriage was liable to produce designs which might not be economically suited to the trend of load distribution in undercarriage design, at least for civil aircraft. The use of separate curves for the three main types of undercarriage was also necessary where different frequencies of application of those types of loading warranted consideration.

**Mr R. L. Mitchell** stated that the Paper gave a very useful summary of the latest developments in airfield pavement design, and was of great interest to him particularly in that the summary of the design concepts of two-layer concrete pavements was clearly expressed.

When designing the concrete hardstanding for the New Salisbury Airport, Southern Rhodesia, Mr Mitchell had considered that matter at length but had eventually decided on a compromise. The Civil Aeronautics

Administration design was used, modified by test data obtained from load-frame tests on existing 8-inch mass-concrete slabs, and led to the choice of a 10-inch reinforced and dowelled slab. His fear, however, had been that in Central Africa, where, although shade temperatures were low, temperatures of 160°F had been recorded on black-top runways, temperature stresses were likely to equal or exceed those of load. Accordingly, he had considered a two-layer design, but had been worried by the rocking of the upper slab, when warped by temperature, on the lower. It was finally decided to use a cement-stabilized sand-clay base of 250 lb. per square inch at 7 days to ensure uniform support, and to eliminate possible pumping, whilst retaining some flexibility. No thickness reduction in the concrete had been allowed, for concrete when underdesigned could not be thickened, unlike flexible paving. That combination of cement-stabilized bases underlying concrete did not appear to have received the Authors' detailed comments.

The existing gravel runway was surfaced with $1\frac{1}{2}$ inch of $\frac{3}{4}$-inch-down asphaltic concrete, the binder being a natural 180/200 penetration bitumen. Load-frame tests on it indicated that it would withstand 400-lb.-per square-inch tire pressures. The Authors had discussed open-graded base courses sealed with dense asphaltic surface courses. Recently it had been noticed on Southern Rhodesian roads that water could penetrate even the dense surfacing and be trapped in the open graded tarmacadam base. Whilst no trouble had yet resulted it seemed to be an undesirable occurrence which should be guarded against.

Finally, there was one matter on which he would cross swords with the Authors. Whilst agreeing that the C.B.R. should be measured at the worst moisture content likely to obtain, he had found that that could not always be achieved even with 4 days' soaking ; it had been necessary to perforate the C.B.R. moulds to correlate laboratory theory with practice, even though Rhodesia's climate might perhaps be described as semi-arid ! The test procedure was to compact a sample at optimum moisture content to the density required in the field, normally 95 per cent of Southern Rhodesia density (approximately 95 per cent Mod. A.A.S.H.O.), and to soak in perforated moulds until all expansion ceased.

Owing to the low density of the residual soils a C.B.R. of 3 was assumed in the natural state and the subgrade was excavated and replaced in 6-inch or 12-inch layers dependent on the compaction plant, to the thickness given by the difference read from the curves between a C.B.R. of 3 and the measured C.B.R. of the sample. Gravel construction between asphaltic concrete was commonly used owing to lack of alluvial aggregates for concrete and the high cost of cement, combined with a normally dry climate where compaction was easy to carry out.

**The Authors,** in reply to Mr Williams, observed that the statement in the Paper concerning standardization related to design methods as applied in practice. They did not mean to imply that the Americans were backward in carrying out fundamental research into airfield engineering problems —in fact they would be the first to agree that America held the pre-eminent position in the world in that field. The Papers by Boyd and Foster [18] and by Pickett and Ray [19] were excellent examples of research into the problems of multi-wheel loads. The former Paper, particularly, arrived at practically the same result as Mr Cooper who, in fact, had discussed his theories with the Americans during the time they were both working on the problem. One of the greatest advantages of the Authors' equivalent-wheel-load approach to design, however, was that it permitted the multi-wheel aircraft to be linked with the L.C.N. system, which was an extraordinarily convenient device for expressing loading effects and one which had, in the Authors' experience, worked very well in all stages of design and practice. That linkage was of course lacking in the American approach and hence application of the research into the effect of multi-wheel loads had lacked the simple device with which to apply it. Fig. 11, put forward by Mr Williams, was an interesting method of applying the equivalent single-wheel-load variable, but one which would, in the Authors' opinion, be more cumbersome to apply in practice than their own method. The work involved in guessing the likely depth of the pavement as a preliminary to the Authors' method of design was probably no more than the trial and error involved in ensuring that the zigzag line CD touched the line AB, which would appear to be a criterion in the method described by Mr Williams if the most economic pavement was to be designed.

On the question of definitions raised by Mr Williams, the ICAO term was of course " equivalent single *isolated* wheel load," which, by its slightly different terminology, had prevented confusion in the past, although the conception was largely the same.

In reply to Mr Brown, the Authors did not share the view that engineers would reluctantly accept their premise on p. 56 that the economics of construction at any particular site would ultimately be the deciding factor in the choice of pavement type. To illustrate how obvious such a choice could be, it was only necessary to examine the details of the pavements that were required to carry an L.C.N. 50 loading on alternative natural foundations of clay and gravel (see Table 1).

It would always be necessary to investigate, for each particular site, the availability and cost of concrete aggregates, base material, etc., both those locally occurring and those available farther afield, particularly when such questions as high concrete flexural strength related to crushed-stone aggregate arose.

Mr Brown's presentation of the single-slab concrete design curves in the form of a nomogram was an interesting alternative to Fig. 8a, but the Authors could not agree that it was any easier to read. Also it was not strictly true to refer to the curves of Fig. 8a as those of Teller and Sutherland. As explained on pp. 79 and 80, the curves had in fact been modified

TABLE 1

| Subgrade | Clay (C.B.R. = 3; $k$ = 100) | | Gravel (C.B.R. = 80; $k$ = 1,000) | |
|---|---|---|---|---|
| Pavement type | Rigid (Fig. 8a) | Flexible (Fig. 9) | Rigid (Fig. 8a) | Flexible (Fig. 9) |
| Details of construction necessary for L.C.N. 50 loading | 4″ rolled dry lean concrete base plus 12″ concrete with flexural strength of 450 lb/sq. in. | 38½″ of construction, i.e. 34″ base plus 4½″ asphalt | 11½″ concrete of flexural strength 450 lb/sq. in. | 4″ asphalt |

from those of Teller and Sutherland to include the results of field testing in Great Britain together with the L.C.N. relation between load and test-plate sizes, and they were closely related to the Teller and Sutherland values only for 12-in.-dia. test plates. In that connexion, detailed information was given in Teller and Sutherland's Paper.[10]

The curves of Fig. 8b for double-slab concrete pavements were based on the modified Teller and Sutherland relation for the top slab corners and the Westergaard assessment of slab-centre strengths for the bottom slab.

The apparent discrepancies between the calculated wheel load for the L.C.Ns between 10 and 70 and the loads that could be read from Fig. 2 were attributable to the diagrammatic nature of Fig. 1. The typical rigid and flexible pavement line of Fig. 1 should correctly be extended into the small plate area from the intersection of the average curve with the 200-sq.-in. line and not in the way shown. The calculated value of the wheel loads did not therefore necessarily lie between the tentative flexible and rigid pavement extensions of the L.C.N. curves of Fig. 2.

In his reference to the bearing value of the natural foundation at Heathrow, the Authors thought that Mr Brown had not perhaps fully appreciated the effect of laying 4 in. of dry lean concrete over a foundation on which to place a pavement proper. Fig. 13, p. 484, had been prepared from recent tests on concrete slabs laid over a natural clay foundation and showed that except for relatively thin concrete slabs (less than 10 in.) the effect was not very pronounced. At Heathrow, a 12-in. concrete slab had been used to carry planned Brabazon loading and for that slab thickness it was necessary to provide an exceptionally good base. That had been done economically by obtaining good gravel from a borrowpit within the site to replace the brick-earth that had existed in the relatively thin layer over naturally occurring gravel. The effect of a thin skin of 4 in. of rolled dry lean concrete over the brick-earth, to improve the equivalent $k$-value, in lieu of that gravel replacement, would probably have been much less.

If Mr Brown's Table of double- and single-slab concrete pavements required for various L.C.N.-values were extended to L.C.N. 100 it would be seen that 16 in. of concrete was required for both double- and single-slab construction. Above L.C.N. 100, double-slab construction would win. It was therefore for the very high loads that double-slab construction came into its own and it was in that application that the Authors had referred to that type of pavement on p. 91. They agreed that single-slab construction was more economical up to loadings of L.C.N. 100, if available compacting equipment was capable of adequately compacting slabs more than 12 in. thick; until such time as equipment was produced, they still maintained that loads in the L.C.N. 60 to 100 range would need double-slab pavements.

Mr Brown's argument concerning slabs being satisfactory without full compaction throughout their thickness depended on the assumption that when corners of the slabs warped downwards they depressed the base on which they were founded to an extent which ensured support over an adequate area. It could not be agreed that that would be so, particularly where dry lean concrete bases were used. Loss of strength arising from lack of compaction was in any case so great, even for a small percentage of air voids, that Mr Brown's statement that the bottom of the slab would "always develop sufficient compressive strength to balance the flexural strength at the top" was not necessarily correct. Scott and Glanville [21] had pointed out that 5% of air voids in a concrete resulted in a 30% loss of strength, whilst 10% air voids resulted in a 50% loss of strength, and lack of compaction of that order or greater at the bottom of a 16-in. single slab of concrete might well result in very little strength being developed at all. Full compaction guaranteed so many other qualities, such as resistance to weathering, slower development of fatigue, less creep, etc., that it would be dangerous to have less than the best compaction as an allowable property.

The estimated concrete flexural strength obtained at 6 months was the value used by the Authors for both large and small works. It was appreciated that in the case of small works, contract periods might be such as to preclude sufficient ageing of concrete up to 6 months before the application of loads. In those cases the risk taken of fully overloading concrete slabs that had not attained adequate flexural strength was reduced by the unlikelihood of there being sufficient number of repetitional loads of such value being applied on any one part of the pavement to cause failure. As mentioned on p. 69, it was probable that a small number of repetitions of load might be taken safely on a pavement since the possible effect was that of reducing the rate of increase of strength rather than reducing the ultimate strength achieved. Such a risk was taken even on the larger jobs where it was always possible that pavements laid towards the end of a contract would be loaded before they were 6 months old.

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21. W. L. Scott and W. H. Glanville, "Explanatory Handbook on The Code of Practice for Reinforced Concrete." Concr. Publications Ltd, London.

The Authors agreed completely with Mr Brown that if the upper surface of the base slab of a double-slab pavement could be finished sufficiently well and smoothly and all joint spaces could be completely filled absolutely true to level, then there would theoretically be no need to provide a separation membrane for any other purpose than to prevent chemical adhesion.

Mr Goode had suggested that small trial areas should be put down to prove the design prior to any pavement being constructed. The Authors, however, considered that, in normal circumstances, satisfactory and economical pavements could be designed without recourse to trial-and-error tests in the field. It was true that with new forms of construction, special loadings, or exceptional site conditions, some form of preliminary site trial would probably be essential; nevertheless, to adopt the principle of test areas in every case savoured too much of lack of faith in design methods as at present developed which, after all, were based on very considerable experience of pavement construction even though the period covered by that experience was so relatively short. A difficulty with test areas was that although it was comparatively easy to accelerate trafficking it was not possible to do the same with weathering and other natural processes.

The question of the value of a lean-mix working-course in terms of $k$ was somewhat difficult to establish. In dealing with the design of a flexible base under a concrete surfacing, the Authors had used the term $k$ to denote its modulus of reaction when measured by a 30-in.-dia. loaded plate placed on top of the base and not in the sense used by Dr Westergaard who had assumed a semi-infinite homogeneous base having the properties of a dense liquid. The $k$-value of 300 lb/sq. in./in. mentioned by the Authors was merely a convenient way of expressing a strength which could perhaps be more readily appreciated if it was described as that which for instance showed a deflexion of approximately 0·1 in. under a load of 10 tons on a 30-in.-dia. plate. Such a strength was considered to represent a reasonable working surface for site traffic. It was also strong enough for proper form laying and ensuring that the forms would be sufficiently stable to stand up to the subsequent loads and vibrations of the concrete spreading and finishing machines. A 4-in. rolled lean-concrete layer, however, could be substituted for the truly flexible granular base on many occasions with advantage, but its strength in terms of $k$ could not be measured, since it acted partially as a rigid pavement and its inherent limited flexural strength introduced altogether different load/deflexion characteristics compared with a truly flexible base. The load-supporting properties of the lean-concrete base when acting as a base under a concrete surfacing had been shown by practical test to be extraordinarily high in comparison with an equal depth of flexible base. Fig. 13 showed the *equivalent k-line* for a 4-in. lean-concrete base plotted on a set of single-slab design curves. That $k$-line had been established from practical full-scale tests in the field. Although its near resemblance in form to the normal $k$-lines permitted it to be regarded as an equivalent $k$-line, it was not possible to give a value for $k$ since it was so far removed from the normal family of $k$-lines.

Mr Goode had expressed some doubts as to the practicability of laying 4 in. of lean concrete on a soft clay formation. The Authors, however, were confident that it could be done if proper laying methods were employed, particularly since the compaction of the concrete could be achieved by very light rollers. In fact if a formation could be excavated by a scraper

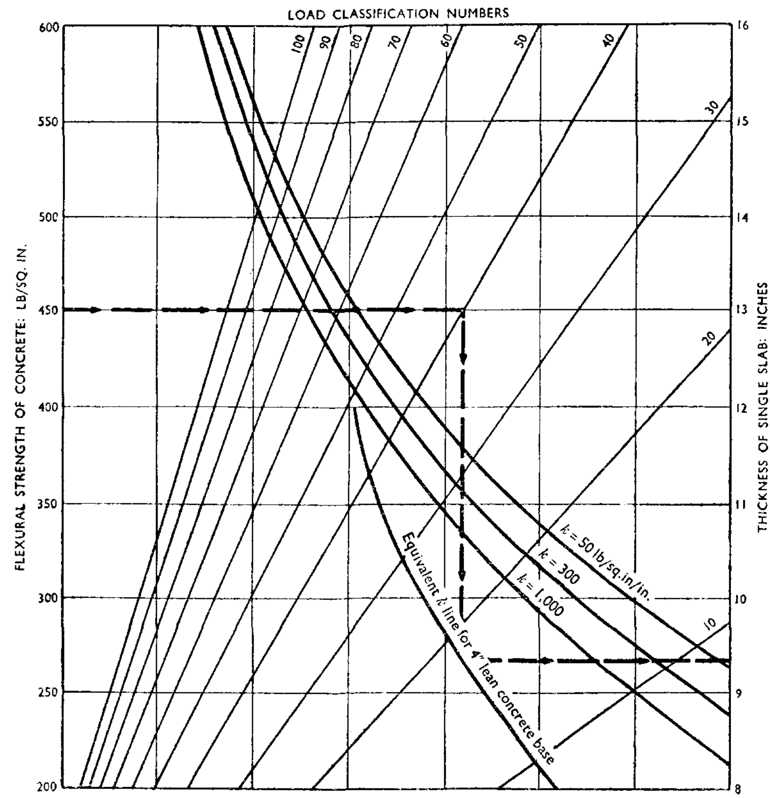


FIG. 13

there was no reason, in the Authors' opinion, why a satisfactory lean-concrete working-course could not be satisfactorily laid on it.

On the question of concrete slab design the Authors did not favour the adoption of keyed joints or hexagonal-shaped slabs, as suggested by Mr Goode. The keyed joint had the disadvantage of altering the section of the concrete, which was very conducive to uncontrollable cracking, and the effort involved in making keyed joints might also be more economically employed by providing the much surer load-transfer device in the form of an underslab under the joint. Hexagonal slabs introduced extraordinary difficulties with machine-laid concrete which would in most cases far outweigh the advantage of the extra strength of the 120°-corner.

On membranes between slabs the Authors had found that the effectiveness of various types of membrane was related to the degree of smoothness of the surface of the lower slab, which could vary greatly in practice, depending on the standard of workmanship employed during laying. With good workmanship a membrane of bitumen from emulsion sprayed at the rate of about 1 gal per 3 sq. yd had given satisfactory results. In other cases building paper had proved satisfactory. Mr Goode's idea that a clay slurry might produce a good cheap " membrane " was interesting—it would of course be very susceptible to damage, particularly during hot and also wet weather.

The Authors' experience on the effectiveness of the stone layer placed between the asphalt and the underlying concrete was related to one particular site where a number of trial sections using depths of stone varying from 3 in. to 8 in. had been tested with plate bearing tests and heavy trafficking. Plate bearing tests showed that the strength of the underlying concrete had not been developed to the same extent as in an adjacent asphalt-surfaced concrete slab. There was a tendency for deflexions greater than would have cracked the concrete slab to be produced at quite low loads, which showed that the stone was being displaced without developing the true worth of the concrete. Trafficking of those trial pavements had shown an irregular settlement probably resulting from lack of uniformity in the stone layer caused by segregation during laying, owing to the hardness of the concrete slab underneath.

Mr Goode's question about the effect of lack of recovery of the formation under the prestressed pavement was of course a problem common to all concrete pavements. The extent to which soil would recover to its original level after being deflected under load was not necessarily related to its basic strength as a foundation but rather to its nature and state of compaction. Full recovery never occurred in practice and that factor was one which the concrete pavement had to contend with whatever its strength. In the case of the 6-in. prestressed pavement the deflexions produced by loadings of the order of L.C.N. 100 would be very small—perhaps of the order of 0·05 in., whereas a deflexion of the order of 0·2 in. or more would be required before the soil was deflected beyond the limit where its recovery was poor. However, the comparatively great strength of the prestressed pavement acting virtually as a jointless slab was the best guard against trouble occurring through lack of recovery of the base. The Authors would point out that the prestressed pavement design mentioned by them in the last part of the Paper had not in fact yet been built and its actual performance could in their opinion only be properly tested by a full-scale trial.

The Authors were in full agreement with Mr Rees's opinion that drainage of a subgrade might prove to be an important factor prior to the construction of a pavement over it, and they had made that point on p. 72. All design did of course have to rely on the assessment of equilibrium conditions achieved in the subgrade which it was assumed would pertain after the pavement was built and the favourable conditions obtained by drains during construction might not always be maintained after the placing of the pavement proper. Such equilibrium conditions were difficult to assess and had to be estimated from such information as was included in Fig. 6, p. 74.

If Mr Rees's suggestion that sub-bases were only necessary if pilot bays were adopted, the resulting pavements might not have non-uniform support, and it was likely that cracking along the edges of the pilot bays would result. It was considered that the most important features of pavement construction were the provision of a uniform bearing at formation level and the provision of the uniform thickness of construction above it.

Mr Rees's criticism of the use of asphalt wearing courses over tar-macadam base courses was agreed where heavy loads and high tire pressures could be anticipated, but for normal low-load aircraft with low-pressure tires, that type of surfacing was perfectly satisfactory and was of course cheaper than the alternative two-course asphalt. In that connexion it was assumed that Mr Rees's description of the base course of dense bituminous macadam did in fact refer to rolled-asphalt base-course material.

Mr Rees's observations on providing a temporary surfacing to flexible pavements prior to the laying of the rolled asphalt base and wearing courses was interesting and if there was any doubt concerning the quality of the sub-base and base materials and their ability to provide high-C.B.R.-value layers, then his suggestion was a practical and desirable one. With good control, however, and proper selection of materials, the build-up of flexible pavements through sub-bases and bases to the surfacing materials could satisfactorily be made to limit any subsequent settlement under load to within safe proportions.

Subgrades of saturated waterlogged sand were of course amongst the most difficult to treat; if it was absolutely essential to construct a pavement over them then it was agreed that stabilization with cement was a possible course to take. The practicability and economics of such treatment would have to be considered for each particular site.

In reply to Mr Marwick's suggestion that a really substantial granular base might react in the same way as the bottom slab of concrete in double-slab construction, the Authors could only point to the results of many field plate bearing tests which had shown that the maximum load carrying capacity of 12-in. single-slab concrete was limited to about L.C.N. 60. That was the sort of value that was obtained with first-class well-graded gravel bases such as those available at Heathrow. Mr Marwick had suggested that perhaps higher concrete working stresses might be permissible in concrete slabs when they were supported by substantial granular bases or by lower concrete slabs. The Authors did not consider that that was so, since by taking cracking as the failure criterion for concrete, failure became independent of what was beneath the slab and was influenced only by the flexural strength of the concrete and the amount through which the slab could be deflected. The action of bases merely restricted those deflexions. It was agreed that slab failure would take place more rapidly through regression of $k$-values through repeated loading, but bearing in mind the small effect that $k$ had on the ultimate load-carrying capacities of pavements, it was considered that fatigue within the concrete slab itself was a more important factor influencing failure.

Mr Marwick's statement that a comparison between available rigid-pavement loading-test results and the figures given by the slab analyses for the particular conditions of $k$ and tensile strength for confirmation of Figs 8a and 8b had of course already been made, and Figs 8a and 8b were well proved by practical tests in the field within the L.C.N. ranges of 10 to 100. Future testing experience would be related to those figures and, if necessary, amendments to the information given would be made.

Mr Marwick's reference to C.B.R. 80% bases of flexible pavements was not correct in assuming that direct in-situ measurements of C.B.R. values had been taken. As stressed by the Authors on p. 75, in-situ C.B.R. testing was made virtually impossible by the presence of larger stones affecting plunger movement and by the fact that in-situ conditions could not be standardized in respect of moisture and density, which was a pre-requisite of the C.B.R. test in its true form. The reference to C.B.R. 80% bases was therefore based on laboratory C.B.R. testing. Such values could only be ensured in the field by observing the requirements of grading and density-C.B.R.-moisture relations previously determined.

As already mentioned in their reply to Mr Rees, the use of tarmacadam beneath asphalt surfacing had now been discontinued in some instances, owing to its deformation under high loads and high tire pressures. More satisfactory results had been obtained by the use of dense rolled asphalt base courses and wearing courses. There was always the possibility of damage to bituminous surfaces being caused by the effects of jet fuel spillage and heat and blast from jet engines, but materials with tar-rubber binders incorporating asbestos were being developed as surfacing materials and early testing appeared to give very satisfactory results.

The possibility of reducing pavement thicknesses on the basis of the frequency of loading by critical types of aircraft had of course been considered and many pavements were allowed in practice to be overloaded for a small number of repetitions of load. Broadly it might be said that aircraft with twice the L.C.N. characteristics of a pavement might make one landing on that pavement without causing damage.

Mr Marwick's observation relating to aircraft having a different L.C.N. dependent upon the types of undercarriage and pavement was of course correct and that was taken into account when using the L.C.N. system. American practice was related to separate C.B.R. design curves for different types of undercarriage. That method was of course quite satisfactory but it had the disadvantage of limiting the use of such curves to aircraft of the particular undercarriage layout for which curves were available.

The Authors fully agreed with Mr Mitchell that under certain circumstances, cement-stabilized bases beneath concrete surfacing would provide the most satisfactory and most economical method of construction and they had in fact observed on p. 75 that, where a sufficiently well graded gravel or sand occurred at the right depth, then in-situ stabilization with cement might be used as a means of improving subgrade strength. Cement stabilization was certainly a method of civil engineering construction that was in its infancy at the present time but which under certain circumstances had obvious advantages. As practical experience of construction increased with its greater use, then it would be used far more extensively and would no doubt be specified as part of the pavement structure. The effect of such a base on the thickness of the concrete surfacing slab would not be great, but in the case of rigid pavements, the introduction of such a processed base went a long way to counteract variations which would otherwise occur in practically all natural foundations. The conception, too, that a concrete slab could not in practice be constructed properly unless there was a base upon which to lay it, had already been referred to on p. 92 and was regarded by the Authors as a fundamental approach to the whole question of rigid pavement construction. It could also be extended as a means of flexible pavement design.

Mr Mitchell's reference to the porosity of bituminous surfacings raised the whole question of how the permeability of such surfacing could be measured. No simple test existed at the present time, and the Authors considered that one was needed and should be developed.

The Authors had put forward in the Paper their opinions on in-situ C.B.R. determinations and those would show that if Mr Mitchell had a soil which, after 4 days' soaking, gave higher C.B.R. figures than those which could be anticipated at the worst moisture content, then the latter figures should be the ones used. In any case the Authors felt that the 4-days' soaking test was fundamentally of little value and were not surprised that it had given extraordinary results on some of the soils of Rhodesia.

Mr Mason had observed that the omission of dowel-bars often led to trouble at the ends of concrete unless the formation was of high bearing value. The Authors had not rejected the use of dowels as safeguards against failure due to foundations of low bearing value but had suggested that whilst it was possible to design satisfactory undowelled slabs then dowels should not be regarded as an essential part of a pavement structure. They only added to the complication of pavement construction. That view had been emphasized by their statement on p. 77. In addition, in practice, dowelling would almost always fall very short of the load transfer characteristics of which they were theoretically capable.

Mr Mason's remarks on reinforcement in pavements were similar to those previously made by Mr Sparks and the Authors' comments were that they felt that the saving in total depth of pavement and concrete thickness resulting from the use of reinforcement was not likely to offset the cost of the steel and the complications introduced into the laying of the concrete. To be of structural value, the amount of steel reinforcement required might easily be equivalent to 3 to 4 in. of concrete and it was very unlikely that its introduction would permit such a large decrease in the depth of a pavement. On p. 113, they had already said that there was probably a good case for experiment to see whether the introduction of a relatively small amount of steel would permit a wider spacing of joints which would, they agreed, be a desirable feature, since joints, apart from being the weakest points of a pavement, affected riding qualities. In spite of arguments concerning the reduced rate of failure by cracking when reinforcement was used, they could not agree that cracking should be accepted and that it was anything but a defect.

Neither could they agree with Mr Mason that the immediate future would be better served by perfecting a technique of reinforced concrete construction. They had already given in some detail on p. 116 a possible solution for the construction of a satisfactory prestressed concrete pavement and they re-emphasized their opinion (given on p. 93) that the greatest advance in airfield pavement design would most probably derive from the possibility of using prestressed concrete. The provision of a 6-in. pavement, evenly stressed in the manner described, would be adequately capable of withstanding any planned aircraft loads of the future.

**Mr A. P. Mason** was surprised to learn that the experience with dowel-bars in airfield pavements was at variance with the results observed in concrete road construction. Load-transfer devices were considered essential in roads and their omission often led to trouble at the ends of the slabs unless the formation was of high bearing value.

It was only in very thin slabs not likely to be used on airfields that dowel-bars should be omitted.

Although reinforcement in concrete pavement did not prevent cracking unless a large quantity was used, its value lay in limiting the size and checking deterioration of any cracks which might form. The cracks in a reinforced slab were more numerous but correspondingly smaller and it was now generally agreed that that type of cracking was not objectionable and did not indicate impending failure. On the other hand a crack in an unreinforced slab rapidly widened and spalled at the edges.

The Authors had stated that 20-foot squares of unreinforced pavement tended to quarter and, as a result, it was the practice to divide the slabs into 10-foot squares by joints and dummy joints. Mr Mason agreed that that introduced a longer joint-length with its allied maintenance problems.

If the slabs were reinforced the joints could be much more widely spaced to the advantage of cost and ridability of the track.

There were so many problems unsolved or insoluble in the construction of prestressed concrete runways that the immediate future would be better served by perfecting the technique of reinforced concrete construction.

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