Airport Papers Nos 32, 33, and 34†

The construction of a soil-cement runway at Christchurch

by

A. O. Barrie, B.Sc., A.M.I.C.E., and Major Jack Cottington

Stabilized-soil pavements at Southend-on-Sea Municipal Airport

by

T. B. Hill, A.M.I.C.E., and K. H. G. Williams

A heavy-duty airfield pavement embodying soil stabilization

by

F. R. Martin, B.Sc., A.M.I.C.E.

Discussion

The following contributions were received in writing.

Mr K. O. Pook (Civil Engineer, Air Ministry Directorate General of Works) observed that one of the most interesting features of the three Papers was the opportunity of comparing methods used for determining the thicknesses of pavement.

355. In all cases the C.B.R. system of design had been used but there was a fundamental difference of opinion as to what C.B.R. the design should be based upon. At Christchurch and Southend in-situ C.B.R. tests were carried out in unpaved areas of the sites concerned, and the designs were based upon average values. In the case of the heavy-duty airfield pavement, however, the in-situ tests were done underneath an existing pavement, where the moisture content could be expected to have reached equilibrium conditions, and design based upon the minimum value obtained. For that reason, higher C.B.R.s had been assumed at Christchurch and Southend than in the case of the heavy-duty airfield pavement, although on the face of it, the gravelly sand soil on the latter site ought to have a higher bearing capacity.

356. The U.S. Army C.B.R. design charts were based upon 4-day soaked C.B.R. values. Whilst that might perhaps be conservative, it was unwise to assume that the soil moisture content and hence the C.B.R. underneath a pavement would remain the same as it was before the pavement was laid. It was generally accepted that, except after periods of heavy rain, the moisture content under a pavement was higher than it would be if the pavement were not there and the water vapour rising through the soil was thus able to reach the surface and evaporate. It therefore seemed more logical to base the design upon the C.B.R. likely to be achieved once the soil had reached its equilibrium moisture content as had been done in the case of the heavy-duty airfield pavement.

357. Furthermore, any engineer experienced in flexible pavement design would be aware that on nearly every site the C.B.R. values obtained varied between wide limits. To assume an average value meant that, if the C.B.R. system of design was correct, half the pavement would be below the designed strength, and it seemed likely that a C.B.R. well below the average should be taken as representative of the site as a whole.

358. Mr Martin gave details of the actual strength of pavement achieved in terms

† Proc. Instn civ. Engrs, vol. 6, p. 577 (April 1957).

of L.C.N., and it would be interesting to know whether any similar tests had been carried out on the other two pavements, and with what results.

Mr G. C. Wilson (Civil Engineer, R. H. Harry Stanger) wished to refer to two aspects of cement stabilization, the first relating to the design of the mix and the second to the field compaction of the cement-treated soil.

360. For the runway at Christchurch the mix was designed to give a minimum field compressive strength at an age of 7 days of 250 lb/sq. in. Allowances were made for reducing mixing efficiency and irregular distribution of the cement in the field by increasing the cement content from 5% which, in the laboratory, gave the desired strength, to 9%. That was equivalent to increasing the laboratory strength from 250 to 680 lb/sq. in.

361. The construction at Southend was based on the soil-cement having different minimum 7-day field compressive strengths for the two layers, 280 lb/sq. in. for the top and 250 lb/sq. in. for the bottom. To allow for site conditions, the cement contents obtained from laboratory tests were increased by the contractor by 2% which corresponded to raising the laboratory compressive strengths to 310 and 280 lb/sq. in. respectively.

362. A different approach seemed to have been followed in respect of the heavy-duty pavement in that the proportion of the soil-cement mixes were decided from the results of tests made with materials produced during the construction of full-scale trial sections. For the in-situ layer the 14-day strengths of the field mix were between 200 and 860 lb/sq. in., whilst for the middle plant-mixed layer the range was from 320 to 720 lb/sq. in. The specified minimum 14-day strengths for the two layers were 250 and 500 lb/sq. in. respectively, the cement contents being determined from further field tests made at the commencement of the main construction. The equivalent 7-day strengths were estimated to be about 180 and 420 lb/sq. in.

363. So far as strength requirements were concerned, therefore, there seemed to be some differences of opinion as to what was required. The first two airfields, although the design wheel loads were considerably different—25,000 lb. and 40,000 lb.—and although they were required to perform different functions, one that of a temporary military and one that of a permanent civil airport, both called for soil-cement having field strengths of the same order, 250 and 280 lb/sq. in. at 7 days. They did in fact achieve averages of 680 and 295 lb/sq. in. respectively, the lower value being for the permanent construction. The heavy-duty pavement which was to carry a wheel load estimated to be about 60,000 lb. required a 7-day strength of more than 400 lb/sq. in. for the intermediate 6 in. of an 18-in.-thick stabilized base, which strength was easily exceeded on average. Messrs Barrie and Cottington quoted a reference supporting their selection of a strength criterion. Could the other Authors give more information regarding their choices of strengths, and particularly could Mr Martin enlarge on the reasons for requiring such a stronger material for the intermediate layer? Furthermore, what justification was there for including in the heavy-duty pavement an even stronger third layer below the 4½-in.-thick asphalt surfacing?

364. It was very disturbing to learn that a large area, 11,000 sq. yd, of cement-stabilized soil constructed at Southend, and presumably complying with the field strength requirements, was severely damaged by frost action. Could Messrs Hill and Williams say if there was any reason to suspect that the work there was in any way unsatisfactory, e.g., was the field strength in fact lower than 250 lb/sq. in.? Had there been any indication since their Paper was written of such damage occurring after the 2-in. surfacing was laid? Would the other Authors say if any frost damage had resulted to the materials with the much higher strengths?

365. It had generally been accepted up till now that soil-cement having a 7-day compressive strength of not less than 250 lb/sq. in. had adequate resistance to weathering. If the material so adversely affected at Southend possessed that strength it would appear that the weathering properties of soil-cement should be studied independently, and the

laboratory examinations of those materials not over-simplified by considering compressive strength alone.

366. Concerning the compaction of the stabilized bases, the desirability of high densities, as distinct from an adequate degree of control to ensure a minimum level of density, did not seem to have been fully appreciated except possibly with reference to the heavy-duty pavement. Rather than determining from laboratory tests what densities should have been obtained, and then selecting plant and devising methods of using it to best advantage to give those densities, in all instances the plant was chosen without consideration of the densities required.

367. So far as control of the compaction operations at Christchurch was concerned, that had been done indirectly by ensuring so far as possible that the appropriate amount of water was present and that a certain minimum compactive effort was expended. However, since the range of moisture content was from 3·3 to 15·8% there was surely the danger that some areas were undercompacted. The difficulties of moisture content and density control were again demonstrated by the data from Southend where the moisture content ranged from 9 to 23% and the dry densities from about 92 to 108 lb/cu. ft. Could Mr Martin supplement that information with more details of the moisture contents and densities measured during the construction of the heavy-duty runway? The percentage compaction values quoted in his Paper were presumably only average figures.

368. Laboratory tests had shown that as dry density was increased, strength and durability of soil-cement, as well as of soil alone, were also improved. Tests of soils made by the Road Research Laboratory demonstrated that with normal plant and proper control of moisture content, dry densities considerably in excess of those obtained in the standard compaction test could be produced, whilst there might be difficulties with mixes of cement and cohesive soils, as were experienced at Southend because of the time interval between mixing and compacting. Mr Wilson was of the opinion that much more attention should be paid to the compaction of the stabilized material, including the control of moisture content. Compaction requirement should be governed by the density necessary for the material to give optimum performance, and not by what plant happened to be conveniently available. The aim should be to increase the dry densities of stabilized bases well above the levels generally accepted at the moment. That called for further improvements in stabilization techniques and the development of plant capable of much better performances. Limits to improvements might be imposed by economic considerations, but those were not easily defined since a pavement for which the stabilized layers were parts should be considered as a whole. Thus, whilst the costs of stabilization might be increased, those might be more than offset by savings on the expensive surfacings and even possibly on the reduced thickness of the better quality bases.

Mr Barrie, replying on behalf of himself and Major Cottington, stated that he entirely agreed with Mr Pook about the importance of the comparison between the methods used at the three airfields for determining pavement thickness; it was obviously a fundamental consideration.

370. He could not, however, agree with Mr Pook's unqualified statement regarding the basis of the U.S. Army C.B.R. design charts. Reports published by the Waterways Experiment Station, Vicksburg, showed that in-situ values had been used for the design charts. For pavement design purposes remoulded laboratory C.B.R. values—soaked or unsoaked—could be used under certain circumstances to predict the probable in-situ C.B.R. value under the pavement when constructed. Under other circumstances the in-situ C.B.R. value could be used for design. Table 18 summarized the design procedure.

371. No one would quarrel with Mr Pook's remark that it would be logical to base the design upon the C.B.R. at equilibrium moisture content—provided that condition would be reached during the life for which the pavement was to be designed.

372. The C.B.R. test was empirical and of all such tests for bearing value it was the

TABLE 18

Soil type / Type of airfield	Cohesionless (P.I. less than 2%)	Cohesive, unwaterproofed	Cohesive, waterproofed
Emergency	In-situ	Undisturbed laboratory samples and 4 days' soaking.	In-situ
Minimum operational	In-situ	,,	In-situ
Full operational	In-situ	,,	In-situ or Lab.*
Permanent	In-situ	,,	Lab. or In-situ†

* Rainfall greater than 40 in. and water-table less than 10 ft down.
† Rainfall less than 25 in. and water-table greater than 10 ft down.

one most generally used throughout the world and would probably remain so until such time as a rational method of pavement design, based on the physical properties of the pavement materials and the subgrade, was evolved.

373. The basic principle of the C.B.R. method was simple enough but the assignment of values, determined in situ, to soils in the field presented many practical difficulties. Since the method had been evolved experience had accumulated and the technique had been improved, but much of the information concerning it did not appear to have been disseminated to the public; clearly the belief that a site had a specific C.B.R. value was a common error.

374. The difficulty in assigning C.B.R. values arose from the nature of soil, which generally had a random structure and whose mechanical properties varied with density and moisture content and, in the case of cohesive soils, with the degree to which they had been remoulded.

375. It was necessary to differentiate between C.B.R. values at a particular location (micro values) and those over a site (macro values).

376. In the field, C.B.R. values could be taken on natural subgrades (compacted or uncompacted, with or without overburden), on imported base materials, or on pavements. So far as the in-situ micro measurements were concerned a number of single readings at each of a number of points, spaced about 1 ft apart, was required. The number taken at any one location would be a matter of judgement. Generally the test was fairly reproducible where conditions were uniform, and variations were due to normal variations in soil characteristics and to the chance orientation of large-size particles directly under the test plunger. The procedure detailed in the U.S. Corps of Engineers Simplified Manual for the Evaluation of Overseas Airfield Pavements gave very practical guidance on procedure. It stated that, for usual conditions, ten to eighteen tests would be required such that an additional test would not change the average materially. That procedure was, however, not normally practicable and generally three tests were made at each location (micro values). The results were averaged when reasonable agreement was secured and tolerances were given for reasonable agreement, i.e., where the C.B.R. was less than 10% a tolerance of 3; 10% to 30% C.B.R. a tolerance of 5; 30% to 60% C.B.R. a tolerance of 10. For example readings of 6, 8, and 9% were reasonable and could be averaged at 8%; readings of 23, 18, and 20% were reasonable for an average of 20%. C.B.R. values below 20% were rounded-off to the nearest point and those above 20% to the nearest 5 points. Where three tests

did not fall within the tolerances, three additional tests were made and the average of the resulting six tests was used as the location (micro) C.B.R. assigned value.

377. If the values for the subgrade at all the representative locations were in reasonable agreement one overall (macro value) should be adopted rather than specific values for individual locations, because the use of specific values made it impossible to arrive at an overall evaluation of the subgrade. The overall value was not necessarily an average of all location values but was a selected value which should not be much above the lowest value in the group; for example, a site having C.B.R. values of four locations of 13%, 18%, 19%, and 20% might be assigned an overall value of 15%.

378. The Uniform Classification System for soils used by the United States Corps of Engineers indicated range of field C.B.R.s attainable in each soil-classification group.

379. The shape of the C.B.R. design curves was such that at higher values, the effect of differences in C.B.R. on the required pavement thickness became progressively less as the C.B.R. value increased. Purely cohesive soils with low C.B.R. values gave consistent micro and macro values which made the problem somewhat easier.

380. No evaluation of the runway at Somerford in terms of L.C.N. had yet been made but M.E.X.E. were now including plate bearing tests in their road and airfield pavement-testing programme in order to relate them to trafficking.

381. If Somerford had been designed purely as a permanent airfield, a prediction of the equilibrium moisture conditions under the pavement would have been made. It had been built as an experiment in Forward Airfield Construction where the life of the pavement would be 6 months in the unsurfaced condition. A 3-in. dense tar surfacing had been added to convert it into a permanent airfield after the flight trials had been completed and, in view of the free draining conditions in the subgrade, the in-situ C.B.R. values were considered to represent reasonably closely the C.B.R. values which would obtain after equilibrium moisture conditions had been reached under the pavement.

382. The Authors both felt that Mr Wilson's contribution was very valuable in that he had summarized the procedures used on the three projects with regard to mix design and field compaction of the cement-stabilized soil. So far as the work at Somerford was concerned the design had been carried out within the limits of knowledge existing at that time, and as a result of the actual construction considerable thought and experimental work had been, and was being, directed towards the formation of a rational mix-design procedure. As Mr Wilson had pointed out, a 7-day compressive strength of not less than 250 lb/sq. in. had so far been generally accepted as the minimum value to provide adequate resistance to weathering. It might well be that that was the required strength for frost susceptible soils and clays, but it did not necessarily follow that it applied to granular materials, and some confusion might arise between the figure required for weathering resistance and that required to provide adequate shear strength. In the case of soil-cement pavements which were to be trafficked without a bituminous surfacing the design would have to take account of weathering and shear resistance; in base or sub-base construction lower shear stresses and less severe weathering influences would obtain. In the case of Somerford the pavement was to be used for 6 months in an unsurfaced condition and a minimum strength of 250 lb/sq. in. at 7 days had been adopted to provide shear resistance. Experimental work was now in progress at M.E.X.E. to determine by weathering trials the cement factor required with clays to provide adequate resistance to weathering when used as a sub-base, base, or pavement. Whether or not resistance to shear or weathering influences was the deciding factor, the problem remained of deciding what strength would be achieved in the field.

383. There were inevitable variations, however small, in soil conditions over a site and samples taken from a treated pavement would have a range of strengths. Considerable confusion did exist regarding the much-quoted figure of 250 lb/sq. in. at 7 days, i.e., whether it was the minimum or mean value. At the time of the Somerford construction it had been taken as a minimum value which had necessitated the adoption of a mean value considerably higher. In addition to the clay-stabilization trials mentioned above, M.E.X.E. were now investigating the performance under traffic of cement-

stabilized pavement, bases, and sub-bases constructed in different soils with different mean 7-day strength values, in order to establish the cement contents required such that field samples would be within certain ranges of value. At present, for practical construction work, one sample per 400 sq. yd of completed pavement was taken.

384. The importance of density could not be overemphasized. At Somerford lack of uniformity had been, to a great extent, caused by the different types of compacting plant used. Even on a site which had superficially the same soil type over its whole area there were variations in soil characteristics and moisture sufficient to cause variations in density when compacted with uniform effort. In addition there were experimental errors in the method of measuring density. The Authors agreed that dry densities well above those generally accepted at the moment were desirable. It was considered that with the present single-pass mix-in-place stabilizing equipment working at constant speed through uniform granular material with uniform moisture content, i.e., ideal conditions, the coefficient of variation of cylinders taken from the processed pavement and tested in unconfined compression after 7-days would not exceed 15%.

385. Density measurements for control purposes were not easy because, to be of value, they had to be taken during processing. Density determinations by means of samples from hardened pavements were simple enough to obtain and were accurate but were of no value except as a record of results achieved. Any machine had a specific compactive performance in a given soil for given operating conditions (height of drop in the case of impact compactors and forward speed) and there was nothing further that the operator could do about it. The compactive capacity of the interim train was known, for it had been fully investigated in different types of soil. On any site of generally uniform soil type there were bound to be, in practice, differences in the mechanical properties of the soil and its moisture condition, which would inevitably give rise to different densities, even though a constant compactive effort was applied. Assuming uniform compactive effort, uniform cement distribution (laterally and with depth), correct and uniformly applied moisture distribution (when the natural moisture content was below design), there would be differences in the compressive strength achieved due to local soil variations, and, in other cases, to a natural moisture content in excess of that required.

386. The logical procedure would therefore appear to be to ascertain, by trial sections if necessary, what density could be achieved in the soil (or soils) to be stabilized, at an optimum moisture content with the appropriate compactive effort. That optimum moisture might not be the optimum obtained in the laboratory density/moisture-content investigation, but should be the best moisture content that could be used in relation to the natural moisture content of the soil on the site; i.e., if the natural moisture content were below laboratory optimum then water would normally be added in the field and conversely, if above, the field moisture content would probably have to be used in determining the field density likely to be achieved.

387. Once the field density had been selected on the basis of soil type, compactive effort and moisture content, the required cement factor to give the desired mean compressive strength could be found from the laboratory design investigation into the relationship between moisture content, density, compactive effort, compressive strength, and cement content.

388. For military operations in wartime, when speed of construction was of paramount importance, recorded data, obtained by previous research, of the performance of the compaction plant, associated with the in-situ stabilizing plant to be used, was particularly desirable in the widest possible range of soil conditions, in order to obviate the necessity of carrying out preliminary work on trial sections before commencing the construction task.

389. Mr Wilson had mentioned a range of moisture contents recorded at Somerford. The Authors would like to point out that those moisture contents were those of the natural soil and that when they were below the required figure, water was added; when above, the soil was turned over, as described in the Paper, until the required condition was obtained.

Mr Hill, in reply, stated that Mr Pook had drawn attention to the different methods of applying the C.B.R. system of design on the three sites and Mr Hill agreed that it would be very helpful if this important point could be clarified. He believed that the originators of the C.B.R. system meant the average C.B.R. value to be taken for design purposes. His own experience on roadwork had indicated that there was a considerable factor of safety in the system itself, and to take either the minimum value, as suggested by Mr Martin in § 184, or to apply the deduction of one or two standard deviations as suggested in § 409, was, in his opinion, too conservative and expensive in capital cost. The deduction of one or two standard deviations as given in Figs 52 and 53 from the mean values in Table 21 would result in very low C.B.R.s indeed. Mr Hill saw no reason for not varying the design to take advantage of better conditions in different parts of a large site, and it was for this reason that the main runway at Southend had been designed on a higher C.B.R. value than the one thought appropriate for the more low-lying sections of the field on which the secondary runway, taxi-tracks, and apron had been constructed. He did not believe that a general C.B.R. value as low as 5% would ever be reached under the new pavements.

391. Mr Pook's other interesting point was the question of the moisture content increasing after a pavement was laid. Mr Hill agreed that as far as possible one should try to predict the equilibrium moisture content figure but felt that the depth of the water-table at a particular site must be taken into account in this prediction. The many Air Ministry tests at Southend had been taken during the winter months and they indicated that the water-table never approached the surface and was, in the main, at least 7 ft below it. The average values of the moisture contents taken from the Air Ministry tests approximately one year after construction indicated that the higher the construction above the water-table the lower the moisture content.

392. In § 408, Mr Martin would mention that the average value of the moisture content contained beneath the old perimeter track was 23·5% and would correctly point out that water had very easy ingress through the pavement which was in a very bad condition and had received no maintenance since the last war. In Mr Hill's opinion these results were not a reliable guide in estimating the equilibrium C.B.R. value which he would now think to average 8% at the ultimate equilibrium moisture content, with a higher value of at least 9% in the main runway across the highest part of the site. He gave his opinion after very careful consideration of the many results obtained as a result of the extensive testing programme carried out by the Air Ministry. As time passed he would arrange to take future moisture contents under the pavements in an endeavour to ascertain the equilibrium condition.

393. Mr Hill then commented on the plate bearing tests at Southend and felt that a very good comparison was now possible with the tests taken at the heavy duty airfield which had been assessed on the same basis. There were, however, two significant points:

(*a*) In 1952 the Air Ministry had appeared to be using criteria of 10,000 repetitions for taxi-tracks and aprons at capacity loads and 1,000 repetitions of load for runways. This was presumably on the basis that runways did not receive the same loading (at least on civil airports) throughout their length as did taxiways and aprons.

(*b*) The "failure" of a pavement as defined by the Air Ministry to be 0·2 in. of deflexion or about 0·1 in. of settlement was an arbitrary criterion of minor failure indicating a slight deformation of the surface which might lead in due course to resurfacing in whole or in part. To a road engineer "failure" meant something more than that and would indicate either extensive crazing of the surface or a much greater settlement.

394. In Table 21, Mr Martin would set out the choice of L.C.N.s which it was suggested could be adopted for Southend Airport and Mr Hill felt some disappointment at the wide range shown in the Table as a result of deducting "standard deviations".

The choice of L.C.N. adopted resulted from a decision on what maintenance expenditure could be allowed and a L.C.N. of 40 was appropriate for a heavy Constellation aircraft, whilst 12 represented a much lighter machine such as a Dakota. He felt that in circumstances like this the capital cost involved in strengthening the whole of the runway to meet theoretical considerations had to be weighed very carefully against the expected aircraft movements and also the normal anticipated life of the asphalt running surface, which he would put at 10 years. It would be appreciated that after a period of time when the airfield required attention, the addition of 1 or 2 in. as a normal re-carpeting programme would probably increase the L.C.N. by 5 to 10 points.

395. He was satisfied that for technical and economical considerations the L.C.N. at Southend Airport could be taken from Column (*a*) of Table 21 and he had agreed to the publication of the figure as 40 (Fig. 55b). It was interesting to note that after 18 months of fairly intensive use by planes up to L.C.N.s of 20 the pavements as yet showed no signs of deformation or wear. He had asked to be kept informed when very heavy planes were using the airport and recently when a Constellation (type 109G; E.S.W.L. 30,000 lb.; L.C.N. 36) had made an emergency landing, it had been left for a period exceeding 24 hours and had carried out engine trials on a recently constructed extension to the apron which consisted of one 7-in.-thick stabilized-soil-layer with a 2-in. gravel-asphalt surface. This was the weakest pavement available at the Airport and had a L.C.N. of only 10; there had been no sign of a failure when the plane left.

396. He was not convinced by the Air Ministry tests that a "reduction factor" could not be used when designing pavements on medium clays such as were found at Southend. Stabilized soil had considerable flexural strength and even when cracked would appear to him to behave like an "ideally pitched" layer of material indicating a strength which would be in excess of the materials traditionally used for flexible construction.

Mr Williams, in reply, stated that economic considerations played an important part in the design of a structure although economy sometimes took second place when it was desired to produce a structure that was durable and maintenance-free. In many designs, an important consideration was the likelihood of failure being a danger to life but in the case of flexible pavements, there was unlikely to be any danger to life should a moderate failure occur, because failures were always gradual and were preceded by surface cracking, which gave adequate warning of a more serious break-up of the pavement.

398. Had the minimum bearing value of the subgrade been accepted for design purposes at Southend, it was estimated that the extra thickness of construction would have cost approximately £50,000 without taking into account any loss of revenue to the airport because of the extra work delaying completion of the hard runways. This additional expenditure represented an increase in cost of the total work of approximately 26%. It was generally accepted that stabilized soil, because of its flexural strength, was stronger than a normal granular base and, therefore, it was permissible to reduce the thickness of bases required by the normally accepted C.B.R. curves. The precise value of this reduction factor varied with the soil type, but if assumed to be approximately 0·75, the C.B.R. of the subgrade under the three-layer work could drop to approximately 6% and under the two-layer work to approximately 8%, before any failures could be expected to occur under frequent use of a 40,000-lb. wheel load.

399. Mr Wilson had referred to frost damage (§ 364). The area affected by frost had been subjected to very severe weather conditions. The stabilized-soil base, protected by only a bituminous curing membrane, had been subjected to a freezing and thawing cycle which had occurred after a period of heavy rain. No doubt a certain amount of water had penetrated the stabilized base before freezing and thawing had taken place, this being responsible for the damage which had been caused. In addition, the stabilized base had been subjected to concentrated use by construction traffic. These were conditions which would not normally occur in the completed and surfaced run-way and it was believed that there would be no further damage of that nature.

400. The compressive strength of the soil-cement in the affected layer was above the specification requirements of 280 lb/sq. in. and the density also complied with the specification requirements of 100 lb/cu. ft.

401. Mr Williams agreed with Mr Wilson so far as the importance of high densities was concerned, and believed that there was little need to increase the compressive strength of soil-cement pavement to ensure resistance to frost. Providing the compressive strength was higher than 250 lb/sq. in., efforts should then be concentrated on increasing the density to a point where the air voids were reduced to the minimum. If there were no air voids, the soil-cement should not take up water and should not, therefore, be damaged by freezing and thawing.

402. In the case of fine grained soils, known to be frost susceptible, the percentage of cement required to produce a frost-resistant material would better be determined by the freezing and thawing test, as described in the revised B.S.1924. Had the density of the soil-cement affected by frost at Southend been higher, it was possible that there would have been no frost damage. It should be remembered, however, that the compaction equipment, suitable for use on cohesive soils, was limited to smooth-wheel rollers and pneumatic-tired rollers and the heaviest available smooth-wheel rollers had been used at Southend. The heavier pneumatic-tired rollers were not generally available in the United Kingdom and, furthermore, they had the disadvantage of producing a severely rutted surface, which was not acceptable in the finished surface of stabilized soil. No further frost damage had occurred and the very thin surfacing was proving entirely satisfactory.

Mr Martin, in reply, offered information which had been promised during the oral discussion. It came under four headings:—

(i) Costs of construction (see § 404).
(ii) Rates of working (see § 405).
(iii) Test data for two layer surfaced construction, all for the heavy-duty airfield pavement (see § 406).
(iv) Test data for the Southend pavements (see § 407 *et seq.*).

404. *Cost of construction.*—The overall cost of the heavy-duty pavements excluding items for airfield lighting and drainage was 28*s.* 3*d.* per square yard of finished pavement. Since this form of cement-stabilized construction had been used for only one section of the taxiway any value, expressed per square yard of all pavements (which included a complete runway and associated taxiway together with standings, hangar aprons, etc.), for the airfield lighting and for the airfield lighting and drainage would be misleading. The make up of the 28*s.* 3*d.* was:—

	s.	*d.*	
(*a*) 22 in. of excavation		10½	per square yard
(*b*) 6 in. of mix-in-place, bottom layer . .	4	4½	,,
(*c*) 6 in. of plant mix, second layer . . .	4	7½	,,
(*d*) 6 in. of lean concrete, third layer . . .	7	2	,,
(*e*) Bitumen emulsion tack coat and cleaning		4	,,
(*f*) 3-in. base course, hot-rolled asphalt . .	7	0½	,,
(*g*) 1½-in. wearing course, hot-rolled asphalt	3	10	,,
Total	28	3	

405. *Rates of working.*—The average rate of working for the bottom in-situ layer had been 400 cu. yd of material processed per 10-hour day and for the plant-mixed second layer 360 cu. yd of material per 10-hour day. The rates for the second layer had been of course very dependent on the position of the stock-piled material in relation to the working areas. The number of men generally required per 10-hour day had been ten for the bottom layer and six for the second layer.

406. *Test data on two-layer surface construction.*—A test had been carried out on an

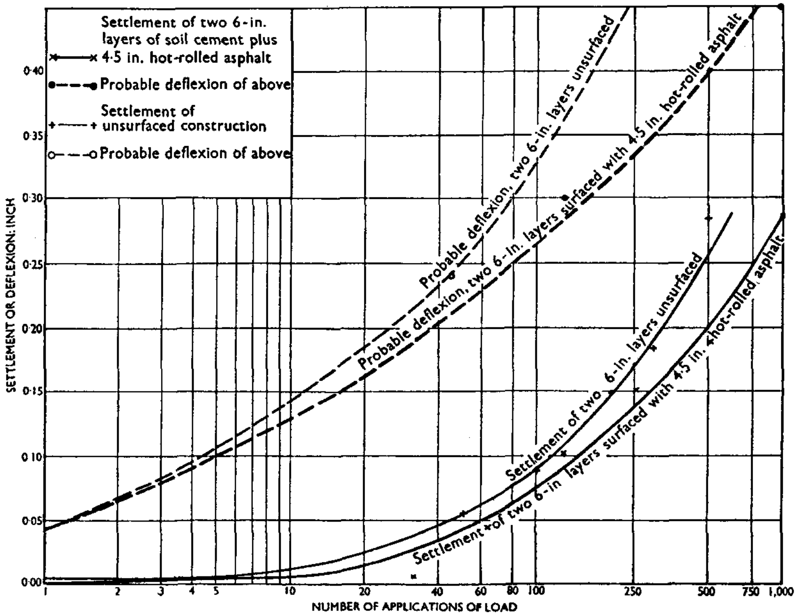

FIG. 47.—SETTLEMENT AND PROBABLE DEFLEXION LINES FOR LOAD REPETITIONS UP TO 1,000

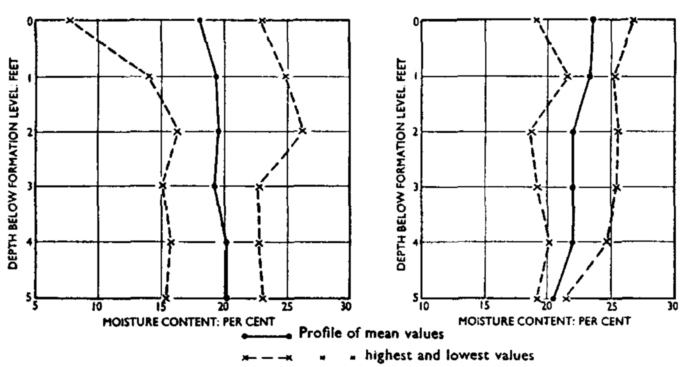

FIG. 48.—NEW CEMENT-STABILIZED PAVEMENTS

FIG. 49.—OLD CONCRETE PAVEMENTS

SOUTHEND AIRPORT. RELATION BETWEEN MOISTURE CONTENT AND DEPTH FOR BRICK-EARTH UNDER OLD AND NEW PAVEMENTS, SHOWING HIGHEST, LOWEST, AND MEAN VALUES

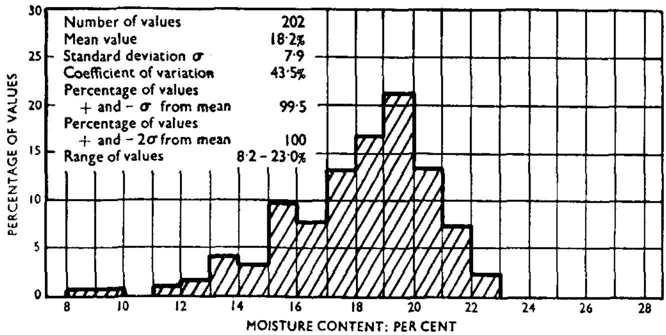

FIG. 50.—SOUTHEND AIRPORT. MOISTURE-CONTENT HISTOGRAM FOR NEW CEMENT-STABILIZED BRICK-EARTH PAVEMENTS

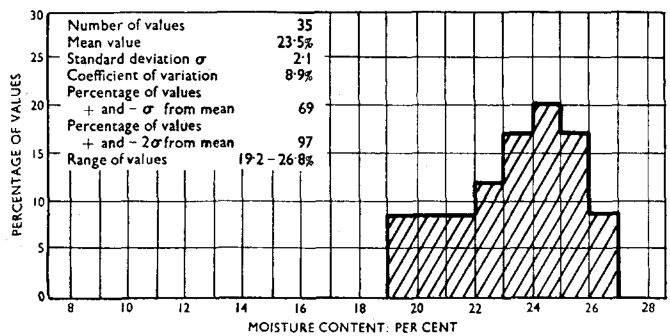

FIG. 51.—SOUTHEND AIRPORT. MOISTURE-CONTENT HISTOGRAM FOR OLD CONVENTIONAL PAVEMENTS

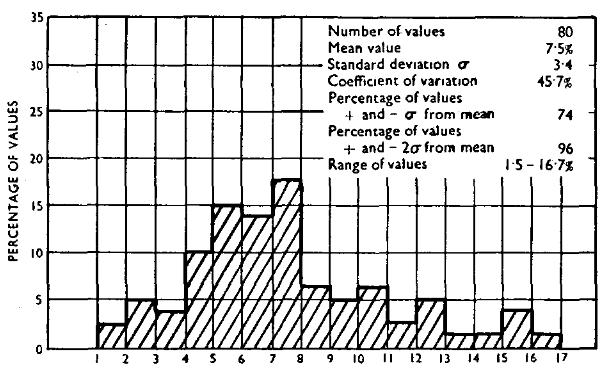

FIG. 52.—SOUTHEND AIRPORT. C.B.R. HISTOGRAM FOR NEW CEMENT-STABILIZED BRICK-EARTH PAVEMENTS

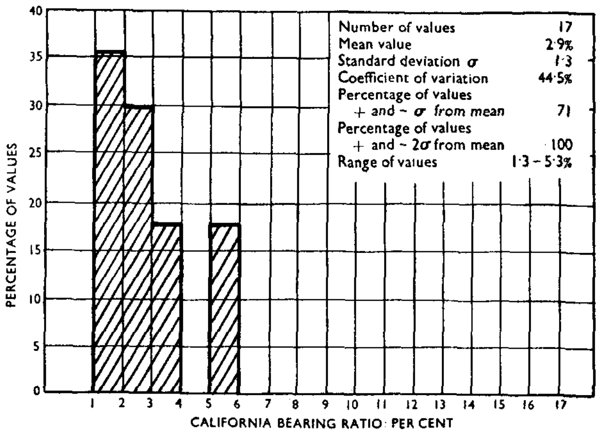

FIG. 53.—SOUTHEND AIRPORT. C.B.R. HISTOGRAM FOR OLD CONVENTIONAL PAVEMENTS

asphalt-surfaced section of the heavy-duty pavement from which the third lean-concrete layer had been omitted. The apparatus used was that illustrated in Fig. 45 (facing p. 616) using a wheel load of 50,000 lb. at a tire pressure of 110 lb/sq. in. (L.C.N. 52). Fig. 47 showed settlement and probable deflexion lines for load repetitions up to 1,000. If the results were compared with those in Fig. 46 it would be seen that the effect of surfacing with 4½-in. of hot-rolled asphalt was to strengthen the unsurfaced two-layer pavement but not sufficiently everywhere to withstand unlimited use of L.C.N. 52 loads.

407. *Test data for the Southend pavements*.—Figs 48 and 49 showed the distribution of the results of the moisture-content determinations beneath the new stabilized pavements and the older existing pavements of Southend and Figs 50 and 51 showed the range of moisture content from the surface to a depth of 5 ft, for both the new and old pavements.

408. From these figures it would seem that the moisture content beneath the new pavement had not reached the ultimate equilibrium value. At formation level, for instance, the average moisture content was 18·2% for the new pavement while that for the old pavement was 23·5%. Some of this variation was doubtless due to the ingress of water through the old pavement and it was difficult to assess the actual effects of this ingress.

409. Figs 52 and 53 showed the in-situ C.B.R.-values obtained beneath the new and old pavements. Each value plotted was the average of three individual test values. It would be seen that the range of values for the new pavement was 1·5 to 16·7% while that for the old pavements was 1·3 to 5·3%. Mr Pook had introduced the very important question as to what value of C.B.R. should be taken for the design of flexible pavements. There was no doubt that subgrade equilibrium-moisture-content conditions eventually achieved beneath any pavement would result in the moisture content being equal to or greater than the plastic limit of the subgrade soil. If therefore the design C.B.R.-value adopted was for a moisture content below that value then of course the pavement strength would later be reduced by the subsequent moisture changes. It was likely that such moisture changes took several years to achieve equilibrium conditions. It would appear that the statistical approach to the problem was probably the most logical. In measuring a series of values of C.B.R., even making allowance for the variations due to varying moisture content, the results would range between minimum and maximum

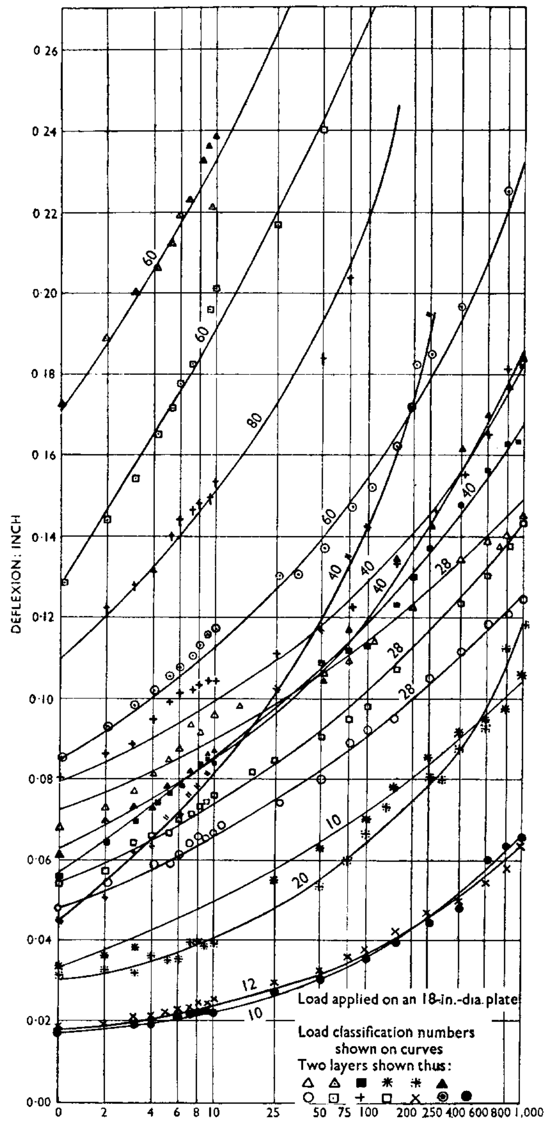

FIG. 54.—TABLE 19 VALUES PLOTTED TO SAME SCALE AS FIGS 32–43

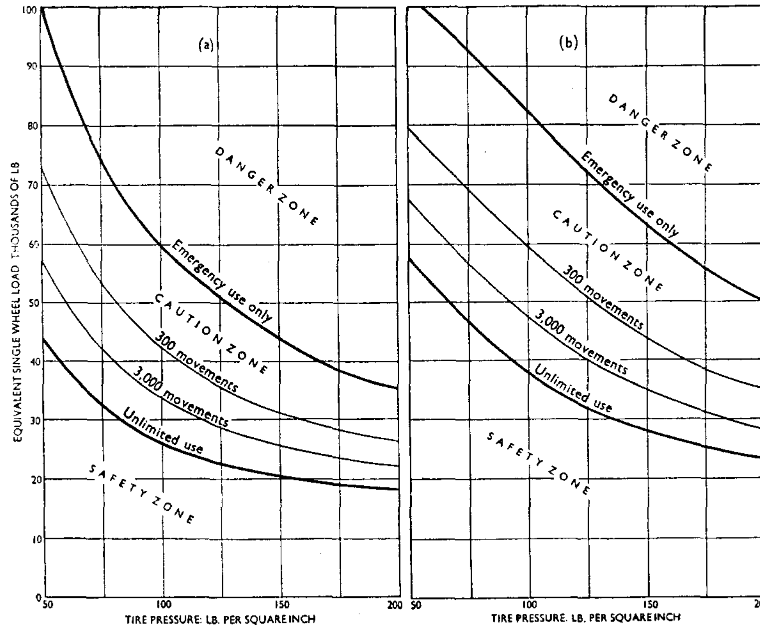

FIG. 55.—SOUTHEND AIRPORT. OPERATING CHARTS FOR (*a*) L.C.N. 30 PAVEMENT AND (*b*) L.C.N. 40 PAVEMENT

(It could be said that these "Safety", "Caution", and "Danger" zones correspond respectively to "normal", "heavy", and "excessive" maintenance of pavements.)

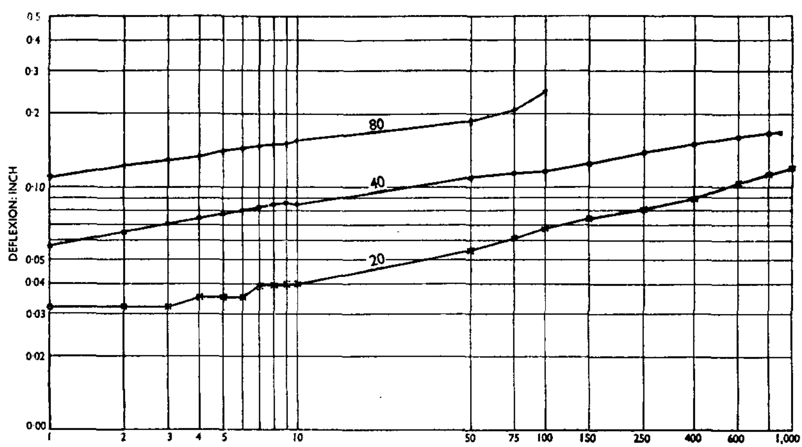

FIG. 56.—SOUTHEND AIRPORT. TWO-LAYER CONSTRUCTION, MAIN RUNWAY

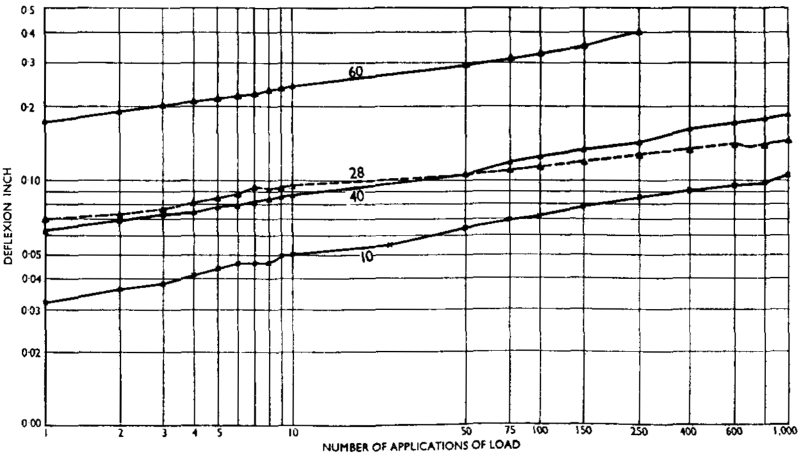

FIG. 57.—SOUTHEND AIRPORT. TWO-LAYER CONSTRUCTION, SUBSIDIARY RUNWAY

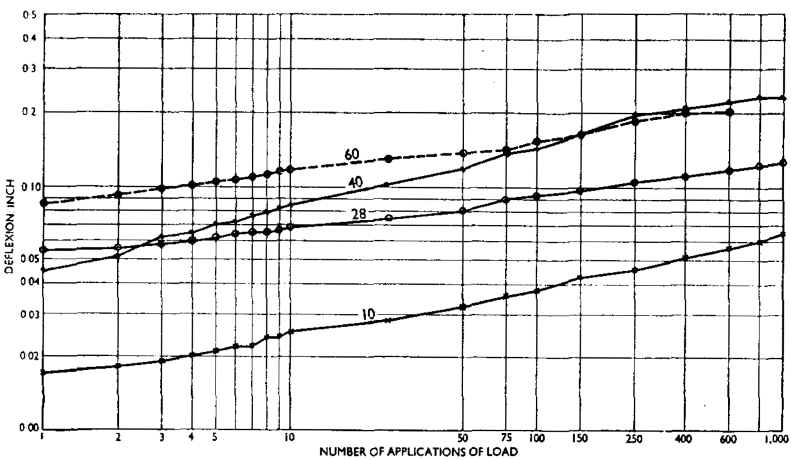

FIG. 58.—SOUTHEND AIRPORT. THREE-LAYER CONSTRUCTION, APRON

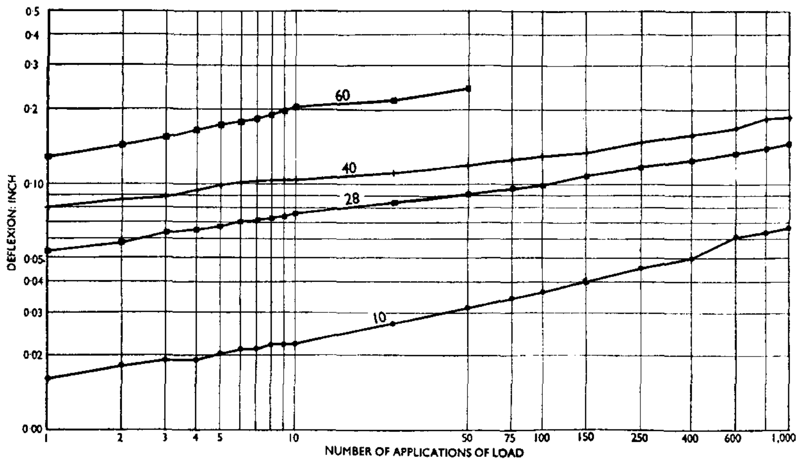

FIG. 59.—SOUTHEND AIRPORT. THREE-LAYER CONSTRUCTION, TAXIWAY

values, and the best measure of their scatter was given by the "standard deviation". Just as a safe "load classification" number was properly assessed by taking into account the scatter of all the test results, then so also should the design C.B.R.-value be assessed. It was suggested that the average value would not normally be taken but rather the average value reduced by say 1 or 2 standard deviations depending on the circumstances of the design. When using a 2-standard-deviation reduction for instance it could be anticipated that only about 3% of the pavement would be founded on a subgrade of lower C.B.R.-value. This was one of the considerations which was in the hands of the pavement designer who could introduce factors of safety into his designs at each stage depending on the known use to which the pavements were to be put, the anticipated life for which the pavements were required, and the economic aspects of maintenance expenditure as compared with capital outlay.

410. With those thoughts in view it was suggested that the design C.B.R. to be adopted from the results obtained at Southend should be in the 5–7% range.

411. Table 19 showed 1,000-applications-of-load tests listing the construction, the load, the load classification number, the number of applications, the maximum deflexion, and residual settlements. Figure 54 illustrated the results graphically plotted to the same scales as Figs 32 to 43.

412. Figs 56 to 59 showed those same results plotted to double logarithmic scales, and it was interesting to observe how closely the plotted lines approximated to straight lines. Because of that it was possible easily to extrapolate for the effects of load repetitions exceeding the measured values and Table 20 showed the load and the L.C.N. for the 1,000-applications-of-load tests and in addition the minimum number of load applications required to give 0·2-in. deflexion estimated from Figs 56 to 59. Table 20 is in the same form as Table 14 on p. 625.

413. Table 21 summarized the remaining tests to assess the load-carrying capacity of the Southend pavements. It gave the L.C.N.-values related to three suggested standards each standard depending on the anticipated pavement usage and the amount of maintenance work which could be provided later. Column (a) related to lightly used runways. For these values the safe loading was the average load required to produce

TABLE 19

Pavement construction	Load on 18-in.-dia. plate: lb.	L.C.N.	No. of applications	Max. deflexion: inch	Max. settlement: inch
3-layer (*b*) apron	15,500	12	1,000	0·065	0·047
3-layer (*b*) apron	25,000	28	1,000	0·125	0·088
3-layer (*b*)	40,000	58	1,000	0·238	0·153
3-layer (*b*) taxiway . . .	12,500	10	1,000	0·065	0·046
3-layer (*b*) taxiway . . .	25,000	28	1,000	0·144	0·093
3-layer (*b*) taxiway . . .	31,000	40	1,000	0·185	0·111
3-layer (*b*) taxiway . . .	40,000	58	50	0·240	0·115
2-layer (*a*) runway (15–33) .	12,500	10	1,000	0·105	0·067
2-layer (*a*) runway (15–33) .	25,000	28	1,000	0·145	0·063
2-layer (*a*) runway (15–33) .	40,000	58	250	0·395	0·270
2-layer (*a*) runway (15–33) .	31,000	40	1,000	0·189	0·092
2-layer (*a*) runway (06–24) .	31,000	40	900	0·164	0·101
2-layer (*a*) runway (06–24) .	19,250	20	1,000	0·119	0·075

TABLE 20

Pavement construction: 6-in. layers of cement stabilized base plus 2 inches asphalt	Load on 18-in.-dia. plate: lb.	L.C.N. (approx.)	Estimated minimum number of load applications to give 0·2-in. deflexion
3-layer . . . (*b*) apron . .	15,500 25,000 40,000	12 30 60	10,000 10,000 500
3-layer . . . (*b*) taxiway .	12,500 25,000 31,000 40,000	10 30 40 60	10,000 6,000 1,500 10
2-layer . . . (*a*) runway . (15–33) . .	12,500 25,000 40,000	10 30 60	>10,000 10,000 3
2-layer . . . (*a*) runway . (06–24) . .	19,250 31,000 50,000	20 40 80	10,000 5,000 75

TABLE 21

Pavement	No. of tests	1,000 load applications			10,000 load applications				
		Average failure load: lb.	Average failure load less 1 standard deviation: lb.	L.C.N. (*a*)	Average failure load: lb.	Average failure load less 1 standard deviation: lb.	L.C.N. (*b*)	Average failure load less 2 standard deviations: lb.	L.C.N. (*c*)
All results .	135	39,900	30,500	38	33,000	25,000	28	17,000	15
Main runway (06–24) .	40	38,300	29,400	36	32,000	23,500	25	15,000	12
Subsidiary runway (15–33) .	50	44,200	34,900	47	32,000	23,500	26	15,000	12
Taxiway and apron .	45	46,000	36,900	51	36,000	26,000	30	16,000	14
Two-layer construction, type (*a*) . .	62	37,500	29,600	37	31,000	24,000	26	17,000	15
Three-layer construction, type (*b*) . .	44	47,000	37,700	53	39,000	31,000	39	23,000	25

TABLE 22

Pavement type	Specified thickness	L.C.N. from C.B.R. design curves for C.B.R.-values of:					L.C.N. assessed from average "failure" load; for 10,000 load applications reduced by 1 standard deviation.
		5	6	7	8	9	
2 layer (*a*) . .	16	17	21	25	29	33	26
3 layer (*b*) . .	20	26	32	37	42	47	39

"failure"* after 1,000 load applications reduced by one standard deviation. Column (*b*) related to taxiways, aprons, and heavily used runways; for these values the safe load was the average load required to produce "failure" after 10,000 load applications reduced by one standard deviation. Column (*c*) related to any areas where repeated heavy use made access for maintenance difficult; for these values the safe load was the average load required to produce "failure" after 10,000 load applications reduced by 2 standard deviations.

414. It was the choice of either standard (*a*), (*b*), or (*c*) that any safety margin considered desirable for administrative and economic reasons was introduced. From statistical knowledge about 84% of the test results lay above the average safe load minus 1 standard deviation, and 97% above the average safe load minus 2 standard deviations.

415. Figs 45 and 45a showed suggested operating charts prepared for L.C.N. 30 and L.C.N. 40 assessments which met the requirements of I.C.A.O. standards proposed by the United Kingdom at Montreal in March 1957. On those two figures the unlimited-use line relates to 10,000 load applications (probably about 30,000 aircraft movements —depending on tire widths, undercarriage layout, etc.) at the pavement L.C.N., and the emergency-use line to 1 load application (probably 3 aircraft movements) at twice the pavement L.C.N. Between those limits there lay a "caution" zone within which various numbers of "overload" movements could safely be planned.

416. Table 22 showed the assessed L.C.N.-values from the normally accepted C.B.R. design curves[8] for C.B.R.-values of 5, 6, 7, 8, and 9% together with the safe L.C.N.s assessed, as in the case of the heavy duty pavements, by the normal Air Ministry acceptance of the safe load being the average load producing "failure" after 10,000 load applications reduced by 1 standard deviation.

417. If a C.B.R. of 7% was accepted as being the highest value which obtained at present beneath the new pavements then the cement would seem to have converted a material that would otherwise have been unsatisfactory for use in pavement construction (particularly for use immediately beneath the thin hot-rolled asphalt layer) into a satisfactory material. It would not seem to have created conditions whereby a reduced thickness of cemented material gave similar strength to a greater thickness of traditional uncemented materials. It would therefore appear that no "reduction factor" to the pavement design depth was applicable to the soil-cement construction at Southend. On the other hand if a C.B.R.-value of 5% was accepted as the figure then a "reduction factor" of 2/3 would apply and this would match well the figure given for the heavy-duty pavement.

418. In reply to Mr Wilson it was agreed that there was much conflicting evidence and opinion as to what was required in the way of compressive and/or flexural strength of cement stabilized layers in a pavement. As already observed on p. 657 the "strength" of small test cylinders could only be a guide to what could be obtained in the actual pavements. The thought behind the specificed strength requirements for the heavy-duty pavement was that the pavement should improve in quality from the bottom to the top. The proof that this was necessary would seem to be given in the test results already published and in the results illustrated in Fig. 47. Whether the criteria of strength as measured in small test cylinders were the correct ones or not had yet to be proved from further testing and research into the problem. Whatever strength requirement was decided upon it could be agreed that the material would always require to be prepared to as economically a high density as possible to reduce any volume change later on through loading. Mr Wilson's plea for the development of robust plant

* The "failure load" for the purpose of flexible pavement testing was taken as that load which would produce, after a given number of applications, a 0·2-in. deflexion beyond the initial settlement caused by one application of the same load.

[8] J. A. Skinner and F. R. Martin, "Some considerations of airfield pavement design". Proc. Instn civ. Engrs, Pt II, vol 4, p. 55 (Feb. 1955).

capable of much better density performance would be supported by all who had had experience of cement-stabilized construction.

419. Mr Wilson had asked for further details of percentage compaction and moisture content values for the in-situ stabilized layer. They ranged in general between 124·5 lb/cu. ft at 5·0%, 125·5 lb/cu. ft at 7·5% and 110 lb/cu. ft at 9·5% moisture.