The value d is the compacted thickness. Compaction is pretty well standardized at around 95 percent Modified AASHO for Airports and 90 to 95 percent Proctor for highways. It did not occur to the writer to elaborate as to the degree of compaction.

There is as yet no definite evidence that mechanical gradation or shape of aggregate particules influence the distributive power of bases. McLeod[4] found "There is no positive evidence that for similar conditions of density and moisture content, all other factors being equal, that any one type of granular base course material has a greater supporting value per unit of thickness than any other type." Dr. McLeod's observations fairly describe our own experience with granular bases.

This factor has long since been recognized in highway construction. It is common practice to improve subgrade bearing value by cross-haul or by importing selected borrow. The proposed formula is of value in determining the bearing power of the subgrade and indicating to what depth this reinforcement should be extended.

[4] Norman W. McLeod "Airport Runway Evaluation in Canada," Res. Rept. 4-B, Highway Research Board (1947), p. 24, 39, 62.

FLEXIBLE PAVEMENTS—DESIGN AND SELECTION OF MATERIALS

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SYNOPSIS

Until recently the Department of Main Roads, New South Wales, used the soils classification developed by the U. S. Public Roads Administration as published in *Public Roads* magazine, February 1942.

Some soils of considerable importance in New South Wales, however, did not fit precisely in this classification; consequently a new system of numerical interpretation based on other simple identification tests was developed locally and tentatively is in use.

This report covers the investigational work through which the new method and its applications evolved. Most of the work has been confined to the eastern half of New South Wales, 29 to 30 deg. South Latitude. Climate varies from wet on the coast to dry at the interior. Drainage is usually satisfactory with the ground water well below the subgrade.

Included in the investigation were: (1) determination of specification limits for surface course materials for use without bitumen; (2) specifications for material courses to receive light bituminous surface; (3) development of a numerical method for designing pavement thickness required over given subgrades or lower base courses.

Involved also were the development of an accelerated weathering test for shales for pavement construction or for the subgrade and a method for assessing quality of sandstones.

Study of traffic influences indicated that magnitude of load was more important than frequency of heavy load for determining if failure would occur. However, frequency of heavy loads influenced the time at which failure occurred. Load had noticeable effect on total pavement thickness, but small effect on the required thickness of surface course.

The design method is set out in two appendices to the report. The first treats the test procedures which follow the Public Roads Administration methods with certain exceptions as explained. The second appendix gives the numerical method of interpretation of test results, and applies two rules: (1) reduce to a single number the effect of departures from maximum density grading taking Wilhelm's exponential series for maximum density; and (2) adjust the variable to a point-score system for evaluation correlated with service.

It is concluded that the same rules and formulas could probably be safely used in other areas with similar climate and traffic

In the construction of flexible pavements the Department was continually called upon to answer two major questions: (1) Will this material do for the pavement? and (2) How thick a pavement is required?

Many methods had been developed for producing answers to these questions. They varied in their practicability, reliability, and scope of application. In the first place, the most promising of the methods then in use were studied and compared with New South Wales experience.

The tests and methods proposed by the U.S. Public Road Administration (then Bureau of Public Roads) were selected as the best suited to the Department's needs. Answers given to the second question were at first based on comparisons with reference soils for which suitable thicknesses had been determined by experience, but in *Public Roads*, February, 1942, the PRA published, inter alia, suitable ranges of pavement thickness for its various standard soil classifications. The Department then adopted the practice of interpolating pavement thicknesses between these class limits according to the test figures reported. This proved generally satisfactory, but as experience was accumulated it was found that:

1. The PRA system did not specify the weight to be given to each test and there was frequently a margin of uncertainty in deducing pavement thicknesses.
2. The system did not cover certain types of soil of importance in New South Wales road works.
3. Certain minor numerical adjustments seemed desirable (e.g. lower limit of pavement thickness should be less for A4 soils than for A5 soils).
4. Some of the tests appeared to be of relatively less importance in interpretation and application or appeared to be closely correlated with others.

A study of the data available indicated that while retaining the basic principles, it might be possible to reduce the number of tests conducted and still produce satisfactory answers. This would reduce considerably the time and effort of testing; moreover the tests retained would only require simple and easily portable apparatus and could readily be conducted on the actual work. Owing to the distances involved and the low population density, this last point is of utmost importance in New South Wales The study also indicated that it might be preferable to recast the method of numerical interpretation along new lines.

These sweeping proposals had to be checked over the widest practicable range of materials and conditions before adoption. The investigation now described was therefore undertaken. The data collected indicate that, for New South Wales at any rate, the system could well be modified on the lines suggested above. The modified system has been developed in detail and is set out in Appendices A and B. It has been tentatively adopted by the Department of Main Roads and put into use on its works. It is proposed to review the methods when further experience has been gained in their practical use and data become available from a proposed joint investigation of flexible pavement design to be conducted by the several Australian States.

This modified system involves a testing routine similar to the PRA system, using fewer tests, and new methods for direct computation of numerical answers from the test figures. Despite the apparent complexity of the formulae this method is simple and rapid in practical use. The new system appears to be fully reliable under the conditions met with in New South Wales and covered by the investigation. The answers produced agree well with the interpolation between PRA (1942) limits of thickness for the soil groups.

BASIS AND SCOPE OF INVESTIGATION

The whole of the work was based on actual experience and the service records of existing pavements. Samples were carefully selected from many different areas in New South Wales to cover, as far as reasonably practicable, all important types of soil and of local pavement materials. Samples taken were correlated with the actual service obtained by inspection prior to sampling and examination of available records. The test results of such samples were tabulated and analysed and conclusions drawn. There is nothing novel in the method; the value of the work rests on the number and scope of the materials sampled and tested, and the consistency of the correlation between service records, test results, and the numerical interpretation system developed.

In flexible pavement construction in New South Wales the following classes of materials are used for the actual pavement:

1. Bituminous mixtures and surfaces of various kinds.
2. Macadam and similar constructions depending on the mechanical interlock of hard angular aggregate.
3. Natural gravels, sand-clays[1] etc. including mixtures of two or more natural materials.
4. Shales, etc. broken up by compaction, weather, and traffic to make a material equivalent to gravel.
5. Crushed materials combined, and if necessary blended with natural material, to make an artificial gravel.
6. Soils, etc. stabilized with bituminous materials, cement, etc.
7. Sandstones, and other rocks of similar texture, which do not produce a good grading on crushing, are too soft for use in macadam etc., but sufficiently hard for use as base courses (see last paragraph of this section).

In addition, there has to be considered the subgrade on which the pavement rests and, with poor subgrades, the possibility of using selected or blended soil sub-bases to reduce the thickness and cost of the pavement proper.

Methods for the design of bituminous mixtures, etc., for the selection of aggregate for such mixtures and for the selection of materials for macadam etc. are well established. The investigation now described was not directly concerned with such constructions.

Natural gravels, sand-clays, and shales form by far the most important pavement materials in New South Wales from the economic point of view. With its low average population density and the high cost of imported bitumen the greater part of the improved road mileage has pavements of such materials either unsurfaced or with light bituminous surfaces only. Only on the more heavily travelled roads are heavy bituminous and concrete pavements provided. In 1946 approximately 1,600,0C0 cu. yds. of gravel, sand-clay, and shale were used on "main roads" alone, and the annual use is increasing.

[1] In New South Wales road practice the term "gravel" is used for graded mixtures of coarse aggregate, sand, silt, and clay depending for their strength on the combined friction and cohesion of a well graded soil mortar. The term "sand-clay" is used for a similar material without coarse aggregate.

Where these materials are used for pavement construction, New South Wales experience has been that the use of inferior upper courses in unsurfaced roads, and of inferior courses immediately under the bituminous surface of surfaced roads, has been the cause of by far the largest proportion of all failures and unsatisfactory service. The most important part of the work was, therefore, to determine the allowable specification limits for such courses. It was apparent that the requirements were somewhat different for materials for use unsurfaced and for materials to receive light bituminous surfaces. In both cases the limits were determined by testing a wide range of samples taken from pavements with known service records.

Having determined these limits they can be applied directly to deposits where the aggregate is hard and not liable to weathering. This is the case with a high proportion of good natural materials. If the aggregate is soft or brittle, samples have to be treated before test to duplicate as closely as practicable the compaction, and consequent fracture, the material will receive in the pavement. If the aggregate is liable to disintegrate under the action of the weather some form of accelerated weathering must be applied to samples from deposits prior to testing. This is of special importance for shales, etc., and a suitable accelerated weathering procedure had to be developed. For this purpose samples were taken from deposits and from old roads known to have been constructed from such deposits without admixture. A suitable weathering procedure was developed by comparative tests on these samples.

In the case of subgrades, sub-bases, and lower courses, the practical design problem is to determine the thickness of pavement or upper course which is required on top of the material in question. Samples were taken from pavements and subgrades in service and test results compared with service records and the thicknesses and types of the overlying materials. This section was confined to surfaced pavements, including both light and heavy types of bituminous surface. The selection of the pavement thickness to be used is relatively unimportant on unsurfaced roads, which continually lose thickness by wear and attrition. Extra thickness in initial construction merely means advancing the date of

necessary expenditure. On the other hand, too thin a pavement can readily be thickened without real loss and the unsurfaced road lends itself readily to "stage construction". The situation is very different once an expensive bituminous surface is constructed.

There are areas of sedimentary country in New South Wales in which natural graded materials and igneous rocks are rare and the shales weather rapidly to clay on exposure to weather. In these areas sandstones are the only freely available local materials for road construction and have been used with considerable success when protected from wear by bituminous surfaces and (or) other upper courses. They are normally too soft to resist traffic abrasion unsurfaced. No satisfactory method of testing such materials for these uses being known, a further section was added to the investigation. Sandstone pavements depend on the wedging and bedding of large pieces, assisted by the jamming and friction of the smaller pieces and sand in the voids, and not on the stability of a graded matrix. The tests and basis of selection are, therefore, quite different from those used in the other sections of the investigation.

TESTS

In the final system so developed the number of different tests conducted on each sample of soil was reduced to the minimum consistant with satisfactory correlation with service. In the earlier stages more tests were conducted but were eliminated when they were found to be of little value in evaluating the soil quality or to be unnecessary because of their correlation with others. Certain of the tests included in the new system for testing soils and graded pavement materials differ from the PRA tests.

Mechanical Analysis—The sieve sizes differ from PRA because the No. 10 and No. 270 U. S. standard sieves have no counterpart in the normal British standard sets used in Australia. Moreover, there is no regular relation between the sizes in the PRA system while a regular size progression greatly simplifies the numerical work. The sizes used are B.S. 7, 36 and 200, equivalent to U.S. 8, 40 and 200 or Tyler 8, 35 and 200.

The analysis by sedimentation is only carried out for 0.0135 mm., the next size in the same regular progression. Data collected indicates that this, in conjunction with other adopted tests, is sufficient for purposes of routine control and that the extra effort and difficulty of finer sizing is not warranted.

The adoption of this single and not very small particle size determination greatly simplifies the mechanical analysis by sedimentation. The old method of complete separation by repeated decantation from a beaker is convenient and practical and, when handling a number of samples, requres less total time than the hydrometer or pipette methods.[2] Over twenty samples can be handled simultaneously by a single operator and the residues, after separation of the less than 0.0135-mm. material, dry screen readily on the 36 and 200 sieves, moreover there is not the need for the great care and precautions to ensure dispersion before start of test that are necessary with the single settlement methods. The continued dilution, leaching, and stirring aids full dispersion. If effective dispersion is not secured, and no known method will effectively disperse some of the soils encountered in the investigation, this becomes apparent during the decantation method, even if masked at first by the dispersion of a portion of the clay.

In clay soils there are more or less systematic differences in the mechanical analysis figures according to the dispersion procedure and peptizing agents used. This is probably mainly due to slight residual flocculation but physical or chemical disintegration of clay minerals may be a contributing factor. Any result reported is to some extent arbitrary and a numerical interpretation system should be based on a standardised procedure. The procedure adopted was designed to secure reasonably satisfactory results with the least possible apparatus and the minimum of attention. It consists of boiling to rehydrate clay and break up aggregates and continued stirring, dilution, and decantation. Decantation by tilting and pouring was preferred to the syphon or side tube methods as it was found to cause less disturbance of the lower layers. Ammonia is used as the peptizing agent as it is satisfactory for the particle size separation desired and its volatility avoids complications.

[2] This would not be the case if sizing was carried out below 0.0135 mm. or if more than one size separation by sedimentation was desired.

Liquid Limit and Plastic Limit[3]—PRA methods are adopted in principle but in routine control accuracy to one unit is considered adequate. With this standard the procedure can be abbreviated, and the time reduced. To expedite the work these tests are carried out by the Department on samples dried at 105 C. before testing. The test data shown in tables were also obtained on samples similarly pre-dried. The small reductions in test figures due to pre-drying have been found to be reasonably consistent and are not of practical importance.

Maximum Dry Compressive Strength—This is determined for consolidated cubes of graded pavement materials to guard against materials that would ravel excessively when dry. There is no corresponding test in the PRA system because non-plastic materials are excluded from the uses for which this factor is of importance. The investigation, however, shows that non-plastic materials otherwise complying with test requirements and of sufficient dry strength are satisfactory and should not be excluded.

Shrinkage—This is a most important test in assessing the quality and performance of a sub-grade. The investigation has shown that the linear or volumetric shrinkage is the most important factor and that the shrinkage limit and ratio need not be determined to secure satisfactory correlation with service. The PRA method provides for the determination of all shrinkage constants but a test to determine the linear shrinkage only is much simpler and more rapid to conduct. Again the PRA standard is to report the shrinkage from the FME which is not reproducible with many soils. The linear shrinkage from LL is much more readily reproducible and is almost independent of shape of mould used. This is therefore the form of the shrinkage test included in the system as developed.

Proctor Compaction—This most useful test is conducted by the PRA method.

In the tables for soils, graded pavement materials, etc. only the results of the tests finally adopted are shown. Shales, etc. are subjected to similar tests after artificial weathering. Sandstone, etc. is tested for wet strength and, if required, for stability against weathering.

The Department for some years has been testing large numbers of samples by these methods. The standardised procedure developed for dealing with this work is set out in Appendix A.

[3] Sometimes called Lower Liquid Limit and Lower Plastic Limit.

CLASSIFICATION BY CONDITIONS

The samples collected and tested were classified and tabulated by climate, traffic loads, service record, etc.

Climate—New South Wales is about 310,000 sq. mi. in area, it is roughly rectangular in outline, bounded on the east by the Pacific Ocean and on the other three sides by other States. It extends approximately from 29 to 36 deg. S. latitude. It is largely a country of low relief, two-thirds of its area is less than 1,000 ft. in elevation and only about 2 percent exceeds 4,000 ft., but there are areas of very rugged topography, principally in the eastern portion. Temperatures are generally mild to hot; ground freezing and frost heave are almost unknown in roads, and the isolated localities where these occur were not included in the investigation. Caution must, therefore, be exercised in extending the results to areas subject to severe freezing.

Rainfall is generally highest on the coast. averaging about 50 in. a year with local maximum exceeding 70 in., decreasing westerly The western 20 percent of the State receives less than 10 in. and the western half of the remainder less than 20 in. per year. There are naturally considerable variations associated with local topography. Evaporation is high, particularly in the western districts, and is closely correlated with rainfall. For road construction rainfall is by far the most important factor in assessing New South Wales climates and, for the purposes of this investigation, the State was arbitrarily divided into four zones·

Wet —Average annual rainfall over 35 in.
Medium —Average annual rainfall 25–35 in.
Dry —Average annual rainfall 15–25 in.
Very dry—Average annual rainfall under 15 in.

The present investigation was confined to the first three zones which, apart from isolated mining areas, include all the more closely settled areas and have a combined area of about 150,000 sq. mi.

One factor that is of importance in considering the data collected is that, in almost all of New South Wales, rainfall is erratic, varible, and practically non-seasonal. Even the high rainfall coastal areas are subject to droughts lasting weeks or even months and any part of the State has occasional periods of soaking rains. The dry areas moreover are mainly plains and country of very low relief from which, sandy areas excluded, water is removed slowly after rains.

Drainage—The normal objective in road drainage is to remove surface water rapidly from the road and shoulders and to keep the free ground water surface well below bottom of pavement or sub-base. It is usually impracticable to lower the free water level sufficiently to keep the pavement clear of the capillary fringe.

This standard of drainage will normally result in the subgrade and pavement becoming quite damp from capillary water, especially where evaporation is checked by a bituminous surface. It is uncertain to what degree the equilibrium moisture content varies with ground water level and other conditions — the time required to reestablish equilibrium after changes in level of ground water is also unknown. Moreover, it is very difficult to determine the maximum level of ground water and the duration of high level periods at any point on an existing work, and almost impossible to predict these factors in advance of construction. It is, therefore, scarcely practical to attempt to take detailed account of variations in capillary moisture conditions.

It is generally tacitly assumed that under normal and satisfactory drainage conditions, where pavement and subgrade are exposed to capillary water only, a given material, if compacted and confined, will attain about the same moisture content, regardless of the usual differences in capillary moisture conditions, and will therefore have a fairly constant shear strength; or, what amounts to the same thing, that the pavement required on a given soil, and the types of materials suitable for pavement construction, are reasonably constant. This assumption was made for the present investigation, all samples were taken from points that, by appearances on the ground, could fairly be described as normally and satisfactorily drained. No attempt was made to undertake the very difficult task of subdividing drainage conditions within this range. The concordance of the majority of the results gives strong evidence in favour of this basic assumption. Doubtless some of the few discrepancies are due to undetected abnormal local drainage conditions.

Traffic Loads—Apart from the dense traffic in and near the cities of Sydney and Newcastle, the traffic on New South Wales roads is generally of small volume by USA or European standards. Maximum loads are, however, quite high throughout the State. Practically all heavy loads move on pneumatic tyres. Metropolitan roads with high traffic densities are commonly of rigid construction, but flexible pavements are used for most suburban and rural roads. The roads may be divided into two main groups, according to maximum wheel loads as follows:

1. *Heavy.* These occur only in and around the two major cities and on a few other routes regularly operated for short-haul bulk traffic from heavy industries, mines, etc. Here road surfaces are good, volume of goods hauled is large and regular, and ample service facilities are available. Some operators regularly load their vehicles well in excess of their rated capacity and single wheel loads up to six tons, or say 13,500 lb., on a dual pneumatic-tired wheel were commonly observed.
2. *Normal.* In other cases the normal maximum wheel load is four tons, or say 9,000 lb., on a dual pneumatic-tired wheel.

In both these cases there are occasional heavier loads but these are rare and were not considered. Loads on single-tired wheels rarely exceeded 60 percent of the above loadings and did not need to be considered.

It was thought at the outset of the investigation that frequency of repetition of large wheel loads might be an important factor and that this would have to be estimated from the meagre census data available. As the work progressed, however, the results seemed to indicate that frequency lf loading greatly affected the time of failure but, as compared

with magnitude of load, had relatively little bearing on whether failure eventually occurred. Reliable records of past frequencies were rarely available and, owing to changes in the traffic pattern due to war conditions, past frequencies could not be reliably estimated from present traffic. This variable was, therefore, eliminated from the data,[4] but care had to be taken that pavements sampled were of sufficient age to ensure that failures had developed. This may require up to ten years on lightly travelled bituminous roads or even more in the drier locations where subgrade and pavement are not always saturated with capillary moisture. Two interesting consequences followed from this. First the data, being collected largely from old roads, contain many examples of the one time popular bituminous penetration construction now relatively little used in New South Wales. Second, surface treatment on gravel and sand-clay is a fairly modern development and the average age of such roads sampled was less than for other bituminous types. This is reflected in the data in Table 2, in which the inconsistencies in the "sound" section would probably be reduced by further failures under longer service life.

The division between heavy and normal traffic is of great importance in the case of subgrades, base courses, and sub-bases, and was assessed by a study of general traffic movements and industrial development. There is naturally no sharp end to the heavy traffic zone in many cases. Samples of subgrade soils etc. were not taken from lengths of road where the traffic classification was doubtful.

For surface courses the maximum tire pressure is the controlling factor. This is approximately the same for both classes of loading, so no distinction was made.

DATA OBTAINED

Table 1.—Table 1 gives the data for unsurfaced materials depending for their strength on the stability of a well graded soil matrix. They comprise natural gravel-sand-clays, natural sand-clays, mixtures, and artificial products. In view of widely held theories on the importance of the coarse aggregate in the stability of such materials, the results were classified by its type and size. "Round", "Normal" and "Angular" classes refer only to gravels, etc. with a maximum size over $\frac{3}{8}$ in. and are otherwise self-explanatory. "Fine" refers to gravels etc. with aggregate not exceeding $\frac{3}{8}$ in. in size, mainly decomposed granite. "Sand-clay" is applied to materials containing practically no coarse (less than 10 percent retained No. 7 BS).

The service records given refer only to the stability of the materials wet and dry and need no further explanation. The data shows that, contrary to the opinion of many engineers, the nature and amount of coarse aggregate in graded materials is not the most important factor in stability under traffic loads. The aggregate has, however, an important bearing on traffc wear, and on the ease of handling and compacting the material and maintaining the pavement. These properties are obvious and well known and required no special investigation.

Table 2.—This table gives the data for materials, similar to those covered by Table 1, that have carried light bituminous surfaces of the "Surface Treatment" type.

Table 3.—Table 3 gives the data for soils, sub-bases, and base course materials under bituminous pavements of varying types and thicknesses. It is divided into four sections. "Failed" and "Sound" are obvious classes. "Stage Construction" means that pavement was progressively built up over many years by successive additions till a stable road was achieved. The pavements so constructed should be about the correct thickness or not very greatly in excess. "Correct Thickness Judged" means that by comparison of failed and sound areas or by slight signs of movement without noticeable failure it was possible to estimate the minimum thickness required to avoid failure.

In the column "Pavement Type" the types of construction used in the various courses are shown in succession commencing with the

[4] Frequency of loading on roads included in the investigation was, for the most part, in the approximate ranges:

- Heavy loading 200 to 800 total vehicles per lane per day.
- Normal loading 50 to 400 total vehicles per lane per day.

TABLE 1[5]

UNSURFACED SAND-CLAYS AND GRAVEL-SAND-CLAYS

R = % of Material Passing ¾-in. square Retained No. 7 B.S.
A = % of Material Passing No. 7 B.S. Passing No. 36 B.S.
B = % of Material Passing No. 36 B.S. Passing No. 200 B.S.
C = % of Material Passing No. 200 B.S. Less than 0 0135 mm.
MDC = Maximum Dry Compression Strength, lb. per sq. in.
USL = Upper Solid Limit.
P = PL if Plastic or LL if Non-Plastic.
For "Aggregate" and "Climate" see text

Sample No.	R	A	B	C	P	USL	MDC	Aggregate	Climate	Service
1	36	48	65	63	4	18	470	Round	Wet	Satisfactory
2	34	55	42	61	5	15	510	"		"
3	37	63	50	50	13	14	410	Intermed.	"	"
4	36	67	56	58	13	17	430	"	"	"
5	37	65	41	59	NP	15	710	"	"	"
6	37	67	46	57	2	15	450	"	"	"
7	52	68	50	43	NP	17	670	"	"	"
8	60	73	48	51	16	14	1200	"	"	"
9	31	69	65	68	4	15	640	"	"	"
10	65	57	55	73	8	21	460	"	"	"
11	34	64	61	61	2	17	510	Angular	"	"
12	37	65	54	58	4	17	450	"	"	"
13	56	52	58	70	4	18	770	"	"	"
14	30	49	55	40	NP	18	610	"	"	"
15	15	56	47	51	9	18	430	Fine	"	"
16	20	43	45	65	10	21	510	"	"	"
17	16	59	52	57	13	18	510	"	"	"
18	25	53	47	56	10	21	580	"	"	"
19	20	57	56	63	5	17	480	"	"	"
20	23	52	50	54	9	22	700	"	"	"
21	16	62	47	65	11	15	450	"	"	"
22	13	51	45	46	8	17	530	"	"	"
23	30	47	55	67	16	22	390	"	"	"
24	7	60	40	62	12	20	410	Sand-Clay	"	"
25	55	69	48	53	2	16	580	Round	Medium	"

[5] Tables 1, 2, and 3 are abridged. The entire tables showing analyses of 775 samples are available in mimeographed form at the office of the Highway Research Board.

TABLE 2[5]

SAND-CLAYS AND GRAVEL-SAND-CLAYS UNDER BITUMINOUS SURFACE TREATMENT

Sample No.	R	A	B	C	P	USL	MDC	Aggregate	Climate	Service
253	31	66	43	58	7	15	380	Round	Wet	Sound
254	32	66	40	56	NP	15	880	"	"	"
255	59	58	39	50	NP	18	480	"	"	"
256	44	41	73	60	9	18	520	"	"	"
257	25	55	46	65	6	18	430	"	"	"
258	39	71	51	50	9	15	860	Intermed.	"	"
259	43	75	40	47	NP	15	1100	"	"	"
260	31	66	51	44	2	20	900	"	"	"
261	32	75	40	53	3	15	780	"	"	"
262	25	70	36	52	4	16	400	"	"	"
263	32	83	43	56	9	16	790	"	"	"
264	45	67	41	59	2	17	680	"	"	"
265	25	72	63	67	6	16	600	"	"	"
266	41	81	47	55	10	18	550	"	"	"
267	45	75	43	55	7	18	1000	"	"	"
268	37	62	50	42	8	14	900	"	"	"
269	50	53	47	52	6	18	440	Angular	"	"
270	31	46	52	40	NP	18	470	"	"	"
271	53	56	37	48	NP	17	550	"	"	"
272	23	64	57	49	NP	19	960	Fine	"	"
273	25	44	48	55	6	20	720	"	"	"
274	49	44	48	48	NP	16	390	Round	Medium	"
275	29	59	49	63	9	16	890	"	"	"
276	31	43	64	62	8	18	890	"	"	"
277	61	56	57	49	8	19	520	"	"	"
278	38	54	52	38	2	19	1000	"	"	"

[5] Tables 1, 2, and 3 are abridged. The entire tables showing analyses of 775 samples are available in mimeographed form at the office of the Highway Research Board.

uppermost and using the following abbreviations:

T = Light bituminous surface treatment.
M = Bituminous premix or roadmix.
P = Bituminous penetration.
G = Materials depending on the combined friction and cohesion of a well graded soil matrix.
I = Materials depending on the interlock of hard angular agregate.
S = Sandstone courses (see last paragraph of BASIS AND SCOPE OF INVESTIGATION).

were more effective per inch of thickness. Good correlation was obtained by computing an "effective cover" by adding to the actual thickness of material without beam strength the actual thickness multiplied by a numerical factor for the layers with some beam strength. This factor was taken as two for solid dense bituminous courses and as one and one-half for semi-dense bituminous courses, and was assessed by judgement at time of sampling. Logically bituminous courses of very open grading on this basis should be taken as equivalent to their actual thickness but no thick

TABLE 3[5]
SUBGRADE SOILS ETC.

R, A, B, C, and USL as in Table 1 and 2
LS = % Lin. Shrinkage from LL (on Pass No. 7 B.S.)
MDW = Max. Dry Weight by Proctor Compaction
Effect Ins. = Total Effective Thickness of Pavement etc. over sample (see Letterpress for Method of Computation and Meaning of Abbreviations)
For "Climate," "Traffic," and Table Divisions see text.

SECTION A—SOUND PAVEMENTS

Sample No.	Mech. Anal. Ratios				Soil Moisture Relations				By Formulae Ins.			Type of Pavement	Effect Ins.	Climate	Traffic
	R	A	B	C	P	USL	LS	MDW	Grad.	SMR	Final				
422	1	99	80	62	8	16	7	117	10	8	9	T.G.	11	Wet	Heavy
423	23	94	21	35	NP	17	0	114	7	7	7	T.G.	8	"	"
424	3	98	38	46	3	17	4	116	6	8	7	M.G.	11	"	"
425	9	89	73	61	13	23	9	122	7	9	8	T.G	9	"	"
426	7	90	79	32	NP	23	3	119	8	8	8	T.G.	11	"	"
427	1	68	46	29	NP	15	0	122	3	5	4	M.G.	6	"	"
428	1	97	54	38	NP	16	4	124	6	6	6	M.G.	8	"	"
429	3	95	79	41	9	18	5	116	9	8	9	M.G.	11	"	"
430	24	85	92	68	28	29	12	112	11	13	12	M.I.	14	"	"
431	2	89	92	78	25	25	11	110	14	13	13	M.I	13	"	"
432	3	96	45	42	NP	18	1	114		8	8	T.G.	11	"	"
433	1	90	33	43	NP	18	3	122	6	6	6	M.I.	12	"	"
434	0	90	70	56	7	18	4	115	7	8	7	T.G.I	12	"	"
435	0	100	98	74	15	21	16	111	15	13	14	T.G.I.	13	"	"
436	10	92	83	57	14	15	8	125	9	7	8	T.G.S.	16	"	"
437	16	84	63	62	10	15	11	119		9	9	P.S.	11	"	"
438	41	61	67	64	11	18	6	129		6	6	M.G.	7	"	"
439	0	97	78	48	3	17	2	116	8	7	8	T.I.S	12	"	"
440	3	90	59	58	12	15	6	117	5	8	7	T.G.	7	"	"
441	31	78	29	54	NP	17	0	132	3	3	3	M.	5	"	"
442	0	98	91	73	40	20	10	104	15	13	14	M.I.S.	19	"	"
443	25	55	46	65	6	18	1	132	0	3	2	T.	1	"	"
444	0	99	76	60	18	21	8	101		13	13	P.I.	16	"	"
445	0	98	39	53	3	22	4	114		9	9	P.I.	14	"	"

[5] Tables 1, 2, and 3 are abridged. The entire tables showing analysis of 775 samples are available in mimeographed form at the office of the Highway Research Board.

It was not certain when the work started what were the relative distributing values of the various types of construction. The data collected showed that there was little apparent difference between all materials without appreciable beam strength that distribute the load almost entirely by arching action (i.e. classes G, I, and S above) but that heavy solid bituminous surfaces (classes M and P above) course of this type was actually found. In Table 3 the "effective covers" so computed are given. Subsequent to the adoption of this practice it was found that similar conclusions had also been reached by other workers.

In many cases it is necessary to consider the distributing effect of soil sub-bases, etc. There is no data available, but if the soil is of a sandy nature and not over-stressed it will

TABLE 4

SHALES AND OTHER SOFT ROCK

Comparison of Materials produced by breaking up and natural weathering in the road (upper figures of each pair) and (or) service records with product made by crushing and artificially weathering samples from quarries (lower figures of each pair). For abbreviations used in column headings see Table 1.

No.	R	A	B	C	P	USL	Service Record
776	Not used without blending 92	 30	 43	 48	 N.P.	 25	Requires blending to secure compaction
777	Not used without blending 88	 42	 48	 50	 N.P.	 23	Requires blending to secure compaction
778	Not used without blending 80	 28	 50	 57	 N.P.	 21	Requires blending to secure compaction
779	Not used without blending 80	 49	 55	 68	 9	 19	Requires blending to secure compaction
780	Not used without blending 75	 41	 59	 60	 9	 18	Requires blending to secure compaction
781	Not used without blending 64	 31	 65	 75	 16	 17	Requires blending to secure compaction
782	71 72	57 47	73 69	60 61	14 13	18 18	Slightly harsh
783	70 66	55 53	62 59	67 70	7 5	19 18	Satisfactory both unsurfaced and with bituminous treatment
784	66 70	58 60	73 74	74 70	12 10	19 19	Satisfactory unsurfaced
785	49 62	59 61	68 67	70 68	8 7	19 19	Failed under bituminous treatment
786	37 38	48 45	40 35	52 50	3 3	18 19	Satisfactory under bituminous treatment
787[a]	36 57	56 45	54 53	50 46	4 7	20 19	Satisfactory unsurfaced
788	59 52	58 39	60 61	66 63	15 16	20 19	Failed under bituminous treatment
789[a]	25 57	68 65	80 76	67 73	10 9	20 19	Unsatisfactory unsurfaced
790[b]	52 22	67 66	79 82	74 76	12 13	20 21	Subgrade only
791[c]	33 26	76 76	83 80	60 61	11 10	21 20	Subgrade only
792	Not available 60	 72	 76	 69	 9	 21	Pavement constructed from this quarry failed and replaced
793	Not available 43	 85	 95	 80	 26	 23	Pavement constructed from this quarry failed and replaced
794	Not available 4	 92	 81	 75	 22	 21	Pavement constructed from this quarry failed and replaced
795	Not located 30	 67	 61	 66	 25	 17	Old quarry, abandoned because found unsatisfactory
796	Not located 20	 64	 73	 75	 6	 19	Old quarry, abandoned because found unsatisfactory
797[d]	Not located 65	 53	 54	 48	 N.P.	 30	Old quarry, abandoned because found unsatisfactory

[a] Typical of materials which are still breaking down at 10 wet-dry cycles.
[b] Material from road subjected to only about 4 months natural weathering prior to tests.
[c] Material from road subjected to less than 12 months natural weathering prior to tests
[d] Fault here is mica (rock is a micaceous schist).

distribute compressive loads by arching in much the same way as gravel etc. and may be counted as equivalent thereto. The few cases met with in the investigation were so treated. If the soil is not sandy its distributing capacity will probably be lower, but the necessary

factor was not determined in the investigation. In practice the Department makes an arbitrary reduction.

Table 4.—The data obtained for shales, etc. where it was possible to compare the product from the accelerated weathering test with the material that had resulted by natural weathering in an old pavement are given in Table 4. Only a limited number of cases could be found in which it was certain the road was constructed entirely of material from one particular quarry without additions from other sources. In these cases there was no reliable record of which length of the road had been constructed from any given part of the quarry; it was therefore necessary to average the test results for several samples from both road and quarry to overcome natural variations in the materials and obtain comparable figures. Also included in this table is a selection of typical figures for a number of materials known to produce insufficient fines or to disintegrate excessively if used for pavement con-

TABLE 5
SANDSTONE, ETC.

No.	Geological Series	Comp. Strength[a]				Wet Brit. Crush. Test Loss		Water Adsorp.	Weight	Service Record	
		Perp. to Bed		Parallel to Bed		¾"–½"	½"–⅜"			Under Light Bitum. Surface	Under Heavy Surface Course
		Dry	Wet	Dry	Wet						
		thousands of pounds per sq. in				*percent*		*percent*	*lb. per cu.ft.*		
798	Hawkesbury	4.6	2.6	4.4	2.2	54	55	7	126	Satis.	Satis.
799	"	7.1	2.0	6.8	0.9	60	55	6	132	Unsatis	"
800	"	6.1	4.6	6.1	4.3	51	51	5	140	Satis.	"
801	"	3.9	0.6	3.9	0.5	77	76	7	125	Unsatis.	Unsatis
802	"	6.6	2.4	6.2	2.1	58	59	4	140	Satis.	Satis.
803[b]	Upper Marine	10.7	8.5	10.7	8.2	39	37	6	138	"	"
804[d]	" C.M.	12.0	12.0	11.8	11.4	24	23	2	158	"	"
805	Narrabeen	14.3	6.8	13.9	6.1	34	36	5	150	Unknown	"
806	"	8.0	2.8	7.5	2.2	49	46	6	134	"	"
807	"	8.2	3.9	8.6	4.3	43	43	6	131	Satis.	"
808	Clarence	9.6	8.2	9.6	8.5	38	37	2	152	"	"
809[d]	"	13.8	12.0	13.0	11.0	30	29	2	154	"	"
810	"	9.6	8.5	9.3	8.3	38	37	2	156	"	"
811	"	9.6	9.3	9.3	8.5	52	53	2	154	Unknown	"
812[c]	Wianamatta	10.0	5.0	8.9	3.9	45	41	6	145	Satis.	"
813[d]	Narrabeen	16.0	5.6	13.0	5.0	37	38	4	148	"	"
814	Hawkesbury	4.7	2.8	3.4	2.2	51	50	4	138	"	"
815	"	7.1	2.3	3.8	1.5	50	49	1	140	"	"
816	"	6.9	3.7	5.8	2.9	52	48	2	141	"	"
817	"	6.8	3.4	5.4	2.9	47	50	5	139	Unknown	"
818	"	2.9	1.3	2.5	1.1	60	63	7	130	Unsatis.	Unsatis.
819	"	3.2	2.4	2.4	2.0	58	58	5	135	"	"
820	"	3.1	2.3	2.7	1.9	64	64	6	134	"	Unknown
821[c]	Wianamatta	5.6	3.2	5.1	3.2	42	40	6	138	Unknown	Satis.
822	Hawkesbury	7.1	5.4	4.6	4.3	42	39	6	136	Satis	Satis.
823	Clarence	Too Soft				65	63	—	—	Unsatis.	Unsatis.
824	Hawkesbury	"	"			70	68	—	—	"	"
825	"	Too variable				68	64	—	—	"	Unknown
826	"	4.2	2.8	3.7	2.7	55	57	5	135	Satis.	Satis.
827	"	5.6	2.2	4.6	1.9	51	54	6	140	"	"
828	"	4.4	2.5	3.8	1.9	51	53	7	140	"	"
829	"	4.3	2.2	3.3	1.8	53	56	7	131	"	"
830	"	4.2	1.4	2.8	0.9	61	62	7	131	Unsatis.	Unsatis.

[a] 2-in. dia. cylinders 2 in long.
[b] A Tuff.
[c] Very fine argillaceous sandstones.
[d] 804 reported as "too hard"; 809 and 813 as "rather hard" for "sandstone" type of construction

struction

struction. Naturally old pavements of such materials do not exist.

Table 5.—Table 5 gives the data obtained for sandstones.

THE SYSTEM DEVELOPED

The data collected as described above were analysed and methods for assessment of pavement thickness and selection of materials developed to give the best practicable agreement with service records. This was a progressive process, the formulae and rules being first based on the early data available, about 20 percent of final number of samples, and modified as necessary to secure better agreement as more and more samples were tested. The rules and formulae adopted in the final system are set out separately, in a form ready for use, as Appendix B. Their development and theoretical basis are as follows:

The data clearly confirm the theory that, with a soil containing only what may be called "normal constituents", i.e. solid particles of not too unusual a shape, mechanical analysis only is sufficient to make a very fair estimate of its quality. It is necessary, however, to check against the presence of what may be called "adverse constituents", i.e. particles that are spongy or elastic and (or) of markedly flaky or fibrous nature, the effect of which is to interfere with the packing of the soil particles, increase voids, and reduce both the cohesion and the internal friction. Principal offenders are peaty organic matter and mica.

If a soil contains an appreciable proportion of adverse constituents, any rules based on grading only fail. There are also a few heavy clay soils which are so flocculated and (or) aggregated that thorough dispersion is impracticable for routine work. In both cases it is necessary to adopt other methods. For each type of material the system provides for

1. A numerical estimate of the soil quality from the grading only, unless failure of dispersion prevents this, and

2. A check as to the possible presence of adverse constituents from other tests, then

3. Assessment of quality on the basis of (1) and (2).

It is more convenient in explaining the system to consider first the rules and formulae for pavement thickness determination (see Table 3 for data). Considering first the grading method; it was evident from the soil and the pavement material data that, with soils free from adverse constituents, no pavement was required on soils following maximum density grading from No. 7 BS sieve to dust, and the thickness required increased with the departure from maximum density grading; also that the effect of material retained on No. 7 BS sieve was to slightly improve the strength and reduce the effect of departures from the ideal grading. These conclusions are quite in accord with theory.

Now the most convenient form of the maximum density grading law for a wide range of sizes is that put forward by Wilhelmi which may be written:

$$\left(\frac{W}{w}\right)^n = \frac{A}{a}$$

Where W = total weight less than linear dimension A

w = total weight less than linear dimension a[6]

n has any value from 2 to 3.

From the analysis of the early results it was decided that, with the wide range of sizes present in soils, it was sufficient to adopt a ratio of about five between the linear dimensions of the successive size divisions. Now in Wilhelmi's rule the average value of n is $2\frac{1}{2}$; also 2 to the $2\frac{1}{2}$ power equals 5.6 approximately. With this ratio for A/a the mean value of w/W for maximum density is 50 percent, and the extreme values 50 ± 8 percent approximately or, say 40 to 60 percent. These numbers are very convenient for use, making the formulae easy to remember and apply and greatly assisting quick approximate interpretations. Work was therefore standardised on this ratio for A/a.

The departure outside the range 40–60 percent for any weight ratio, on the cumulative passing system, between successive sizes is then a measure of the departure from the maximum density grading over this range of size. Whether the relation between departures and thicknesses required was linear and whether simple addition of departures was

[6] The usual assumption that weight may be substituted for volume, i.e. that specific gravities of different sized particles do not materially differ, is made here.

suitable had still to be determined. Again, the ratios could theoretically be determined right down to colloid sizes but this is very laborious, and in fact impracticable and uneconomical for routine applications. The necessary and usual procedure is to carry the sizing determination only as far down the scale of size as is required to give the accuracy need for the work in hand, substituting some indirect test (e.g. PL and LL tests) for the finer sizing.

From an analysis of the data it was found that correlation, sufficiently accurate for practical use, could be obtained by sizing down to one standard 5.6 ratio below the No. 200 sieve, or to 0.0135 mm. and using the PI to assess the effect of smaller sizes. With this rather large size ratio the error due to the different basis of measurement in the sieve and sedimentation methods does not seem to be important. The formula so developed is simple to use, being linear and additive in form, except that a small modification is made in the case of sands which can easily be explained by theoretical reasoning.

Should other diversions be adopted over this range of sizes it would be simple and safe to compute other maximum density ranges for the ratios adopted and factors for departures therefrom to give equivalent formulae. They would, however, be more complex and more difficult to use. It would not be desirable to extend the method to a wider range of sizes without further correlation with service as, from theoretical considerations, it is probable that the numerical importance of given departures in ratio decreases for very small particle sizes. The data show that the error from ignoring this effect is not significant in practice for the particular ranges of size considered. It may also be desirable to make an adjustment, which would be small, if a different dispersion procedure is used for the mechanical analysis.

The check against adverse constituents and ineffective dispersion[7] is, in the case of pavement thickness, made by computing an alternative figure by a "point-score" formula based on other tests. Several such formulae had been developed by previous workers but none was found to fit the data collected satisfactorily for the whole of the wide range of soil types included. By a statistical analysis a new point-score formula, called herein the Soil Moisture Relations rule, was developed based on three tests only:

1. Maximum dry weight by Proctor compaction test. It is evident that this is a measure of the combined effects of departures from maximum density grading and the interference of adverse constituents with the packing. It is given by far the largest effective weight in the formula.
2. What may be called the "Upper Solid Limit", i.e. the PL if plastic or LL if nonplastic. A separate investigation by mixing different size fractions indicated that this test is an approximate measure of the voids under very different conditions of moisture and compaction.
3. The linear shrinkage from LL on the portion passing No. 7. This is a good measure of the harmful effect of excess clay and colloids.

Except for sands, where an increase in voids does not necessarily imply a decrease in strength and this interferes with the Soil Moisture Relations rule, the two formulae give values in reasonable agreement if the soil is practically free from adverse constituents, but, if the soil contains a considerable proportion of adverse constituents, the value for pavement thickness computed from the grading formula will be much the lower of the two.

In the case of gravel to receive bituminous surface treatment, the grading rule also applies. The requirement is a very close approximation to maximum density grading with practically no adverse constituents. The method of checking for adverse constituents has to be modified as the second pavement thickness formulae is not very suitable in this case. The reason for this is that these materials are high in sand and often in gravel passing $\frac{3}{16}$ in. sieve, which are likely to vary somewhat in specific gravity, and the materials as a whole have high maximum dry weights. The formula is therefore very sensitive to variations in specific gravity of the soil particles. On the other hand the maximum dry weight in the vicinity of maximum density grading varies but little with changes of grading sufficient to markedly affect the value given by the grading rule. The difference between the two rules is therefore not a satis-

[7] The latter is also normally noted during test and reported.

factory means of detecting the small quantities of adverse constituents now in question. Satisfactory correlation with service is, however, obtained by imposing limits for PI, upper solid limit, and maximum dry compressive strength. Where the pavement has to carry traffic for some time before surfacing, the dry compressive strength should be as for an unsurfaced pavement. If it is not to be exposed to general traffic until surfaced, a lower dry strength will meet the needs of the constructional operations. The minimum figure adopted for the latter case was determined by observation and tests on works in progress. In the case of gravels not to be surfaced, rather more clayey materials are permissible, and in fact desirable. While satisfactory service is given with materials having grading ratios as low as 40, the tendency in service is for all ratios to fall by loss of fines as dust under traffic and maintenance operations. Moreover the complete disintegration of the material from deficiency of fines is the worst form of failure.

Shale, etc. is comprised of all those soft, laminated or much jointed rocks that are broken up in compacting and by the action of traffic and weather to yield a graded pavement material. Here the requirement is that the product should be equivalent to a gravel or whatever other type of material the broken up rock is to replace. The method developed is to subject crushed material to alternate wet-dry cycles and then to compaction and subject the product of this accelerated weathering test to the tests appropriate for the proposed use. Originally the method was to subject several portions to varying numbers of wet-dry cycles, then test the first showing no further breakdown. Analysis of the results for materials from this investigation and new materials proposed for use, about 160 different sources in all, showed, however, that in well over 90 percent of these cases no further breakdown occurred after ten cycles. In the remainder the greater part of the breakdown was completed by ten cycles and the decision as to acceptance or rejection would not have been affected by further breakdown after ten cycles. These materials comprised shales of all degrees of induration, from soft shales that deteriorated to clay in three cycles to materials that had Los Angeles losses of the order of twenty and showed no change at all on wetting and drying, transition types of shale verging on sandstone and on slate, also schist, phyllite, chert, serpentine, tuff, and various partially weathered igneous rocks. It was therefore decided to take only one portion and test it after ten wet-dry cycles.

Sandstone, etc. comprises, in addition to sandstone itself, a few other granular textured rather soft rocks that are resistant to weathering and can be used in pavements when protected from wear by upper courses and (or) bituminous surfaces. In their construction large gauge stone is wedged and ground together by heavy rolling, surface voids are then filled by rolling in smaller gauges and fines. These materials generally have much lower strengths wet than dry and a minimum wet strength is the principal requirement. Both loss by British crushing test and compressive strength of individual specimens gave good correlation with service. For practical reasons the British crushing test is preferred, being an "averaging" method of test, it is much less dependent on judgment in taking of samples and in selection of pieces for test. The correlation is good for both sizes but is slightly better for the larger size tried ($\frac{3}{4}$ in. to $\frac{1}{2}$ in.) which was therefore adopted as the standard. While further improvement might result with even larger gauges there are serious practical objections to the large samples then required. Weight and water absorption were also tested but do not seem to be significant. As little information is available on sandstone testing all figures are included in Table 5.

RELIABILITY AND ACCURACY

Practically none of the data now presented is of a type suitable for the normal statistical investigation of errors. Section D of Table 3 is the only part of suitable form. This is only a small, though important, section of the data on which thickness determination is based, and it would be unreasonable to assess the overall accuracy from this portion only.

Considering the data in Table 3 as a whole, it seems that the probable overall error does not exceed one inch of pavement thickness. Overall error includes errors in judgment and measurement in the field, errors due to undetected abnormal drainage and loading changes,

etc., and errors due to rounding off measured thicknesses of actual pavements to nearest inch, as well as errors in the actual testing and in the system itself. It seems, therefore, that the probable error in testing and using these formulae now proposed is less than one inch in pavement thickness. This is comparable with the normal variations in the actual construction of most flexible pavements and any reasonable practical system of sampling must result in comparable or greater errors from failure of the samples taken to represent the full variation in natural soils, etc., i.e. the proposed system seems to be of adequate accuracy for the work for which it is intended.

The Soil Moisture Relations Rule holds, with sufficient accuracy for practical use, for the full range of materials for which it is intended. It gives too high an answer, however, for materials suitable for top courses; this is evident from the data in Table 3. The effect is marked for materials requiring little or no cover but disappears if Effective Cover required exceeds 3 or 4 inches. This implies that this simple linear relation between test figures and cover required does not hold as maximum density grading is approached, which would not be unexpected on theoretical grounds. The rule could be adjusted by abandoning the linear form but the complication thus introduced is not considered warranted. Top course materials are judged on a different basis for other reasons and the cover provided over materials not quite complying with top course requirements is governed by the practical minimum thickness of pavement courses.

There is a further factor which contributes to the strength and stability of certain materials, which is not very common, and which has been ignored in the investigation. This is the chemical or physico-chemical cementing action of certain components. Thus clean crushed limestone, coral, etc., make good pavements even when the grading departs markedly from the maximum density limits. The increased stability from the cementing action of dissolved and redeposited calcium carbonate is responsible but silt, clay, etc. interfere with this action. It cannot be relied upon with impure or dirty materials and is generally non-existent in nodular limestone gravels where the matrix is soil of normal chemical composition. There is also evidence that iron hydroxides and basic iron carbonates give a cementing action that may increase the stability of natural materials. This probably accounts for the discrepancies noted in samples 495 and 497 which contained approximately 30 percent and 25 percent respectively of free iron hydroxides, etc. It is also quite probably responsible for a proportion of the inconsistencies in the "Intermediate" sections of Tables 1 and 2. Concretionary lateritic gravels are widely used in New South Wales and were so classed. Some of them appear to give better service than their test figures would indicate but the action is unreliable as others, similarly graded and superficially similar, give only the type of service that would be expected from the tests. No test has yet been found or devised to assess this cementing action, or even to determine whether or not it exists in any given material. Again, certain soft sandstones are known which pulverize under compaction and traffic to yield a badly graded material that still sets firm and gives fair service as an unsurfaced pavement. This is doubtless due to the original cementing agents binding the sandstones, which in many cases are the calcium and iron compounds previously mentioned.

The investigation showed that the simple Public Roads Administration tests correlated remarkably closely with the properties and qualities of soils. This is believed to be the basic reason that permits the suggested reduction in the number of tests. It is indeed surprising, considering the variations met with in soils, that the correlation is so consistently satisfactory. The few wide misses in Table 3 are considered to be probably due to errors in observation or judgment in the field, undetected abnormal drainage, cementing ingredients, or insufficient time for failures to develop, rather than to failure in the correlation.

It might be mentioned here that, in the normal case of plastic soils, pavement thickness selection is really a problem in economics. There is not a definite thickness below which a pavement fails utterly and above which a pavement stands without signs of failing. Serious and wide-spread failures occur with small thicknesses and the rate of development of failures and the magnitude of the areas affected decrease continually with increasing

thickness, at first rapidly, then more and more slowly. If the capitalised cost of repair of failures could be estimated for varying thicknesses and a curve of total cost drawn, it would have a minimum at the point where reduction in cost of repairs just balances the cost of extra initial thickness. The thickness corresponding to this minimum should normally be used, but the curve, as is usual in such cases, would be very flat near this minimum and a reasonable approximation to this point is sufficient for practical applications. In the case of peaty soils, though mode of failure is different, similar considerations apply.

This suggests that with these soils the economic thickness may not be entirely independent of the frequency of loading. The data do not exclude this possibility but suggest that the variation is not large with the traffic volumes experienced on the roads studied in the investigation. It is very difficult to secure suitable data on this point because of the paucity of data on traffic loads and frequencies and because of the marked correlation between maximum wheel load and total tonnage moved. The rather arbitrary division of New South Wales into heavy and normal traffic zones seems to be the best local solution in the present state of knowledge, but further investigation of the correlation between the frequency of application of the heavy loads and the economic thickness is desirable.

In the case of sands the conditions are very different. Total failure may result from a single excess load and it is probable the thickness is determined by the greatest wheel load applied irrespective of repetitions. While similar traffic zones were used in the investigation, the greatest wheel load is probably not the same as the normal maximum load, and arises from altogether different traffic. Its exact magnitude is somewhat uncertain in either case. Failures on sand being sudden and complete, pavements are immediately strengthened. No serious cases of failure on sand could be found still in existance. Pavement thicknesses are not large and the penalty for deficient thickness is heavy, so the practical problem is to determine a safe minimum thickness.

The requirements proposed for graded pavement materials give reasonable correlation with the service records bearing in mind the difficulties due to low average age of surfaced gravel, etc. the need to classify unsurfaced materials as satisfactory or otherwise from a single inspection, in a wet or in a dry season, assisted by the local engineer's memory of its behavior under different conditions, and the probability of cementing action interfering in a few cases. There was also the tendency of engineers to class the best of the unsurfaced materials in a given locality as good, even if only fair or poor by general standards, and so on. It was hoped by the division into fair and unsatisfactory classes to determine the relative importance of various departures from the desirable limits. Results are not sufficiently consistent for this to be done with any certainty. It appears that if a material fails to meet the desirable limits, the degree of service rendered depends on traffic volume, climate, and local drainage, as well as on the nature and amount of the departures in test figures.

In the case of shales, etc., the principal differences between the products of natural and artificial weathering are in the coarser ratios R and A. The grading of these coarser portions could obviously be greatly influenced by the degree and nature of the crushing or of the mechanical fracture during the compaction etc. in the pavement. The ratios B and C and the LL and PL figures differ little throughout, even if weathering is incomplete in either case, showing that the finer fractions of the two products are very similar. The effect of the variations on the assessment of quality is small and the artificial weathering test now proposed can be accepted as a sound guide to actual performance.

It is possible to prepare materials in which the grading ratios now proposed are not a guide to quality (e.g. a material with no fraction between No. 7 and No. 28 BS sieve, nor between No. 52 and No. 200 BS but much between No. 28 and No. 52 BS) but the chance of finding such materials in nature is extremely small. Again it is possible to find unusual combinations for which other limits fail, but these cases are rare and, when they do occur, the other samples tested normally disclose the faults. All deposits of natural materials are more or less variable. In using such deposits for pavement construction it is essential to arrange for mixing from various parts and layers to equalize the variations and produce a more uniform material. The main argument

for simplified testing procedures, such as that now proposed, is that they make it practical to test more samples and thus determine in greater detail the amount of distribution of the variations, so that the work can be properly planned and controlled. Again the numerical valuation of the mixed material determined from the simplified tests of a number of samples is a more probable approximation to the real value than the corresponding valuation deduced from a lesser number of samples, no matter in what detail, or to what precision, the latter are tested.

No special comment on the sandstone tests is necessary. In sandstone sampling is the most difficult portion of practical control and the most probable source of error. Satisfactory sampling of stratified rocks, especially the coarser types, is very difficult.

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APPENDIX A

TESTING PROCEDURES FOR FLEXIBLE PAVEMENT MATERIALS

A. SOILS ETC.

Subgrade, sub-base, and lower course materials are all tested as soils. The tests conducted are:

Mechanical Analysis
Linear Shrinkage from LL
LL and PL
Proctor Compaction

1. *Preparation, etc.*
(a) Dry whole sample, by gentle warming on hot plate if necessary, till in suitable condition to handle.
(b) Pass whole sample through ¾-in. square sieve, breaking up lumps as required and brushing adhering material from aggregate retained.
(c) Weigh portion retained and portion passing; compute oversize (retained ¾ in.) as percentage of whole sample (neglecting the moisture content of sample).
Note: Effect of residual moisture is negligible here and in (e) following.
(d) Take a 2000-g. portion from material passing ¾ in.-hold butt.
(e) Pass 2000-g. portion through ⅜-in.[1] 3/16-in.[1] and No. 7 B.S. sieves rubbing down as required with pestle and mortar. Weigh fractions and compute grading down to No 7 B.S. sieve (neglecting moisture content).
(f) Reduce portion passing No. 7 to about 200 g. for linear shrinkage and mechanical analysis.
(g) Pass sufficient of balance passing No. 7 through No. 36 B.S. sieve to yield about 50 g. for LL and PL test (rubbing down as required).
(h) Pass sufficient of butt from (d) above through 3/16-in. square sieve to give at least 2,000 g. for Proctor test.

2. *Mechanical Analysis of Material Passing No. 7 Sieve.*
(a) Dry sufficient of pass 7 to constant weight at 105 C.
(b) Take 50 g. of dry material and boil with 500 ml. of water for one hour, stirring briskly about every 10 min.
(c) Cool and transfer soil and water to 1,000 ml. beaker (diameter about 10 cm.) marked at 3 cm. and 11 cm. from base. Adjust volume to top mark and add ammonia to make solution about 0.2 percent of strong ammonia.
(d) Stir with rubber-tipped glass rod, working sand on bottom towards pouring side and finishing with to and fro motion (to avoid rotation on ceasing). Stand as follows:—

[1] Though not included in rules grading on these sieves is of interest to Field Engineers and is reported.

Water temperature	70 F.	75 F.	80 F.
Time .	8 min.	7½ min.	7 min.

Carefully pour off liquid (without turbulence that would lift settled material) till it will stand at lower mark.

Make up to top mark with 0.2 percent ammonia and repeat process until liquid poured off is clear.

Notes: Twenty seconds for filling and stirring and 20 seconds pouring allows over 20 samples to be placed in line, all stirred, then all poured alternately by one operator, or all stirred by one and poured by a second.

Incomplete dispersion in initial pours is not important if continual dilution with dilute ammonia and stirring will eventually complete the dispersion. In certain badly flocculated heavy clay soils the method fails. This is evident from the behaviour of the sample undergoing test and, for routine tests, this fact only is reported.[2] In heavy clay soils first two pours are best delayed (say two min.) because of the possibility of the coarser particles being slightly retarded in their fall by interference.

(e) Stand beaker after last pour at angle of about 45 deg. for 5 min. and pour off balance of liquid. Dry residue on hot plate and weigh. Loss is material less than 0.0135 mm.

(f) Screen residue dry through No. 36 and No. 200 B.S. sieve. Weigh fractions and compute grading.

3. *Linear Shrinkage.*
Bring sufficient of pass 7 material to its LL or slightly above.[3] Place in a lightly greased mould and strike off. Allow to air dry slowly (to prevent distortion and cracking) and complete drying in oven at 105 C. Measure shrinkage and report as percentage of original length.
Note: A very convenient form of mould is a trough of semi-cylindical section 10 in. long, (split brass or copper 1-in. int. dia. tubing brazed on ends), the shrinkage being read directly in percent by measuring space in mould with a rule divided in inches and tenths.

4. *LL and PL*
Standard method abbreviated to suit the accuracy desired (1 percent of dry weight of soil).
Notes: This standard of accuracy permits the elimination of refinements such as flow curves and cooling in desiccators. To expedite work the passing 36 material is predried to constant weight at 105 C. and the LL determined volumetrically (volume of water by burette to weight of dry soil.) With practice this can be determined to 1 percent sufficiently rapidly to avoid significant error by evaporation or absorption of water. The PL is determined gravimetrically by drying out the sample at 105 C.

The ASTM specifications are not complete for these tests and allow some scope for personal interpretation by the operator. Tests on the same sample by different operators (both in the one laboratory and in the different laboratories of various State Road Authorities of Australia) have shown that these differences in interpretation may produce differences greater than 1 percent of dry weight of soil in both tests. These differences are greater than the differences obtained by one operator by the standard and by the simplified methods.

Comparison tests on undried samples have shown that the pre-drying reduces both the PL and LL values. Reduction is very small for limits of the order 20 or less, but increases roughly in proportion to the excess over 20. The difference in no case was sufficient to make any practical difference to the pavement thickness computed from the formulae.

5. *Proctor Compaction.*
Standard P.R.A. method. (Max. dry weight to be determined to nearest lb. per cu. ft.)

B. GRAVEL, ETC.

Gravel-sand-clay, sand-clay and other natural or artificial graded materials proposed for pavement upper courses (with or without bituminous surface treatment) are tested as gravels. The tests conducted are:—

Mechanical Analysis
LL and PL
Max. Dry Compressive Strength

Samples from compacted pavements or samples from deposits in which the aggregate is hard and stable are prepared and tested as received. Samples from deposits etc. in which the aggregate is soft or brittle are compacted to about the same degree as it would be in road

[2] Numerical results could be obtained by different dispersion procedures, but probable error will not be reduced appreciably as compared with the S.M.R. rule only as latter is reasonably reliable for such soils.

[3] Method of test of LL same as for standard test on pass 36 (or 40 U.S.) material. The old hand method is of sufficient accuracy and convenient for checking water content.

prior to preparation and testing as described below. Special treatment may be given in some cases before testing, e.g. concreted laterite and weak conglomerates may be best crushed or broken (as they would be in the road). If resistance to weather is doubted an accelerated weathering test may be applied (see shales, etc. following).

1. *Preparation, etc.*
   (a) to (g) as for soil except that only not less than 50 g. dry material passing No. 7 sieve are required in (f). (h) Take three portions each 1,000 g. from butt in (d) for dry comp. test.
2. *Mechanical Analysis of Pass 7 Material.*
   As for soil. (Dispersion never fails if the material even approaches the requirements set out in Appendix B).
3. *LL and PL.*
   As for soil.
4. *Max. Dry Compression Strength.*

(a) Bring to approximate optimum moisture content for special compaction method used.

(b) Pack sufficient material in a greased $2\frac{3}{4}$-in. split cube mould with removable top section and steel packing block (as used for British Standard cement mortar test), and compact with 450 blows of a 4.4-lb. hammer falling 8 in. (compacted height to be 3 in. $\pm \frac{1}{4}$ in.) Strike off excess and smooth the upper face.
Note: Three cycles in standard Bohme hammer machine used for mortar test gives this compaction.

(c) Air dry cube overnight and complete drying to constant weight in oven at 105 C.

(d) Determine compression strength of dry cube in pounds per square inch (test between opposite sides—not top and bottom—to give good bearing).
Note: If a mechanical compacting machine is available, cubes should be moulded at say three different moisture contents and the highest strength only reported. If hand compaction is used a single cube may first be made as the moulding water can generally be gauged by judgment if the operator is experienced. In this case if strength is below requirements and other test results satisfactory other cubes should then be moulded with different moisture contents, tested and the highest figure reported.

C. SHALES, ETC.

This test is applied to shales and other soft laminated or jointed rocks to be broken up during the winning and compaction and by the action of traffic and weather to yield graded pavement materials, also to subgrades of shale etc. that are expected to deteriorate by weathering. It comprises an accelerated weathering test by alternate wetting and drying followed by compaction. The same type of test is also applied to gravels etc. when the resistance to weathering of the coarse material is open to suspicion. The wet-dry cycle test may also be used on any gauge of stone proposed for macadam etc., or to sandstone etc., when resistance to weather is suspect.

Procedure:

(a) Crush sample to pass $\frac{3}{4}$-in. sieve.

(b) Take a portion weighing 4,000 g. and place in enamel dish. Saturate with water and allow to stand overnight. Carefully decant any surplus water and dry by gentle heating on a hot plate.

(c) Repeat wetting and drying till portion has had 10 wet-dry cycles.

(d) Compact once at approximate optimum moisture content by Proctor method.

(e) Test resulting material in appropriate fashion. (e.g. as gravel if for top course or as soil if for lower course or sub-base.)

D. SANDSTONE, ETC.

This is applied to sandstone and other granular rocks not prone to weathering, too soft for macadam, etc., and which do not produce a satisfactorily graded material on crushing.
Procedure: Conduct standard British Aggregate Crushing Test ($\frac{3}{4}$- to $\frac{1}{2}$-in. material) on a wet sample after 24 hours soaking in water. If resistance to weathering is in doubt subject another portion of sample to ten wet-dry cycles (as set out for shales, etc.) and note whether there is any disintegration.

APPENDIX B

METHODS OF INTERPRETATION OF TEST RESULTS

1. GENERAL NOTE

The rules and requirements following assume:—

(a) *Grading Rules and Requirements.*
That the specific gravity varies little with particle size.

(b) *Rules and Requirements involving LL, PL and (or) PI*
That the mineral particles passing 36 B.S. (40 U.S.) sieve are non-porous and of not too abnormal a specific gravity.

(c) *Rule involving Maximum Dry Weight.*

That the specific gravity of the mineral particles does not differ too much from the normal value of 2.65 (tolerance is less with high weights than with low).

If these conditions are not fulfilled the rules affected cannot be applied directly. In some cases test figures can be adjusted for the variations from normal and the rules applied to the corrected figure.

For example with ashes or cinders or other porous materials the grading rules may be applied in many cases, but the S.M.R. Rule or the U.S.L. check never. If mineral constituents are non-porous but of abnormal specific gravity all figures can be corrected and any applicable rule then applied.

2. PAVEMENT THICKNESS

The effective thickness of pavement required over a given subgrade or of upper courses over a given sub-base or base course, assuming normal and satisfactory drainage and proper compaction and that a bituminous surface or upper course is to be provided, is in general, to be computed two ways, as follows:—

(a) *Grading Rule.* Disregard all material retained ¾-in. square sieve.

Compute the following ratios:—

Title	*Ratio (percent) of all*	*To all Passing*
R	Passing ¾-in. sq. sieve but retained No. 7 B.S.	¾-in. sq.
A	Passing No. 36 B.S.	No. 7 B.S.
B	Passing No. 200 B.S.	No. 36 B.S.
C	Less than 0.0135 mm.	No. 200 B.S.

Let D, E and F be departures of A, B and C respectively outside range 40 to 60.

e.g. $D = A - 60$ if A is greater than 60.
$= 0$ if A is from 40 to 60.
$= 40 - A$ if A is less than 40.

If neither A, B, nor C, is less than 40; compute the sum of $D + E + F$.

If neither A nor B is less than 40, but C is; compute the same sum but count not more than 20 for F.

If A is not less than 40 but B is; compute the same sum but count not more than 20 for $E + F$.

If A is less than 40; compute the same sum if it is less than 20, otherwise count $D + E + F$ as 20.

To the sum so determined add one half the plastic index and subtract one quarter R. Call this final total T.

Effective cover required in inches is then:—

$0.15\,T$ for Heavy Loading (Max. wheel = 6 English Tons or say = 13,500 lb.)

$0.12\,T$ for Normal Loading (Max. wheel = 4 English Tons or say = 9,000 lb.)

(b) *Soil Moisture Relations Rule.*

Let U = Upper Solid Limit.
= PL if plastic.
= LL if non-plastic.
S = Linear Shrinkage from LL of Pass No 7 B.S. Portion in percent of original length.
W = Max. Dry Weight (lb. per cu. ft.) by standard Proctor compaction method.

Effective cover required in inches is then:—

$32 + 0.16\,U + 0.27\,S - 0\ 24\,W$ for heavy loading.

$26 + 0.13\,U + 0.22\,S - 0.195\,W$ for normal loading.

Tables A and B cover all usual cases.

TABLE A
HEAVY LOADING

U	$0.16\,U$	S	$0.27\,S$	W	$32-0.24\,W$	W	$32-0.24\,W$
11	1.76	1	0.27	85	11.60	109	5.84
12	1.92	2	0.54	86	11.36	110	5.60
13	2.08	3	0.81	87	11.12	111	5.36
14	2.24	4	1.08	88	10.88	112	5.12
15	2.40	5	1.35	89	10.64	113	4.88
16	2.56	6	1.62	90	10.40	114	4.64
17	2.72	7	1.89	91	10.16	115	4.40
18	2.88	8	2.16	92	9.92	116	4.16
19	3.04	9	2.43	93	9.68	117	3.92
20	3.20	10	2.70	94	9.44	118	3.68
21	3.36	11	2.97	95	9.20	119	3.44
22	3.52	12	3.24	96	8.96	120	3.20
23	3.68	13	3.51	97	8.72	121	2.96
24	3.84	14	3.78	98	8.48	122	2.72
25	4 00	15	4.05	99	8.24	123	2.48
26	4.16	16	4.32	100	8.00	124	2.24
27	4.32	17	4.59	101	7.76	125	2.00
28	4.48	18	4.86	102	7.52	126	1.76
29	4.64	19	5.13	103	7.28	127	1.52
30	4.80	20	5.40	104	7.04	128	1.28
31	4.96	21	5.67	105	6.80	129	1 04
32	5.12	22	5.94	106	6.56	130	0.80
33	5.28	23	6.21	107	6.32	131	0.56
34	5.44	24	6.48	108	6.08	132	0.32

(c) *Application of Rules.* If the dispersion fails, the ratios A, B and C are not determined and the grading rule cannot be applied. The S.M.R. rule only is taken into account.

If the dispersion does not fail:

1. If the adverse constituents are present in considerable quantity the grading rule does not apply and the S.M.R. rule only is taken into account.
2. In A3 soils[1] free from appreciable adverse

[1] Passing 200 less than 10 percent of pass 7 or less than 15 percent of pass 36.

constituents S.M.R. rule does not apply and grading rule only is taken into account. (If much adverse constituents present, S.M.R. rule applies but not grading—vide 1 above).
3. In other cases average of the two rules is taken.

Except in the case of A3 soils the computation by the two rules is usually a simple guide as to presence of adverse constituents. If grading rule exceeds (S.M.R. rule minus 2 in.) the adverse constituents may be neglected and the mean of the two rules taken. If grading rule is less than (S.M.R. rule minus 4 in.) they are present in quantity and grading rule is discarded.

TABLE B
NORMAL LOADING

U	$0.13\ U$	S	$0.22\ S$	W	$26-0.195\,W$	W'	$26-0.195\,W'$
11	1.43	1	0.22	85	9.42	109	4.74
12	1.56	2	0.44	86	9.23	110	4.55
13	1.69	3	0.66	87	9.04	111	4.36
14	1.82	4	0.88	88	8.84	112	4.16
15	1.95	5	1.10	89	8.64	113	3.96
16	2.08	6	1.32	90	8.45	114	3.77
17	2.21	7	1.54	91	8.26	115	3.58
18	2.34	8	1.76	92	8.06	116	3.38
19	2.47	9	1.98	93	7.86	117	3.18
20	2.60	10	2.20	94	7.67	118	2.99
21	2.73	11	2.42	95	7.48	119	2.80
22	2.86	12	2.64	96	7.28	120	2.60
23	2.99	13	2.86	97	7.08	121	2.40
24	3.12	14	3.08	98	6.89	122	2.21
25	3.25	15	3.30	99	6.70	123	2.02
26	3.38	16	3.52	100	6.50	124	1.82
27	3.51	17	3.74	101	6.30	125	1.62
28	3.64	18	3.96	102	6.11	126	1.43
29	3.77	19	4.18	103	5.92	127	1.24
30	3.90	20	4.40	104	5.72	128	1.04
31	4.03	21	4.62	105	5.52	129	0.84
32	4.16	22	4.84	106	5.33	130	0.65
33	4.29	23	5.06	107	5.14	131	0.46
34	4.42	24	5.28	108	4.94	132	0.26

In intermediate cases the test results should be examined in detail to decide the point (if uncertain there is little error in taking 1 in. less than the S.M.R. rule).

In A3 soils there is a transition zone where application of S.M.R. and grading rules is uncertain, but this is a rare case in practice and thicknesses are not unduly large. A safe method is to take the higher of the two rules in this doubtful zone

(d) *Effective Cover.* The effective cover is the sum for all overlying courses counted as follows:—

Macadam: Stone;	
Sandstone:	Actual Thickness
Gravel-sand-clay; Sand-clay:	" "
Bituminous Courses:	
Dense and solid:	Twice actual thickness
Semi-dense grading:	One and one-half times actual thickness.
Very open grading:	Actual thickness.
Surface treatment only:	Neglect.
Soil sub-bases, etc:	
A1, A2, A3 (1942 PRA):	Actual thickness
Other groups.	Two-thirds actual thickness.

For unsurfaced pavements the same thickness is required on sandy non-plastic soils as for bituminous pavements. On plastic and high organic soils the thickness may be reduced by one third if unsurfaced

3. GRAVEL AND SAND-CLAY TO RECEIVE SURFACE TREATMENT

The requirements given are for material as finally compacted in the pavement. Samples from deposits etc. may require special treatment prior to testing (see Appendix A). The material will be suitable if it complies with all the following requirements:—

1. Total T not to exceed 5.
2. Upper Solid Limit not to exceed 20.
3. Plastic Index not to exceed 8.
4. Maximum Dry Compression strength not to be less than 400 lb. per sq. in. if normal practice of compaction under traffic is to be followed or 250 lb. per sq. in. if special provision is made for watering, rolling, and immediate surface treatment.

A limit is also normally desirable for oversize (i.e. proportion retained ¾-in. sq. sieve); desirable limit is 5 percent of whole.

4 GRAVEL AND SAND-CLAY FOR USE UNSURFACED

Here strict compliance with a definite specification should not be universally enforced as it may be economical to use more or less inferior materials, depending on local supplies available, traffic using the road, climate, etc. Referring to (2) preceding for meaning of symbols, and to (3) for treatment prior to testing, the following are, however, the requirements for first-class materials:

Riding comfort: Tire Damage:

(a) Oversize (Ret. ¾-in. sq.) Max. 5 percent.

Easy Maintenance: Wear Resistance:

(b) R over 40 percent V. Good.
R 20–40 percent Good.
R 10–20 percent Less satisfactory.
R Under 10 percent Poor.

Stability against Corrugation, rutting and ravelling:

(c) Total T substituting 65 for 60 in computing excesses D, E and F, not to exceed 5.
(d) Neither A, B nor C to be less than 45.
(e) Upper Solid Limit not to exceed ($20 + \frac{1}{4}$ PI).
(f) Plastic Index not to exceed 15.
(g) Max. Dry Compression Strength not to be less than 400 lb. per sq. in.

5. SHALE AND OTHER SOFT ROCKS

The test results on the product from the artificial weathering test are treated as for soil if material is for a subgrade or base course. If the material is to be used as a top course it should comply with the requirements for gravel etc. It may happen in the latter case that the grading tends to the hungry side, i.e. ratios A, B and C tending to 40 or less, with PI zero and Max. Dry Compression Strength low. In such cases the material will be satisfactory as top course with the addition of a little fine blending material which, if the amount required is small, may even be supplied by the dust created by the grinding action of traffic on the material itself. In some of these cases the U.S.L. exceeds the permissible value of 20. It is then necessary to determine whether this is due to the open grading and the angular shape of the sand sizes (in which case the material is acceptable and the upper solid limit will fall below 20 with the extra fines) or due to the presence of mica etc. (in which case the material is inferior).

6. SANDSTONE, ETC.

Loss by British Crushing Test, conducted on wet $\frac{3}{4}$- to $\frac{1}{2}$-in. material after soaking for 24 hours, is to be within the following limits—

If to receive a thin bituminous surface course only:—from 30 to 55 but preferably 35 to 50.

If to receive thicker surface course or courses:—from 30 to 60 but preferably 35 to 55.

(*Note:* Loss less than 30 means material too hard for this type of construction but suitable for broken stone construction).

In addition the material should stand ten cycles of alternate wetting and drying without distintegration. This test need only be conducted if poor resistance to weathering is suspected.

PAVEMENT EVALUATION BY LOADING TESTS AT NAVAL AND MARINE CORPS AIR STATIONS

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SYNOPSIS

This paper is a second progress report of an extensive pavement evaluation study, sponsored jointly by the Bureau of Yards and Docks and the Bureau of Aeronautics of the Navy Department.

Pavement evaluation data have been obtained for 54 airfields. The total thicknesses of the flexible type pavements are, with few exceptions, between the limits, 6 and 12 in. This seems remarkable in view of the fact that these pavements have accommodated planes having wheel loads varying from 2,500 to 60,000 lb. for the most part without serious damage to the pavements and without excessive maintenance costs.

Complete subgrade data for both asphalt surfaced and concrete pavements are presented for 32 airfields. This extensive study has indicated beyond doubt that under the pavements that have been in use, the in-place subgrade moisture content has been less and the in-place subgrade density has been greater for most soil types than has been generally assumed in design.

It has been observed that triaxial shear data obtained with undisturbed subgrade samples provide a good index to the bearing value of the subgrade.