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Application of an Impact Test to Field Evaluation of Marginal Base Course Materials

BADEN CLEGG

A procedure for selection of base course materials that involves the use of a rapid in situ test based on an instrumented modified AASHO compaction hammer is discussed. Results of this test are compared with those of more common tests such as California bearing ratio and Texas triaxial, which have been used to evaluate marginal base course materials in Western Australia for use as surface courses under light seal coats. It is also used to derive an elastic modulus and to determine variability. Strength changes during and after construction, in terms of the impact value obtained from this test, are examined. From these observations, a procedure is suggested for selection of base course materials in the field. It is concluded that the effect of traffic compaction on strength improvement is worthy of further investigation and evaluation.

The selection of materials for base courses for low-volume roads involves to a large degree strength evaluation in relation to the in-service moisture environment. For "wet" climates this has led to testing "saturated" or "soaked"; for "dry" climates, testing after "drying back" from compaction moisture content has become common. In the latter situation, Australian practice is now related to the establishment of a design moisture content (DMC) that is based on an equilibrium moisture content (EMC) (1). By coupling DMC with the expected density, a design strength is established from laboratory tests, usually in terms of California bearing ratio (CBR). The procedure of using in situ strength testing at expected moisture-density conditions is provided for, particularly when there are similar existing pavements that can be used as a basis for comparison. The most common test used in this respect is the cone penetrometer (2). Field CBR testing has the advantage that it can be related to laboratory tests, but it is relatively cumbersome and time-consuming to perform.

In an effort to overcome these testing difficulties, I developed a device that has become known as the Clegg impact soil tester, sometimes referred to as the Clegg hammer. This device enables rapid in situ and laboratory testing to be performed with a minimum of inconvenience and hence enables strength changes during and after construction to be monitored with ease. It has proved particularly useful for evaluating base courses that are to be only lightly surfaced, such as with seal coats, and also for stabilized materials and unsealed gravels.

This paper presents the latest developments with respect to the impact soil test and correlations with other common tests such as CBR, Texas triaxial, and an elastic modulus. It also shows how field measurements of impact value are related to performance of base courses and how, by monitoring strength changes during construction and throughout pavement life, it is possible to evaluate the potential serviceability of a base course material. The main thrust of the discussion is that, by close interaction between testing and construction, design and performance can likewise interact more closely with the consequent better exploitation of materials.

TEST PROCEDURE

The impact soil test was developed to assist with an evaluation of low-volume roads in Western Australia. Extensive research had been undertaken to determine why prime and seal coat surfacings were giving many years of satisfactory service (3-5). This research indicated that the field moisture contents in the state were often considerably lower than modified American Association of State Highway Officials (AASHO) optimum and that excellent sealing techniques had been developed that prevented water entry. The climate, which is characterized by an appreciable excess of dry days over wet days, appeared to be a contributing factor. For this research, the Texas triaxial test had been used as a basis for the strength evaluation. However, this test could not be performed as a field test for direct comparison with performance--as could, for example, the CBR--and it also had other disadvantages, particularly with respect to lack of portability. The logic for the development of a rapid, inexpensive test became evident.

The concept of the impact test developed is quite basic and straightforward (see Figure 1). An accelerometer is fitted to a compaction hammer, and the response on impact is measured with an electronic device. In particular, a piezoelectric-type accelerometer is used to obtain the peak deceleration on impact after appropriate filtering to avoid the need for elaborate surface preparation. The earlier models used an analog readout and multiple switches, but the latest version has digital display and single-button operation. This peak deceleration in gravities divided by 10 has become known as the Clegg impact value (CIV). The initial studies with this device consisted of laboratory tests in CBR molds to observe response with changing density and moisture content (6). These tests confirmed the heavy, 4.5-kg compaction hammer with 45-cm drop height as being the most suitable combination. They also showed that the fourth blow reading is a convenient terminating point, since the CIV was found to increase with successive blows as the impact surface became flattened and continuing on after four blows could, with some materials, lead to a gradual increase in value due to compaction of the test specimen. It was also determined that the 0-100 scale for CIV covered the full range of materials up to those with CBRs of about 700 percent. Since the introduction of the impact soil tester in 1976, approximately 150 of these devices have been manufactured for use in Australia and elsewhere.

CORRELATIONS AND VARIABILITY

The most useful correlation for any such device is that with CBR (7). Experimental observations indicate that the relation is as shown in Figure 2. The data points were obtained from a wide range of materials and conditions by several different research workers. The actual data are given in Table 1. It should be noted that all the tests were performed on unsoaked samples with no surcharge. Added to Figure 2 is a theoretically derived curve based on the observation, as a result of examining the response on a wide range of soils, that deceleration versus time may be assumed to be sinusoidal. By double integration of the area under such a curve, penetration versus time is obtained. The peak force may also be obtained from the deceleration. Thus, a force-penetration curve may be obtained as illustrated in Figure 3 (8). Based on commonly accepted correlations between CBR and Texas Class Number (TCN) (9), together with some actual correlation tests, an approximate relation between CIV and TCN is as shown in Figure 4. The estimated curve was derived by first using Figure 1 to obtain the CBR and then using the relation given by Yoder and Witczak (9), based on the AASHO Road Test, to obtain the TCNs. Test data were obtained by performing impact tests on some samples used for TCNs in previous research (4,5).

A further development from the theoretical concept is a derived "elastic modulus". From the relation shown in Figure 3 and application of elastic theory for a rigid bearing plate, a modulus is obtained as shown in Figure 5. This E_s should, of course, be seen only in relation to this particular test method. Some other derived relations have been added for comparison. Previous research (4,5) provided data for determining a correlation between CBR and E_s from Texas triaxial tests at a confining stress of 85 kPa. By using the experimental curve in Figure 2, the corresponding impact value was obtained. Some unconfirmed compression tests on cement-stabilized crushed rock produced an axial stress-strain modulus for comparison with impact tests on the same material. For further comparison, data from the AASHO Road Test (9) that correlate E_s with CBR were used together with the experimental curve of Figure 2. The relations as shown depend on the test procedures, but there is clearly justification for considering the CIV as producing a "compaction hammer modulus".

Of particular interest in this context is the relation between the CIVs and the performance of base courses under traffic. To investigate this aspect, I undertook a survey of pavements around Australia, the results of which are given in Table 2 (10).

The terms used in the table to describe performance can be defined as follows (1):

1. Poor denotes that the seal was showing visible distress such as rutting, cracking, and/or bleeding due to cover aggregate penetration into the base. There was evidence of maintenance patching. The general consensus of opinion was that a few years of life with high maintenance was all that could be expected.

2. Satisfactory corresponds to minor distress such as edge fretting or minor rutting. General opinions were not strong one way or the other but indicated that several years of reasonably satisfac-

tory life could be expected.

3. Good indicates no signs of distress that could be attributed to the base course. Generally unequivocal opinions indicated the soundness of the base course and probably many future years of satisfactory service.

It should be noted that "performance" was a very subjective term used by field engineers based on the fact that the material could be processed and a seal coat could be applied with the expectation of more or less maintenance and short or long periods between reseals. In most cases the seal could be removed before testing, but in others the seals could not be removed easily and testing was then carried out on top of the seal or at the shoulder. Despite the obvious variations, it is suggested that a CIV of about 30 corresponds to poor performance, 50 to satisfactory performance, and 70 to good performance. From Figure 2, it can be seen that the correlation between CIV and CBR is as follows:

CIV	CBR
30	50
50	150
70	300

In Australia, the commonly acceptable lower limit of CBR for base course materials is 80; a value as low as 50 is sometimes considered acceptable. Thus, the field observations agree in a general way with current practice. A further point of interest is that McInnes (11) concluded that a TCN of less than 2.3 is desirable for conditions in Western Australia. From Figure 4, this indicates a CIV of 30-40, which

Figure 2. CIV versus CBR.

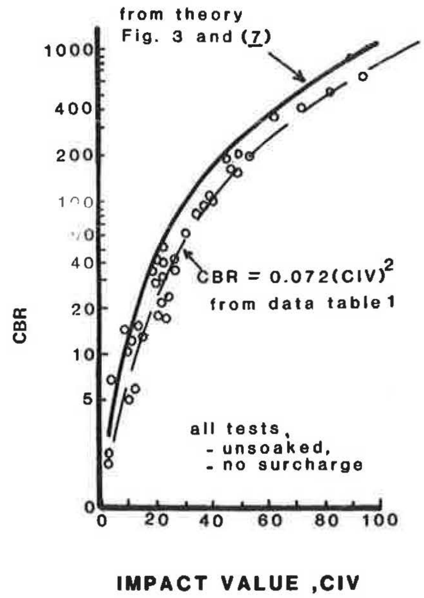


Figure 1. Impact soil tester in use, digital display, and carrying position.

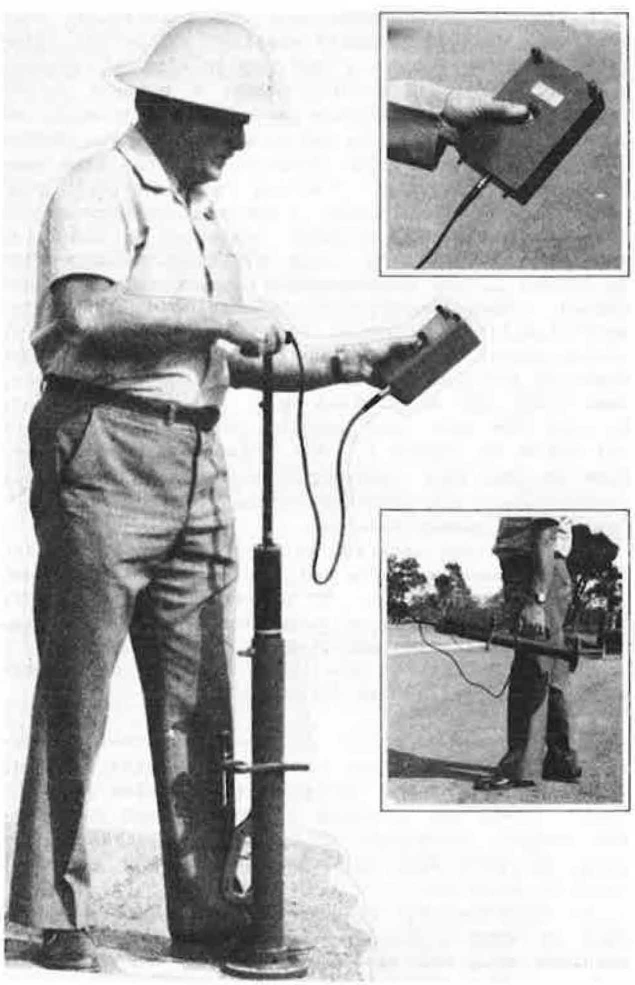


Table 1. Data for correlation of CBR and CIV.

Sample No.	Modified AASHO Compaction (%)	Dry Density (Mg/m^3)	Moisture (%)	Cement (%)	CBR (%)	CIV
C1	-	2.45	5.2	3	650	95
C2	-	2.29	5.7	3	500	85
C3	-	2.13	6.5	3	400	72
C4	-	2.07	4.8	3	355	60
RR71	96	2.20	5.5	-	217	48
RS	-	-	3.8	-	207	54
C6	-	1.96	5.5	3	198	45
RR44	94	2.06	10.5	-	160	49
C5	-	2.00	5.7	3	160	58
RR44	95	2.09	6.2	-	151	51
47	102	2.22	5.0	-	110	44
RR41	96	2.05	10.1	-	107	42
RR25	95	2.07	5.1	-	107	41
R596	-	2.06	8.0	-	91	38
H12	105	2.09	6.3	-	90	35
H6	97	2.11	5.0	-	80	34
RR50	87	1.91	4.8	-	63	28
H11	103	2.05	6.3	-	60	31
RR33	-	1.89	3.7	-	58	25
RR25	-	1.79	4.2	-	50	22
RR44	-	1.80	4.8	-	42	21
RR56	87	1.92	13.5	-	40	21
H4	93	2.03	5.0	-	40	25
RR48	87	2.06	5.5	-	37	22
H3	92	2.01	5.0	-	35	23
76M19	-	1.80	10.6	-	33	24
H10	97	1.93	6.3		30	19
H2	90	1.96	5.0	-	25	19
RR48	91	2.16	12.4	-	22	22
77M15	-	1.72	1.1	-	17	20
H9	94	1.87	6.3	-	17	13
H1	88	1.92	5.0	-	15	14
RR41	95	2.02	12.4	-	15	16
RR59	-	1.73	3.4	-	15	14
RR57	-	1.69	8.9	-	12	13
77M17	-	1.72	0.6	-	12	11
H8	89	1.77	6.3	-	7	4
R596	-	1.73	5.0	-	5	11
RR56	-	1.59	22.6	-	2	4
RR56	76	1.67	21.0	-	2	3

Figure 3. Development of force versus penetration from deceleration versus time.

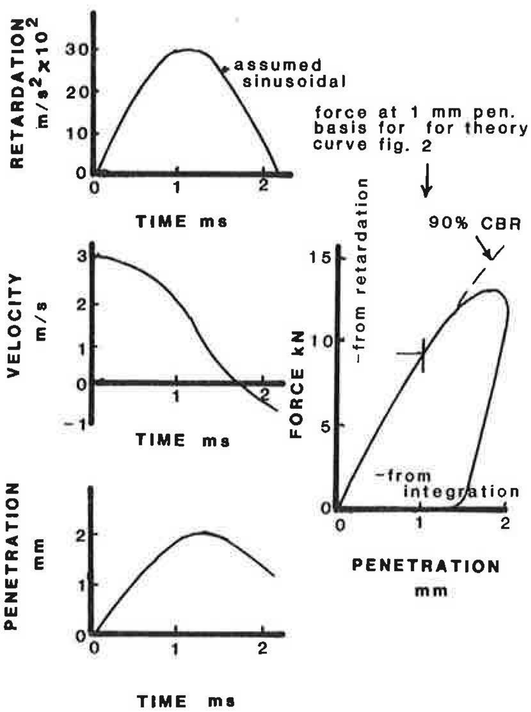
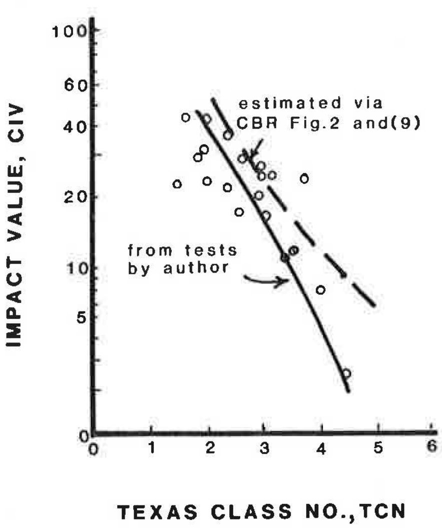


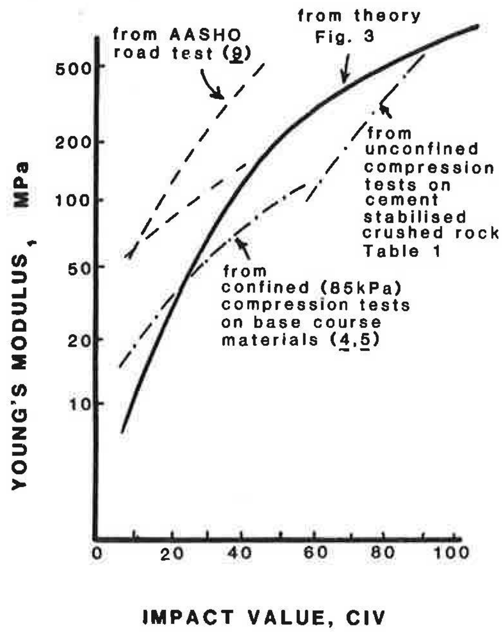
Figure 4. CIV versus TCN.



also shows broad agreement with the field observations.

Of particular interest is the as-constructed variability of base courses over the area constructed or being considered. Tests across the lane being compacted invariably show lower values along the outer edge where less rolling has been applied. In addition, field trials (10) showed, for example, that a well-graded fine-crushed rock base course gave impact values with a coefficient of variation of 4 percent along the center line, up to 13 percent along the edge lane, and 5 percent across the lanes. Furthermore, it appeared that low coefficient of variation was related to high percentage of compaction. On relatively large areas, it has been found to be possible to plot the strength contours

Figure 5. Young's modulus versus CIV.



by using computer techniques. Figure 6 shows the results of testing an area of unsealed crushed rock while it was being prepared for a construction yard. The area, about 110x180 m, was covered on a 10-m grid with about 250 tests by one person in about 3 h. Because the impact test took only seconds to perform, the time to get from station to station became the major factor in the overall testing time. This ability to quickly scan large areas while they are under construction provides a valuable assessment of overall quality and should lead to a better end product.

STRENGTH CHANGES DURING AND AFTER CONSTRUCTION

It is well known that strength resulting from compaction depends on the interaction between compactive effort (machine, layer thickness, passes, etc.) and moisture content (above or below optimum for the compactive effort) and that these together result in a certain degree of packing (density), which in turn has an effect on the strength of the particular material. Specifying percentage of compaction as the only compaction control parameter neglects the effect of water content on strength and therefore does not guarantee adequate strength or necessarily mean that the full-strength potential of a material has been mobilized before sealing.

Field observations of CIV confirm that for construction in relatively dry climates there are four "equilibrium" points or conditions that may be considered:

1. As constructed--Immediately after watering and rolling;
2. Shortly after construction (e.g., days)--When moisture equilibrium is established with the environment, generally means some drying out;
3. Some time after sealing (weeks)--When moisture equilibrium is reached, CIV is controlled largely by conditions 1 and 2 above; and
4. Long term (e.g., months or years)--Variations due to factors such as seasonal changes, CIV tending to be very pronounced at the edges and outer wheel paths.

Table 2. Compaction requirements for granular, cement-bound granular, and soil-cement materials.

Poor				Satisfactory				Good			
Postcode/ Locality	Common Description	Surface Condition	CIV	Postcode/ Locality	Common Description	Surface Condition	CIV	Postcode/ Locality	Common Description	Surface Condition	CIV
2880/ Broken Hill	Limestone schist	SR	30	2880/ Broken Hill	Soft rock (amphibolite)	US	45	2880/ Broken Hill	Sandy clay	SR	60
3280/ Warrnambool	Limestone	SR	35	3280/ Warrnambool	Scoria	SR US	55 40	3280/ Warrnambool	Crushed rock	US	50
3393/ Warracknabeal	Sandstone	SR	40	3280/ Warrnambool	Stabilized tuff	US	40	3280/ Warrnambool	Limestone and scoria	US	70
3393/ Beulah	Limestone	SR	40	3280/ Warrnambool	Limestone	SR	60	3401/ Tooan	Ironstone gravel	SR	70
3395/ Galaquil	Limestone	SR	30	3401/ Horsham area	Sandstone	SR SR	60 50	3844/ Traralgon	Limestone	US	55
3400/ Horsham area	Sandstone	SR	40	3453/ Harcourt	Granitic sand	US	45	4725/ Barcaldine	Ironstone gravel	SR SR	80 70
3401/ Tooan	Limestone	SR	35 30	3875/ Bairnsdale	Quartzite sand top	US SR	40 40	4730/ Longreach	Ironstone gravel	US	80
3550/ Bendigo	Sandstone	US	35	4725/ Barcaldine	Kopai sandstone	US	40	5690/ Ceduna	Crushed limestone	US US	70 70
3875/ Bairnsdale	Quartzite	US	45	4725/ Barcaldine	Kopai	SR	50	5690/ Ceduna	Crushed limestone	US US	70 70
4730/ Longreach	Sandstone	SR SR	20 35	4730/ Longreach	Stabilized limestone	US	35	5700/ Port Augusta	Crushed rock	US US	90 70
4730/ Longreach	Sandstone	SR	10 30	6163/ Spearwood	Stabilized limestone	US	50	6152/ Mount Henry	Crushed rock	US	50
				7251/ Exeter	Quartzite gravel	SR SR	50 60	6152/ Mount Henry	Crushed rock	US US	80 60
				7251/ Exeter	Quartzite gravel	SR SR	60 55	6201/ Byford	Laterite gravel	US	70
								6642/ Meekatharra	Laterite gravel	US US	70 80
								7207/ Antill Ponds	Laterite gravel	US	60
								7310/ Devonport	Quartzite gravel	US US	60 50

Note: SR = seal removed; US = unsealed.

With regard to condition 1, common practice aims for compaction at a moisture content corresponding to laboratory optimum for the compactive effort being applied. The CIV increases as compaction proceeds until either a stable strength condition is reached or, alternatively, particularly when the field condition is wet of the line connecting the optimums. Experiments have shown that if the compaction water content is varied a peak strength occurs just dry of optimum for the particular compactive effort, as shown in Figure 7. Thus, to maximize the strength potential of a given material, the material should be processed by using an effort equivalent to heavy compaction, with the water content at such a level that any increase results in loss of strength.

Field tests on crushed-rock base courses showed that, at modified AASHO optimum water content, CIVs from about 30 to 35 correspond to compaction better than 95 percent (10). If one considers the performance relation in Table 1, this is the lowest strength that should be considered as making a seal coat acceptable unless drying takes place that leads to an improvement in strength.

With regard to condition 2 in the list above, it has been observed that, where relative humidity and/or windy conditions are such that drying occurs, an increase in CIV results. The extent of this increase appears to be related to the amount and nature of the silt and clay fines. In the case of a nominally nonplastic, well-graded base course, the 30-35 CIV at compaction water content is likely to increase to 40-45 in one or two days and to greater than 50 after about four days when it stabilizes. A more plastic gravel tends to produce a CIV of less than 30 after compaction, but there is a rapid increase to 60 or more within a few days. A more borderline material such as a nonplastic soft sandstone may produce a CIV of 30 at compaction, but drying may not produce a CIV of more than 40, even under conditions of high evaporation.

If, after the initial equilibrium condition is reached, a prime or seal is applied, moisture will move back toward the surface, and in due course another level is reached--condition 3, in which the CIV is a little lower than it was in condition 2 but not as low as condition 1. Moisture movement studies by McInnes (11) and others have shown that this equilibrium condition is often largely related to the "sealed-in" moisture content and, hence, to the moisture conditions at the time of compaction and that this may have an influence for a number of years. In due course, the influence of construction period is less significant and condition 4 is reached. This condition is largely related to environmental factors through moisture movement in and out of base courses via shoulders, flooding, seal failures, etc.

Another factor that may lead to a substantial increase in CIV is densification under traffic. This results in much higher values under wheel paths. Under these conditions, CIVs of 50 are likely to increase to 80 in the trafficked areas, particularly if the densification is accompanied by drying. On the other hand, traffic compaction under wet conditions may lead to appreciable loss of strength and premature failure.

The above observations suggest that, by monitor-

Figure 6. Variability as indicated by CIV contours.

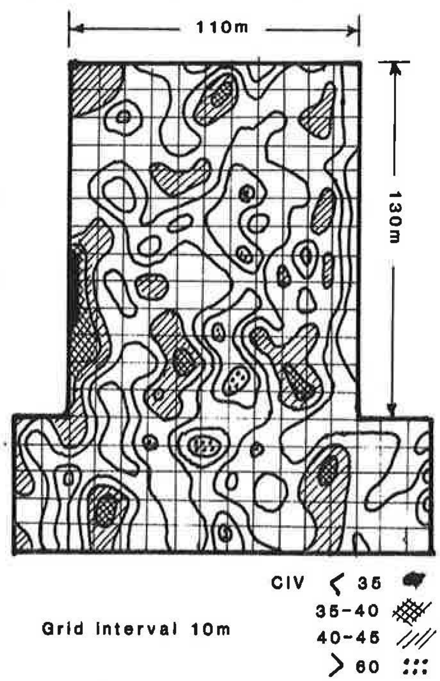
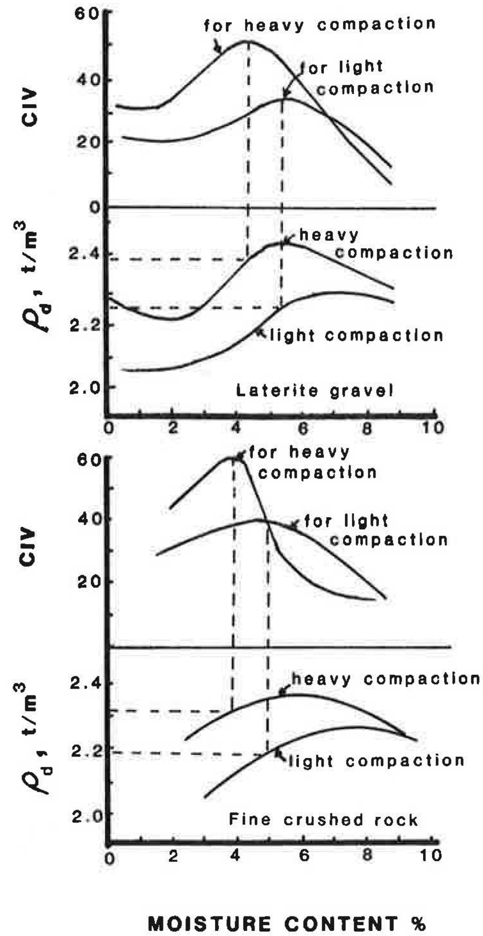


Figure 7. Peak impact values in relation to compaction curves.



ing the CIV and assessing changes in relation to environmental events, it may be possible to determine by field CIV observations alone whether a material is suitable and the best procedure to use to maximize its potential. The exploitation of the so-called "substandard" base course materials referred to earlier in this paper, which led to the research program at the University of Western Australia, grew out of necessity. In many areas of the State, there were just no other materials available. Subsequently, the research was undertaken to establish the reasons for the successful performance of these materials. The original basis for acceptance seems to have been that, if you could take a given material, process it by watering, rolling, and trafficking, and by waterbinding produce a good hard surface that would take a prime and seal, it should last long enough to justify its use. There was also the notion that the seal should be impermeable to water but have some permeability to air. Thus, acceptability was largely determined by "processability", the only test being the "ring of a pick handle", or some other stout piece of wood, when dropped on the base course before priming. No doubt such procedures have been and still are common elsewhere.

Today, sophisticated soil testing is commonly regarded as essential. However, in remote areas, testing facilities may not be readily available and, although that form of control might be considered desirable, judgment is often still the major factor in the assessment of suitability. The concept of a rapid, meaningful in situ test to back up this judgment thus has considerable appeal, particularly if the test can be performed by the engineer or supervisor. Although this concept is particularly appealing for remote areas, it is not without application to higher-standard pavements, where variability is of major concern, and in situations where as-constructed strength evaluations may be neglected or overshadowed by emphasis on control by density testing.

SUGGESTED FIELD DESIGN PROCEDURE

On the basis of the foregoing and with the objective of achieving close interaction among design, testing, and construction, the following procedure is suggested (it should be emphasized that this proposal is in the developmental stages and may need to be modified in order to meet a particular need as experience dictates):

1. Depending on climate, select a possible material. This may be on the basis of previous experience with similar materials or from soil classifications. If the climate is such that field moisture conditions below heavy compaction optimum are anticipated, finer-grained, more plastic materials such as clayey sands may be suitable. For greater than modified optimum moisture conditions, good grading and low-plasticity materials such as gravels and crushed rocks may be essential.
2. Select an appropriate compaction procedure. This may be based on previous experience, and availability of plant will no doubt be a major factor. A helpful guide is given in Table 3 (12).
3. Construct a trial area. Adjust the moisture content and compact according to step 2. First select the water content that feels right--i.e., is workable, molds easily, and is not sloppy.

After compaction, carry out CIV tests along and across the trial area. Perform not less than 15 tests and aim for as small a variation as possible, performing more compaction where required to achieve this. Repeat the watering and compaction process, using, say, first a higher then a lower moisture content than at the beginning. This procedure should be repeated until the maximum CIV is deter-

Table 3. Base course performance data.

Type of Compaction Plant	Category	No. of Passes for Layers Not Greater Than 110 mm	150 mm	225 mm
Smooth-wheeled roller	Mass per meter width of roll			
	>2700-5400 kg	16	U	U
	>5400 kg	8	16	U
Pneumatic-tired roller	Mass per wheel			
	>4000-6000 kg	12	U	U
	>6000-8000 kg	12	U	U
	>8000-12 000 kg	10	16	U
	>12 000 kg	8	12	U
Vibrating roller	Mass per meter width of vibrating roll			
	>700-1300 kg	16	U	U
	>1300-1800 kg	6	16	U
	>1800-2300 kg	4	6	10
	>2300-2900 kg	3	5	9
	>2900-3600 kg	3	5	8
	>3600-4300 kg	2	4	7
	>4300-5000 kg	2	4	6
	>5000 kg	2	3	5
Vibrating-plate compactor	Mass per unit area of base plate			
	>1400-1800 kg/m^2	8	U	U
	>1800-2100 kg/m^2	5	8	U
	>2100	3	6	10
Vibro-tamper	Mass			
	>50-65 kg	4	8	U
	>65-75 kg	3	6	10
	>75 kg	2	4	8
Power rammer	Mass			
	100-500 kg	5	8	U
	>500 kg	5	8	12

Note: U = unsuitable.

mined or, alternatively, until the value before an increase in water content produces a drop in CIV. This value should be used as the target value to be achieved at the time of compaction and also to assess the suitability of the material since the performance requirement is greater than 50 CIV. If wet--i.e., over optimum--conditions are considered possible, the compacted area or part of it should be flooded, allowed to soak overnight, and then retested. For good performance, the desirable 50 CIV should be maintained; at the same time, it should be noted that 30 is the lowest acceptable CIV and this for light traffic only.

If dry conditions are anticipated, there should be a 24- to 48-h wait and then another test. Drying in this period should improve the CIV by about 10. Any greater increase probably means high clay content and that the material is moisture sensitive. In this case, the material must not be used in areas that are likely to become flooded; in any case, good priming and sealing will be essential. It may be thought desirable to test again after sealing (with the seal removed), or the surface might be covered with plastic sheets for a period to check any further change when evaporation is prevented.

The CIV values used here are only tentative at this stage and may need adjusting in the light of experience.

By using a compaction procedure that is known to be capable of producing adequate density, the objective of a target strength as described in step 2 will not only maximize strength potential but should also produce adequate density; i.e., there will be little subsequent wheel-path rutting as a result of low base course compaction.

CONCLUSIONS

The impact soil tester described in this paper shows considerable promise as an alternative to the commonly accepted procedures for the evaluation of base courses. It has the decided advantages of portability and simplicity while producing a direct read-out (CIV) that does not require any manipulation since it is the actual strength parameter--that is, equivalent to a "hammer modulus". It may be seen as an extension of the long-practiced art of "sounding" a base course with instruments such as a pick handle or a geologist's hammer.

The ease with which the impact tests are performed offers the possibility of monitoring changes in strength as construction proceeds. By recognizing the factors that may lead to these changes, such as compaction and environment, it is suggested that a procedure can be developed to assess the suitability of base course materials under actual field conditions. Establishing a close relation between design, testing, and construction will not only maximize a potential material's performance capacity but will also effect a general improvement in the overall standard of workmanship and a reduction in variability and result in better use of marginal materials.

Field performance observations have shown that gravel base courses can, under traffic and dry conditions, achieve strength levels of 80 CIV or more (E = 500 MPa). Under these conditions, seal coats appear to give many years of life under low-volume traffic and may even support high-volume traffic. It has also been observed that similar high levels of CIV may be reached in some other materials--e.g., fine crushed rock, where construction has involved a slurrying of the surface and a continuation of rolling well beyond that for normal compaction as the material dries out. The usual acceptability criterion of a minimum CBR of 80 does not accommodate these higher values, which correspond to several hundred percent CBR and are in the order of values for stabilized base materials. This should mean a longer life for seal coats and greater ability to cope with heavier traffic and is therefore an aspect worthy of more investigation and evaluation.

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Calibrating Response-Type Roughness Measurement Systems Through Rod-and-Level Profiles

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Road roughness is highly correlated with serviceability and various components of user costs. Response-type systems (e.g., the Mays meter, the roughometer, and the bump integrator) are frequently used to measure roughness mainly because of their relatively low cost and high measuring speed, whereas a sophisticated device such as the Surface Dynamics Profilometer is necessary for calibrating those systems. An analysis of rod-and-level measurements of pavement profile is presented. Rod-and-level measurements represent a feasible alternative to the Profilometer in that they provide an accurate means for calibrating response-type roughness-measuring systems. Four different profile summary statistics from the literature are used to establish a stable roughness scale: wave amplitude, root mean square vertical acceleration (RMSVA), mean absolute vertical acceleration (MAVA), and slope variance. RMSVA computed for different base lengths is recommended for characterizing road roughness. Accurate RMSVA estimates can be obtained when a 500-mm sampling interval is used to collect pavement profile data with rod and level. It is found that rod-and-level measurements of pavement profile currently constitute the most feasible means for the general transfer of roughness standards. Moreover, the rod-and-level method is particularly appealing for developing countries, where the social costs of labor-intensive procedures may be significantly less than the costs of procedures that depend on sophisticated imported instruments. Rod-and-level measurements are very slow when a short sampling interval of 500 mm is used. Therefore, use of the method is recommended for keeping updated records of pavement roughness on about 20 control sections. These sections, in turn, can be used for calibrating response-type roughness-measuring instruments.

Since the American Association of State Highway Officials (AASHO) Road Test, where the concept of pavement serviceability was developed by Carey and Irick (1), increasing importance has been given to user-related pavement evaluation. This type of evaluation is concerned primarily with the overall function of the pavement--that is, how well it serves traffic or the riding public.

The serviceability of a pavement is largely a function of its roughness (2), and several models can be found in the literature that estimate serviceability as a function of roughness alone (3,4). Moreover, it has been demonstrated that roughness is the principal measurement of pavement condition directly related to vehicle operating costs (5,6).

Roughness is normally measured with response-type measuring systems, which are relatively fast and inexpensive. However, the output of these systems is not stable over relatively long periods of time. Consequently, it is necessary to establish a stable roughness scale against which response-type measuring systems can be calibrated.

A convenient roughness scale was provided by the surface dynamics (SD) profilometer for a United Nations Development Program (UNDP) project in Brazil (7). This paper is devoted to developing a procedure, based on rod-and-level measurements of roadway profiles, through which the roughness standard provided by the profilometer can be transferred among different regions or countries. Furthermore, it is expected that the rod-and-level profile summary statistics presented here can be used to characterize pavement roughness over a wide range of wavelengths in a more reliable manner than can be done by using the SD profilometer.

USE OF PROFILE SUMMARY STATISTICS TO QUANTIFY PAVEMENT ROUGHNESS

The motion of a vehicle on a pavement results from the excitation of a dynamic system, the vehicle, by the vertical displacements of the pavement profile. If the parameters defining the dynamic system are known as well as the roadway profile, vibration theory can be used to determine the vertical movement of the vehicle at a given speed (8,9).

Most vehicle parameters (tires, suspension, body mounts, seats, etc.) are relatively similar. Moreover, on any particular road, most cars will be driven at similar speeds. Therefore, the excitations of the car, and thus the riding characteristics, become primarily a function of the road profile (2).