A Weakly Cemented Sand and Its Behaviour in a Model Pavement Structure

Etude d'un sable faiblement cimenté et de son comportement dans un revêtement routier modèle

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SUMMARY

This paper describes an investigation into the properties of weakly cemented sand with particular reference to this material's behaviour in a pavement structure. The sand used was a medium to fine sand, typical of the sands in the Perth area of Western Australia and the binder used was 4 per cent of Portland cement. The cohesion due to the binder was varied by changing the curing time from zero to seven days. This produced tensile strengths from about 10 psi to about 25 psi, with corresponding changes in unconfined compressive strength, stress-strain, and compressibility characteristics. It also provided means of varying the stiffness of beams which were tested on artificial subgrades consisting of closely spaced proving rings of various stiffnesses. Stiffness ratio in pavements is discussed and a concept of stiffness index for this type of granular pavement is introduced.

SOMMAIRE

On décrit une étude effectuée sur les propriétés d'un sable faiblement cimenté particulièrement en ce qui concerne son comportement dans un revêtement routier. Le sable utilisé se composait de sable moyen à fin, caractéristique des sables de la région de Perth en Australie Occidentale, et le liant contenait 4 pour cent de ciment Portland. La cohésion produite par le liant a été modifiée en faisant varier la durée de la cure de zéro à sept jours. Ceci a eu pour résultat de donner des résistances à la traction passant de 10 à 25 livres au pouce carré approximativement, avec des augmentations correspondantes de la résistance à la compression, des caractéristiques de compressibilité et de la relation contrainte-déformation. L'investigation a aussi fourni les moyens de faire varier la rigidité des poutres qui furent mises à l'essai sur des fondations artificielles constituées d'anneaux de mesure de rigidité variable et étroitement espacés. Le coefficient de rigidité des revêtements routiers est discuté et on présente un concept d'indice de rigidité pour ce genre de revêtement granulaire.

A GREAT DEAL OF RESEARCH has been undertaken in recent years into the fundamental properties of cohesionless granular materials. This work has ranged from studies of shear strength and compressibility characteristics to consideration of interparticle forces, and includes idealized models to explain observed behaviour. In practical application granular materials have been used extensively for the construction of flexible road and airfield pavements. Purely non-cohesive materials have been used for sub-bases and bases while, for surface courses, some form of binder has usually been added. This binder has varied from just sufficient to hold the grains together up to highly cohesive asphalts and Portland cements.

The most noticeable effect of adding the binder is that the non-coherent mass takes on the appearance of a solid and has the ability to resist tensile stresses. This enables it to function as a beam and to distribute the load by flexural action. The increased load-carrying capacity imparted to a pavement by the cohesion has led to concepts of equivalent thickness whereby a relationship is established between the thickness of stabilized material required, and the corresponding thickness of gravel. However the significance of the cohesion when it is small is not at all clear, since failure due to excessive tensile stresses, manifested in the form of cracking, will not necessarily lead to complete failure of the structure. The subgrade will be required to support more of the load, with the pavement tending to revert to its non-cohesive condition.

This paper describes tests and model experiments with a weakly cemented sand. They were designed to observe the effect of small amounts of cohesion on the strength characteristics and to study the material in the laboratory under simplified pavement-type stress conditions. This investigation has particular significance in areas such as Perth, Western Australia, where fine sandy soils predominate, and is part of a wider study of the engineering properties of these materials. It also has wider implications in its possible application to the stabilized-soil pavement design problem.

THE MATERIAL

The sand chosen for the experiments was a medium dune sand typical of the sands commonly used for concrete and mortars in the Perth area of Western Australia (Clegg, 1962). Its grading is as follows (per cent passing British Standard Sieve): No. 25, 85; No. 52, 16; No. 100, 2; 200, Nil. The nature of the predominantly quartz particles is indicated by sphericity number 0.78 and roundness 0.46 (Krumbein, 1941). The prevalence of this sand and the possible practical application of the results made the use of this material preferable to an artificially produced grading.

Various binders, such as sodium silicate, hydrated lime, bentonite, and Portland cement were considered. The latter was finally chosen for availability, ease of handling, and because use could be made of its rapid increase in strength with curing time to change the cohesion without changing the particle arrangement. The mix used for the experiments described in this paper consisted of 4 per cent cement, 4 per cent water, mixed in a small rotating-drum mixer, one batch per test specimen. Compaction was carried out with a kneading compactor to a relative density of 70 per cent.

Two types of specimen were prepared—8-in. high by 4-in.

diameter cylinders and 15 in. by 3¼ in. by 1½ in. beams. The moulds in each case were designed so that they could be stripped from the material immediately after compaction and the specimens carried on pallets to the curing room. The pallets for the beams had to be particularly smooth and flat. Quarter-inch thick mild steel plate, ground flat, was found to be suitable.

Curing was carried out at 21 C in a saturated atmosphere. Care had to be taken to prevent direct wetting of the newly moulded material as it was easily eroded. At the end of the desired curing time, which varied from zero to seven days, the specimens were transferred to a drying oven for eight hours at 105 C. After drying they were stored in polythene bags until required for testing. Extreme care had to be exercised in handling the low curing time specimens as they crumbled when touched and broke easily, particularly the beams.

THE EQUIPMENT AND TEST PROCEDURES

Triaxial cylindrical compression tests were used for the determination of c, ϕ, and E. The equipment conformed generally to that suggested by Bishop and Henkel (1962). As the material was dry and relatively porous it was possible to measure volume changes by the volume change in the pore air. Stress-controlled loading was used, each increment of load being held until creep appeared to have ceased—usually after a few minutes.

In the case of direct shear tests it was found possible to carve the specimens out of cylinders with a blunt hack-saw blade. These were tested for ultimate shear strength in a conventional strain-controlled shear box. The cylindrical specimens were particularly well suited to the indirect tensile test (Hondros, 1959). Fig. 1 serves to illustrate this method of test and also gives some indication of the nature of the material.

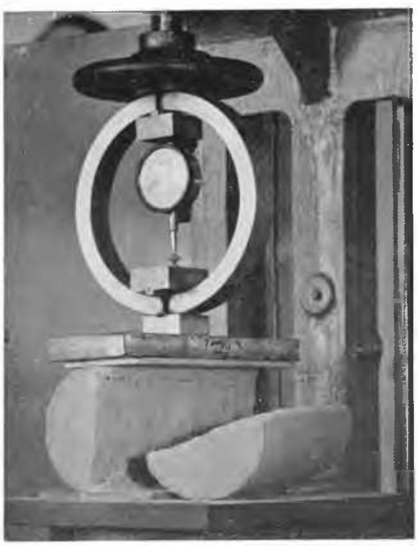

FIG. 1. The indirect tensile test.

The beam test arrangement finally used is shown in Fig. 2. It consisted of a row of proving rings cut from 16-gauge seamless steel tubing and mounted on adjustable bases. The deflection of the rings was measured indirectly through strain gauges, one inside and one outside at the point of maximum stress thereby providing maximum sensitivity. The load-transfer segments top and bottom were lightly held in place by set screws and formed a yoke rather than a narrow strip or knife edge type of load transfer device. This arrangement

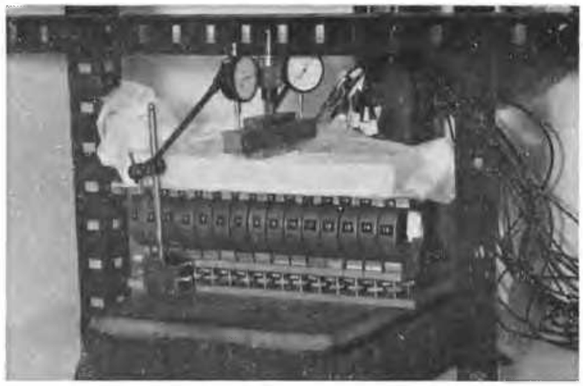

FIG. 2. The beam test arrangement.

increased the stiffness and reduced the sensitivity but gave increased stability. A thin sheet of rubber was placed on the segments to ensure proper seating of the beams to be tested. Calibration of the rings was first carried out separately, and then, as a group, by loading them together through a relatively stiff steel beam. Rings, 4 in. diameter by ⅜ in. width, 4 in. diameter by ¾ in. width, and 3 in. diameter by ¾ in. width gave stiffnesses of 0.0032, 0.0012, and 0.0004 inches per pound. The softest ring was considered to correspond to a coefficient of subgrade reaction of just under 100 pci, while the stiffest ring was considered to correspond to about 800 pci. This covered a range of subgrade soils from about CBR 3 to CBR 100.

Before deciding to use proving rings as the base, coil springs (Herner and Aldous, 1950), simple beams (Baker and Papazian, 1960), and a rubber tube device were considered, but the rings proved to be the most convenient to make and assemble. The whole apparatus was placed on 150-lb capacity platform scales, a frame bolted to the bench top being used to provide the necessary reaction through a capstan head. The dial gauges for deflection measurements were mounted on the platform of the scales by means of magnetic clamps.

The cemented sand beams were prepared for test by brushing with a soft brush to remove any excessively loose grains and then sliding them off the pallets onto a thin polythene sheet. This sheet was then used as a sling to carry them into place on the base as well as to prevent sand grains from falling down between the rings. Because of the rough nature of the top of the beams the centre section under the load was smoothed by filling the voids and surface irregularities with French chalk. This chalk was also used on the sides of the beams to aid in crack detection.

Before testing, the rings were adjusted by first placing a stiff steel beam on the base and then raising or lowering the rings to give approximately equal load on each. Switching difficulties with the strain-measuring equipment made it necessary to use a repeated loading procedure, changing to the next ring and zeroing at each load repetition. The maximum load used was 0.5 to 0.66 of the estimated failure load. After the fifteen load repetitions required to obtain the deflections of all the rings the deflection of the centre ring was observed while the load was increased to failure. This was indicated by cracking or sudden deflection accompanied by marked increase of pressure on the centre ring.

THE TEST RESULTS

An important feature of the test results is the effect on the various strength properties of changing the curing time. It was found that between zero and three days' curing there was a very considerable increase in strength. At zero curing time the material was too weak to handle so five hours was generally used as the minimum, particularly with the beams.

A typical result of a triaxial test is shown in Fig. 3. The volume change gauge showed compression until dilation and shearing commenced leading to complete failure. The volume changes under the application of the cell pressures provided the data for the pressure void ratio curves shown in Fig. 4, while Fig. 5 shows the effect of curing time on cohesion and internal friction. The relationships between compression modulus, tensile strength, direct shear strength, and curing time are shown in Fig. 6.

The beam tests provided both load/deflection and pressure distribution data. A typical load/deflection relationship

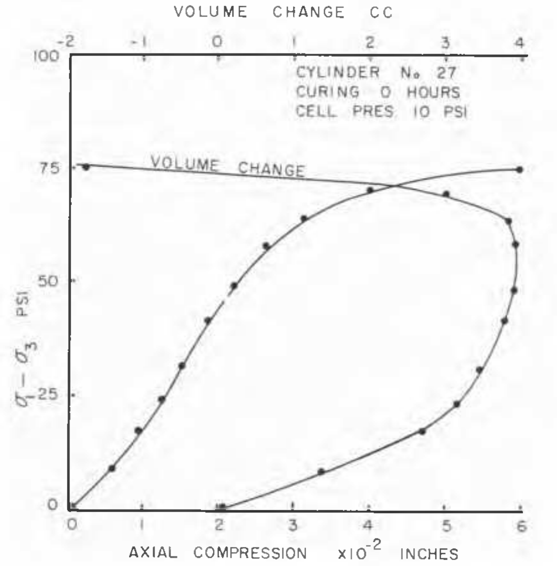

FIG. 3. A typical triaxial test result.

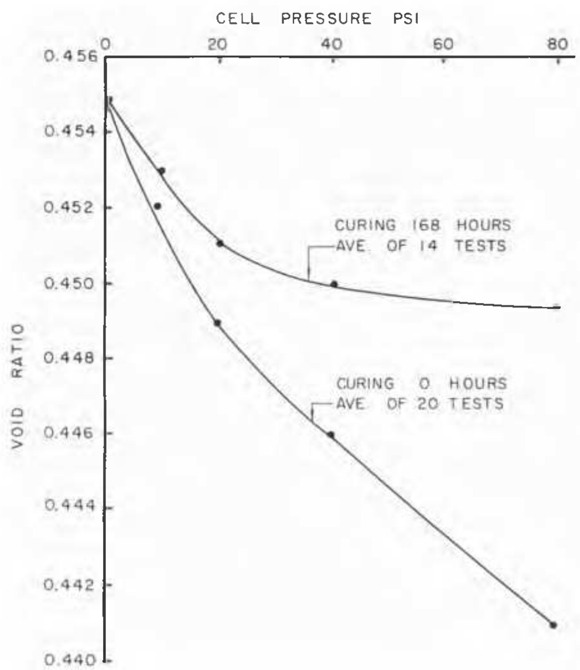

FIG. 4. Typical pressure void ratio curves showing the effect of curing time.

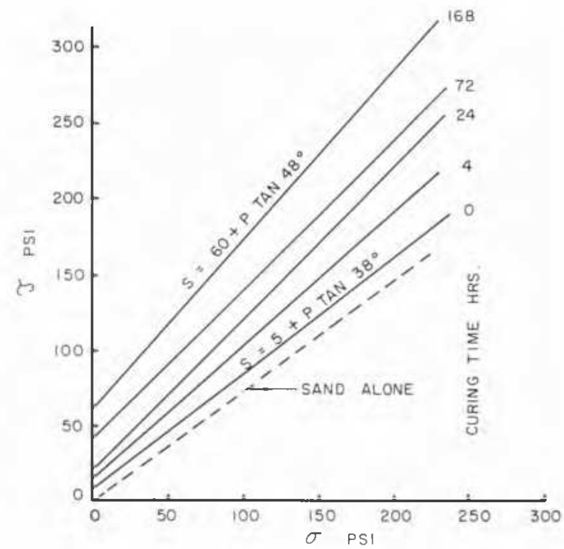

FIG. 5. The effect of curing time on cohesion and internal friction.

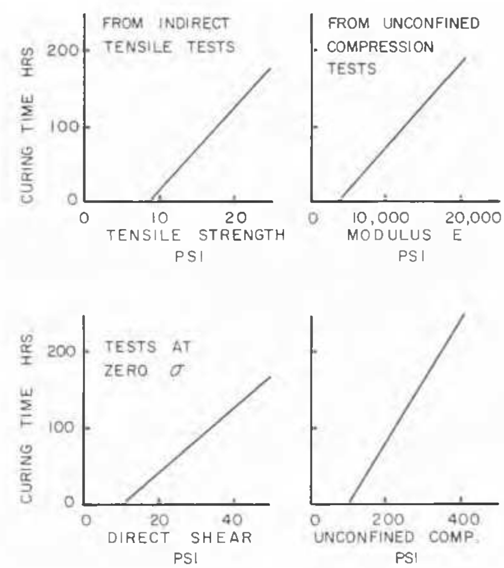

FIG. 6. The relation between curing time and strength properties.

is shown in Fig. 7. Fig. 8 illustrates the effect of relative stiffness on the type of pressure distribution and Fig. 9 the change in distribution when the beams crack compared with the distribution through sand without a binder.

DISCUSSION

The test results presented serve to illustrate the basic physical properties of the material. The addition of binder, while adding cohesion, also slightly increases the friction angle. The compressibility characteristics are similar to those of sand but the tensile test indicates a brittle nature. Another phenomena observed during the test, but not herein recorded, is that of long-term creep and the associated effect of repeated loading. Microscopic examination showed definite clustering of particles with ample larger voids into which particles could be projected, so precipitating a chain reaction of particle re-adjustment.

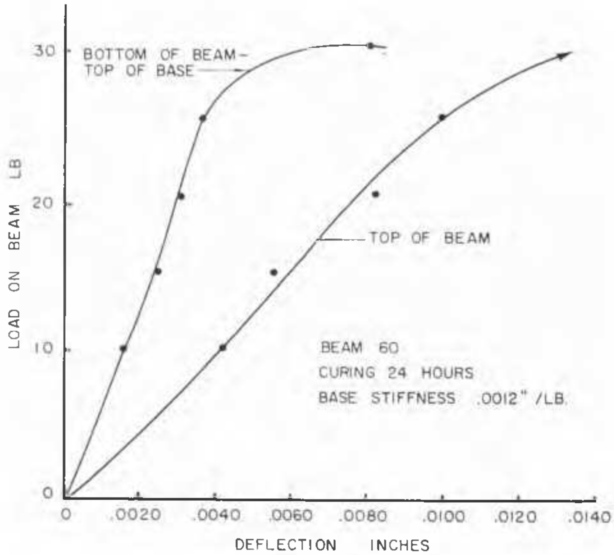

FIG. 7. Beam experiments: typical load-deflection relationships.

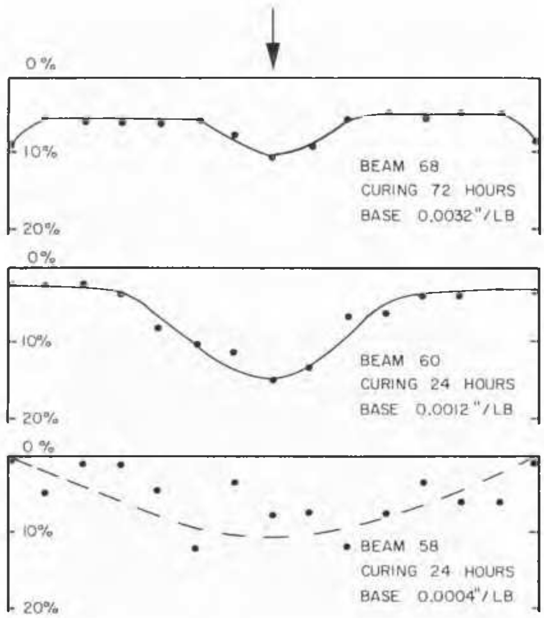

FIG. 8. Beam experiments: pressure distributions for different relative stiffnesses.

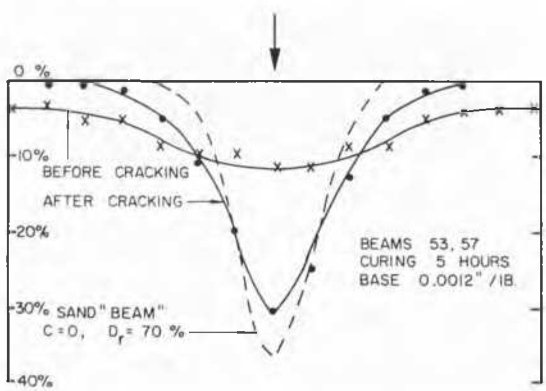

FIG. 9. Beam experiments: the effect of cracking on pressure distribution and the distribution through uncemented sand.

Before considering the significance of the beam tests it is appropriate to review briefly the basis for the "stiffness ratio" concept of pavement behaviour. As described by Baker and Papazian (1960) this term embraces "radius of relative stiffness" as used by Westergaard wherein rigidity of the slab is compared with subgrade support. It is also similar to Burmister's ratio E_2/E_1 where E_2 and E_1 are Young's moduli for the subgrade and pavement respectively. These two approaches produce somewhat different solutions to the stress and deflection of a pavement system, the differences being mainly due to assumptions regarding the nature of the subgrade. Westergaard assumes a discontinuous foundation similar to a bed of closely spaced springs, the deflection at every point being proportional to the reactive pressure which is vertical with no shear strains; Burmister assumes a completely continuous foundation material with full shearing strains at the interface. Assuming an elastic soil, a rigid beam produces a uniform distribution of vertical strain with high pressures at the ends. A flexible beam produces a parabolic type of distribution with a maximum under the load. The type of stress distribution and hence the bending moments in the beam depend on the stiffness ratio.

The experimental model described in this paper is not strictly the series of independent springs required by the Westergaard analysis. The necessary inclusion of the rubber and thin plastic sheet for practical reasons and the horizontal stresses at the interface produce indeterminable conditions. However, the concept and possible significance of stiffness ratio is not changed. The experimental results give the appearance of a "flexible distribution" superimposed on a "rigid distribution." If a uniform distribution is considered to be 100 per cent rigid, and the flexible type of distribution as 0 per cent, it is possible from the relative proportion of each type of distribution present to estimate a numerical "stiffness index" for the particular combination. For example the pressure distribution for Beam 60 in Fig. 8 can be considered as a uniform pressure approximately equal to that at the ends of the beam plus a bulb in excess of this under the central section. The area under each of these sections of the diagram is 490 units and 510 units respectively which leads to a stiffness index of 49 per cent for this particular case.

The material under examination is weak in tension, relatively compressible, and basically frictional in nature. Without a binder it can distribute the load only by frictional action, and it might be thought of as a purely granular beam without stiffness. However Hveem and Sherman (1962) consider that a layer of granular material has "stiffness" by virtue of its own resilience and its ability to spread the load, thereby reducing deflections. This apparent stiffness is a function of frictional properties of the material and the external restraints. The weakly cemented sand is likely to lose some or all of its structural integrity at even relatively small proportions of the complete failure stress and will tend to revert to its basically frictional character. The effect of loss of structural integrity through development of cracks is illustrated in Fig. 9. The distribution of pressure through a beam of sand was obtained by using a wooden frame with a rubber bottom to contain the sand which was placed to approximately the same relative density as the cemented beams. This granular beam failed by lateral flow and penetration of the loaded strip, whereas the cemented beams and particularly the strongest ones failed suddenly with a more or less vertical break at the edge of the load. With the intermediate category, having indirect tensile strengths of about 10 psi, failure cracks first appeared on the underside of the beams away from the centre and generally in a diagonal direction. This cracking corresponded to the point of curvature in the load-deformation graph (Fig. 7) and did not mean complete collapse but rather a redistribution of the load. While the method of failure of the strongest beams was not noticeably affected by the base stiffness, the weaker beams were very susceptible to variations in base stiffness, generally failing suddenly on the softest base.

It appears then that there are two limits to the behaviour of the weakly cemented sand as observed in these model experiments. The lower limit is when there is no binder and the pressure transmitted to the base is concentrated under the load, failure taking the form of lateral flow with downward penetration of the load and a wedge of the material similar to that indicated by the Prandtl theory for footings. The upper limit is indicated when the beams are sufficiently strong in relation to the base that the pressure transmitted to the base approaches a linear distribution, failure taking the form of a sudden break suggestive of failure in tension due to bending. Between these two limits there is a combined effect which gives the appearance of the "flexible" distribution superimposed on a "rigid" distribution referred to earlier. However it does not seem reasonable to consider this as having any theoretical basis, such as the stiffness ratio concept from elastic theory, when a prime requirement is for the material to develop continuity of stress. On the other hand the concept of a stiffness index would not be so unreasonable. Zero per cent would indicate that the behaviour is essentially that of a granular material and 100 per cent would indicate that the material has sufficient cohesive strength and receives sufficient support from the base to give a more or less uniform distribution of pressure, although this may take the form of a shallow dish as suggested in the case of Beam 58, Fig. 8. Rather than consider 100 per cent as indicative of uniform distribution it might be better to consider this as representing the condition above which theoretical treatments requiring continuity in the materials may be applied.

The stiffness index concept introduced as a possible way of describing the behaviour of the weakly cemented sand in these model experiments may have useful practical applications. Clegg and Yoder (1963) have suggested a structural classification of pavements based on relative stiffness and covering three groups—frictional, complex, and flexural, in order of increasing relative stiffness. The frictional group ranges from gravel through to modified bases. The flexural group covers stabilized bases with asphalt surfaces and, in the extreme case, Portland cement concrete while the complex middle group includes a wide variety of higher quality type of flexible pavements depending on both frictional and flexural characteristics to distribute the load. The principal design considerations are determined from the grouping and allowances can be made for any stiffness that may be imparted to a granular pavement through the addition of stabilizing material. Applying the idea of a stiffness index the gravel would be rated as 0 per cent and the pavement combination in which flexural strengths are considered to be the prime design consideration would have an index of 100 per cent. Just in what way the 100 per cent situation would be estimated in practice still remains to be determined, but the indications from the experimental work are that it could be related to the pressure distribution and hence the profile of the deflection basin of the subgrade. It might also be possible to use the surface deflection profiles from Benkelman beam and other similar techniques currently used in the field. Further experimental work with the model may indicate the appropriate strength parameters to use in determining the stiffness index but other factors such as end restraint of the beams, friction between the beams and the base, width of the loading strip, and thickness of the beams will need to be considered. In the field such factors as cracking due to temperature and shrinkage will also need to be considered in arriving at the estimate of stiffness index and hence gaining some indication of the ability of the particular pavement to behave as a continuous structural unit.

CONCLUSIONS

1. The effect of binder strength on the properties of a weakly cemented sand has been conveniently examined using the technique of changing the curing time of the Portland cement binder. By this means binder strengths, measured in terms of indirect tensile strengths from about 10 psi to 30 psi, were produced and the corresponding changes in modulus, direct shear strength, and unconfined compressive strength were determined. Triaxial tests indicated a slight increase in friction angle over this range of binder strength.

2. The use of a pavement model consisting of a base of small proving rings to represent the subgrade appears to have been a convenient way of studying weakly cemented sand under simplified pavement-type stresses. From experiments with beams on this base some indication of the effect of variations in binder strength and subgrade stiffness on the pressure distribution and failure characteristics was obtained. To aid in describing this behaviour the concept of a stiffness index was introduced, 0 per cent indicating a purely granular pavement and 100 per cent indicating a pavement that is essentially a continuous structural unit. It is concluded that this index might have useful practical value when applied to the description of modified or stabilized pavements for classification purposes.

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