THE USE OF AN IMPACT ROLLER IN COMPACTING A COLLAPSING SAND SUBGRADE FOR A FREEWAY

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The problem of differential settlement on roads in southern Africa has occurred on certain silty sands with high void ratios, due to the "collapse" of the soil structure. In comparative field trials of possible solutions to this problem, a flat-sided impact roller was found to cause consolidation to greater depth than vibrating or pneumatic rollers. This impact roller was later used for the compaction of a silty sand subgrade under a new freeway near Witbank. Three deep profiles were then studied: under the freeway where the impact roller had been used, under an old existing asphalt road, and in the undisturbed natural condition. The variations with depth of density, moisture content, and penetration resistance were determined. "Double oedometer" tests were carried out in the laboratory on undisturbed samples, making possible the prediction of collapse settlement on inundation under various loads. It was found that the state of compaction achieved by the impact roller was sufficient to prevent any later collapse settlement with normal traffic loads under conditions of inundation. Further, it was concluded that special measures should be taken to compact such subgrades to a depth of 1.4 meters (4 ft 6 in.), in order to prevent collapse settlement under heavy traffic.

•IT has been known for some time that structural damage to buildings and roads in southern Africa occurs on certain silty sands possessing high void ratios (commonly known as "collapsing sands") because of the "collapse" of the soil structure. There have been several publications since the first by Jennings and Knight in 1957 (1), but the nature of the problem is not fully understood. The full scope of the problem in the Transvaal province alone is not known, but difficulties could well be experienced in some parts of those areas of sandy soils containing iron oxides, which are referred to on the soil map of southern Africa as "lateritic soils" or "ferruginous lateritic soils." There are many other parts of the Republic where loose windblown sands exist to considerable depths, particularly in the northern Cape, Orange Free State, coastal Natal, most of South West Africa, and Botswana. The severity of this effect on a major road was referred to by Knight and Dehlen in 1963 (2) when it was shown that a settlement at the surface of about 150 mm (6 in.) had occurred and that there was evidence of some densification under traffic to a depth of about 1.3 meters (4 ft 6 in.). It had been noted by Williams (3) that any soil with a dry density of less than about 1,600 kg/m^3 (100 lb/ft^3) could give rise to difficulties in settlement and that if a density of 1,450 kg/m^3 (90 lb/ft^3) existed, the possibility of collapse was very likely.

When the layer of windblown sand is less than a meter thick, there is little problem in carrying foundations for a building down to a hard stratum, or in compacting this layer for a road foundation. When the deposit is thicker than this, some special treatment is required in both cases. In other parts of the world deep loessial deposits give rise to similar problems, although the material is much finer than these uniform sands.

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THE ROAD PROBLEM

The construction of a new freeway traversing areas of collapsing sand in the Witbank area 100 km (60 miles) east of Pretoria was begun in 1968. The section of freeway concerned is 16 km (10 miles) long and accommodates 3,000 to 4,000 vehicles per day. There are two traffic lanes in each direction, and the shoulders are surfaced. The value of the 3-year contract awarded for construction was R5.5 million ($7.7 million). The pavement has been designed by the interim AASHO method to a "structural number" of 4.5 and is composed of the following layers:

1. 50 mm (2 in.) of asphaltic concrete;
2. 150 mm (6 in.) of cement-treated crushed stone base, 5.0 MN/m^2 (750 psi);
3. 150 mm (6 in.) of cement-stabilized lateritic gravel subbase, 1.7 MN/m^2 (250 psi);
4. 150 mm (6 in.) of imported subgrade compacted to 95 percent modified AASHO density; and
5. 450 mm (18 in.) of in situ or imported sand subgrade compacted to 95 percent modified AASHO density.

At the time the contract was drawn up, it was recognized that there might be a problem of possible settlement; however, the only proven method of compacting the in situ sand subgrade was by removing 600 mm (24 in.), compacting the exposed surface with a heavy pneumatic or vibrating roller, replacing the excavated material, and then compacting this to 95 percent modified AASHO density. This procedure proved almost unworkable in practice. The scrapers repeatedly got bogged down, often due to the high water table in some areas, and even the lowering of the water table by means of drains had relatively little effect.

There was thus not only the immediate practical problem of how to achieve compaction economically but also the doubt about what depths and limits on density should be specified in this deep loose sand subgrade.

FIELD INVESTIGATION

The foregoing problems had received little attention in South Africa until during the course of the Witbank road contract some evaluation trials were carried out on a new type of pentagon-shaped roller that appeared to be a very feasible solution to the problem of compaction in depth. The action of the roller (Fig. 1) delivers a series of impacts as its 11,000-kg (24,000-lb) mass drops about 200 mm (8 in.) onto a rectangular area 1.2 by 0.6 meter (4 x 2 ft). While operating at about 8 km/h (5 mph), this applies impact loads every 0.75 meter (30 in.) at a frequency of about 3 cycles per second. The results of this work reported by Clegg in 1969 (4) showed conclusively that the impact roller achieved an appreciably better effect than does a heavy pneumatic or vibrating roller and had a high work output. In the course of this work, it was found that the use of a small cone penetrometer proved very useful for following the progressive change in condition of a deep sand subgrade as rolling proceeded. Some further studies were reported by Williams in 1970 (5); these confirmed that the effect of the impact roller was much greater down to a depth of about 1.5 meters (5 ft) than any of the other rollers and gave information on the pressures generated through the soil profile.

It was not known, however, if the state of compaction achieved under the impact roller was sufficient to prevent collapse settlement later with normal traffic loads as a result of inundation. The problem could be framed differently: to enquire whether the over-consolidation achieved by the impact roller at the field moisture content would prevent further compression of the sand on inundation under the traffic loading. A further field and laboratory study was thus planned to seek information on these points.

Figure 1. The impact roller about to deliver a blow.

Three sites were selected for the study. They were located some distance east of where the roller trials reported by Clegg had been carried out. Site A was considered to be undisturbed by any heavy equipment because it lay in an old orchard. Site B was under the old road, which had carried normal traffic for more than 25 years. Site C was on the new freeway, complete except for base and surfacing. The selected subgrade layer had been subjected to twelve passes of the impact roller. It had been observed by the resident engineer that during construction considerable additional settlement of the surface had occurred when the impact roller was used after normal rolling (6).

It was intended that a comparison be made between conditions existing before any treatment was given to the soil and those finally obtained under the impact roller. It was also hoped that the conditions under the old existing road would reveal whether densities such as those produced by many years of trafficking were attained by the use of the impact roller on the selected subgrade.

The dynamic cone penetrometer described by van Vuuren in 1969 (7) had been found to give a useful indication of the state of compaction of the collapsing sand after various passes of the impact roller. This device simply consists of a 10-kg mass falling through 460 mm onto a 20-mm diameter point. The penetration resistance of the point with depth was therefore determined at each site. At each site, two test pits 2.1 by 2.75 meters (7 by 9 ft) were excavated carefully by hand to a depth of 1.8 meters (6 ft). The soil profiles of the six holes are shown in Figure 2. Before excavation was started, the dynamic cone penetration resistance was measured to a depth of 2 meters (6 ft 6 in.) at each of the four corners of the test pits.

At vertical intervals of 300 mm (1 ft) the in situ density was determined by both the sand replacement method and a Hidrodensimeter nuclear moisture-density gage. Samples for moisture content determination and laboratory testing were also taken. Subsequently, undisturbed block samples were cut from the walls of the test pits for oedometer testing. The depths at which the samples were taken are shown in Figure 2.

RESULTS OF FIELD WORK

The variations in natural moisture content and dry density with depth are shown in Figures 3 and 4. The variation with depth of dry density as a percentage of standard AASHO is shown in Figure 5. The in situ "estimated CBR" values as determined with the dynamic cone penetrometer are shown in Figure 6.

ROUTINE LABORATORY TESTS

On each sample taken at 300-mm (1-ft) intervals in each hole, the moisture content was determined by oven drying, a grading analysis was carried out, and the moisture-density relation was determined.

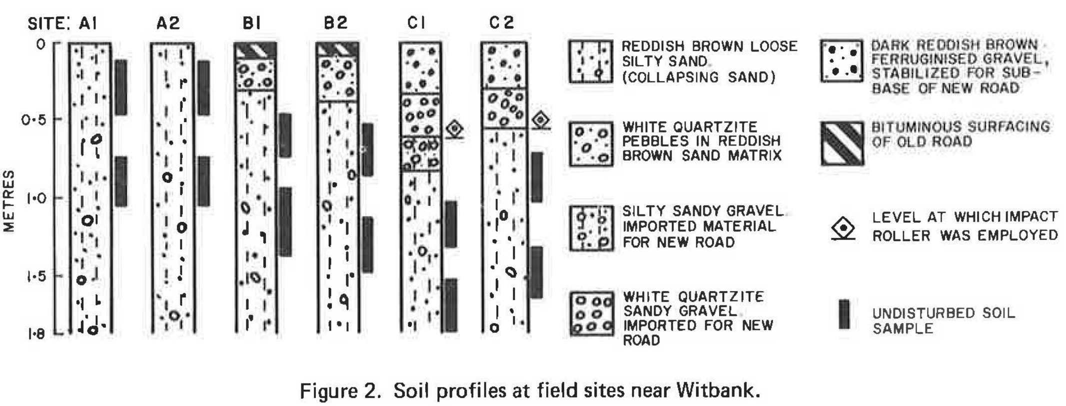

Figure 2. Soil profiles at field sites near Witbank.

The particle size distribution of all samples fell within a fairly narrow envelope, as follows:

U.S. Sieve Size	Percentage Passing
4	99 to 100
10	90 to 99
40	65 to 81
200	16 to 38

There was a tendency for the finest fraction to increase with depth, and the plasticity also increased from "non-plastic" in the upper horizon to a maximum plasticity index of 13 at a depth of 1.8 meters (6 ft).

Specific gravity was measured on the samples, and the tests gave an average specific gravity of approximately 2.66.

In order to throw new insight into the state of the material either in its natural condition or after compaction, it was thought that the use of Burmister's "relative density" concept (8) might prove rewarding. This term should not be confused with relative compaction, which is a ratio between the actual density and some maximum density achieved by standard methods such as modified AASHO. The relative density of Burmister is defined as

$$R.D. = \frac{e_L - e}{e_L - e_D}$$

where

e_L = void ratio of the soil in loosest state,
e_D = void ratio of the soil in densest state, and
e = void ratio of soil in question.

Tests were thus carried out by following the method specified by Akroyd (9). Method III.I. (ii) (2,000-ml cylinder method) was used for the minimum density, and method III.H. (i) (standard compaction mold method with vibration) was used for the maximum density.

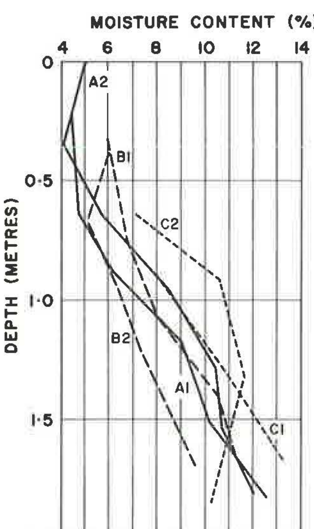

Figure 3. Natural moisture content profile.

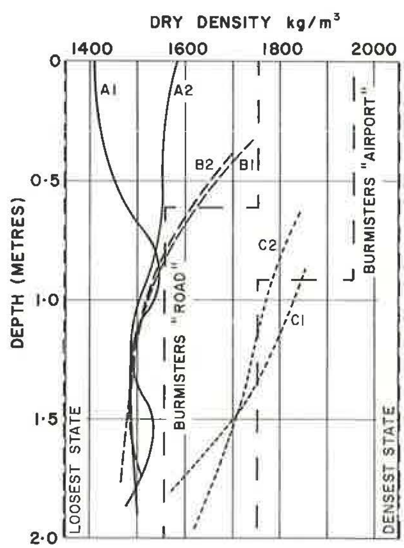

Figure 4. Dry density against depth.

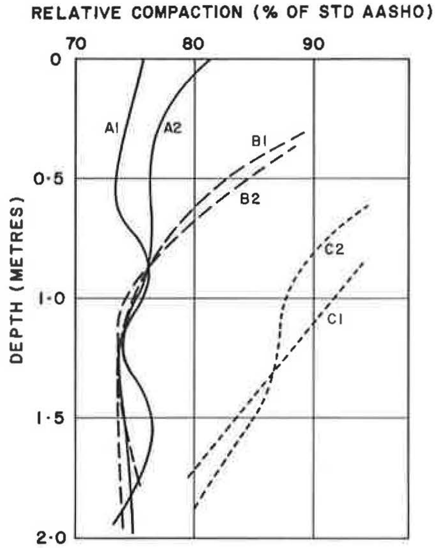

Figure 5. Relative compaction with depth.

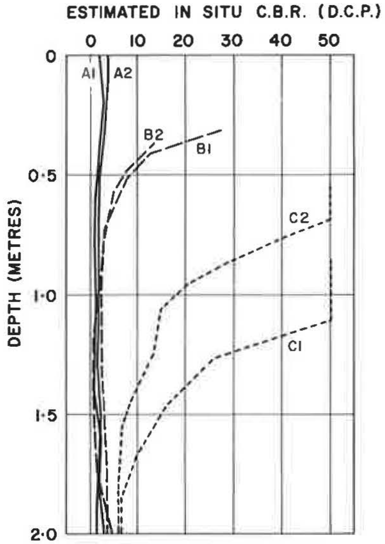

Figure 6. Estimated CBR values with depth.

The results may be summarized as follows: The minimum density averaged 1,365 kg/m^3 (85 lb/ft^3), a void ratio of 0.95, and the maximum density averaged 2,050 kg/m^3 (128 lb/ft^3), a void ratio of 0.30. There was very little difference in the values obtained between tests on all the six test pits. The values are shown in Figure 7, and this relative density diagram now allows one to read off the relative density for any value of soil density that may be encountered. This will be referred to later when comparisons are made between various field data.

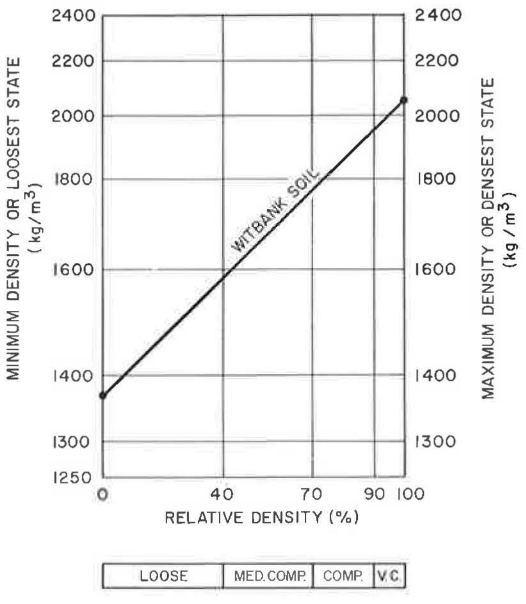

Figure 7. Relative density diagram.

DOUBLE OEDOMETER TESTS

Several oedometer tests were carried out on materials, in both wet and dry states, during the first investigation to study the consolidation characteristics, and the results can generally by summarized as follows:

1. Compression index, C_c = 0.20 to 0.30;
2. Preconsolidation load, P_c, dry = 70 to 90 KN/m^2 (10 to 13 psi); wet = 17.5 to 70 KN/m^2 (2.5 to 10 psi);
3. Initial void ratio, e_o = 0.85 to 0.95; and
4. Void ratio at 4,425 KN/m^2, e_f = (say) 0.30.

For the sites nearer Witbank further "double oedometer" tests were carried out, and typical results are shown in Figure 8. In these tests, undisturbed samples for testing in standard oedometers were trimmed into molds about 75 mm in diameter and 20 mm thick (3 in. in diameter, 0.75 in.

thick) from a larger block of soil carefully cut out in the field test pit. Initially the method of testing first suggested by Jennings and Knight (1) was used, in which two similar samples are loaded—one at the natural moisture content and one inundated with water. It was found that there was often little difference between the compression curves obtained, presumably because the natural moisture content was fairly high (about 10 percent in some cases) following rain. Because the potential "collapse settlement" was not clearly demonstrated, it was decided to allow samples to dry out in the laboratory atmosphere (about 50 percent relative humidity) and then carry out the two tests. This treatment showed up the potential collapse very clearly. It was noted that many of the in situ void ratios approached 0.95, which is equivalent to a density of 1,365 kg/m^3 (85 lb/ft^3), and that under the highest pressure possible of 4,425 kN/m^2 (640 psi), the void ratio approached 0.30, which is equivalent to a dry density of 2,050 kg/m^3 (128 lb/ft^3).

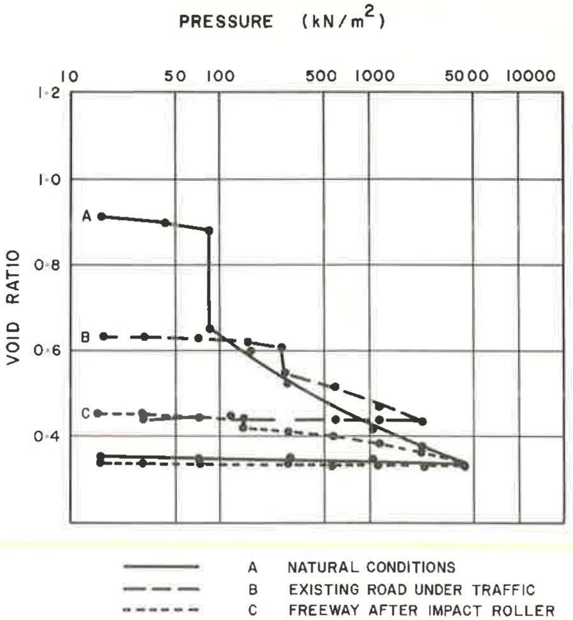

Figure 8. Typical double oedometer tests.

In general, three samples from each of two levels in the six pits were tested. One sample was loaded in the dry condition in increments up to 4,425 kN/m^2 (640 psi). The second was loaded to an applied pressure estimated from the pneumatic roller curve in Figure 9 for that depth and was then inundated before completing the loading cycle. The third sample was loaded to a pressure estimated from the impact roller curve in Figure 9 for that depth. This figure was obtained (5) from the results of a previous installation containing miniature earth pressure cells, and it will be noted that the impact roller generated a pressure of about 200 kN/m^2 at 1 meter depth (30 psi at 3 ft) compared with the heavy pneumatic roller giving 70 kN/m^2 at that depth (10 psi at 3 ft).

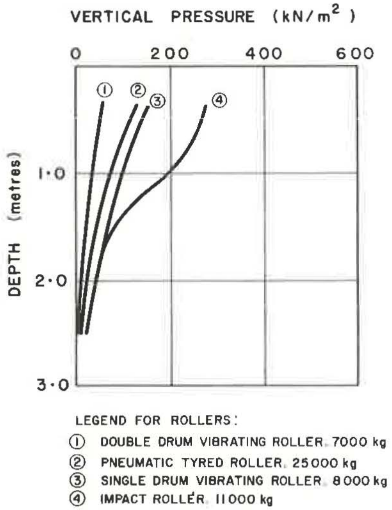

Figure 9. Vertical earth pressures under various rollers near Witbank.

A very good idea could thus be obtained of the settlement behavior between the dry and saturated condition under different loadings that were near the range of loading that might be experienced under traffic.

INTERPRETATION OF TEST DATA

An idealization of the compression characteristics of the Witbank sands is shown in Figure 10; the characteristics are very similar to those described by Knight (10). He had also found that a family of compression curves could be obtained between the dry and completely wet conditions, depending on the mositure content. When a "critical moisture content" of about 15 percent was exceeded, however, the material no longer exhibited a sudden "collapse" but followed the virgin compression curve.

The potential collapse settlement of any element of soil beneath a road can be assessed (Fig. 10) once the void ratio of that element and the pressure to which it will be subjected by traffic loads are known. The void ratio at any depth in the soil profiles can be obtained from Figure 4; a rough approximation of the pressure distribution under a heavy wheel load may be obtained from curve 2 in Figure 9 for the pneumatic-tired roller. In this way the upper points shown in Figure 10 can be plotted to represent conditions under the old existing asphalt road (site B). The lower points in Figure 10 represent the void ratios that are achieved at similar depths after compaction with the impact roller (site C).

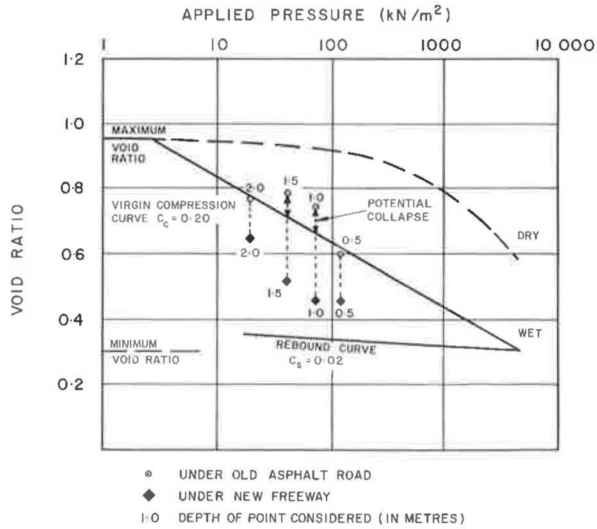

Figure 10. Compression diagram for typical collapsing sands at Witbank.

Thus, for example, at a depth of 1.0 meter (3 ft) beneath the old road the void ratio was 0.77. The material at this point might have been subjected to a pressure of about 70 kN/m^2 (10 psi), and reference to Figure 10 shows that this could have collapsed on inundation to a void ratio of 0.66 at the same pressure. At a similar depth of 1.0 meter under the new freeway, the impact roller would have preconsolidated the material to a void ratio of 0.46. Therefore no further settlement, or collapse, would occur under the pressures of about 70 kN/m^2 that might be generated by traffic. This is much the same for the points at other depths shown; therefore, the impact roller shows great promise for treatment of collapsing or, rather, highly compressible sand subgrades.

DISCUSSION OF RESULTS

It is thought that the use of the term "collapsing sand" may not be fully understood by some people in that, if the sand is wet, the actual phenomenon of sudden settlement (or collapse) will not occur. It should be emphasized, however, that considerable compression will still take place if the material is already wet. A better way to convey this meaning to those not fully aware of the subject, therefore, may be to emphasize that these loose windblown sands are very compressible. A coefficient of compressibility, C_c, of 0.2 to 0.3 is similar to that of some remolded clays, and it will thus be realized that appreciable settlement can occur purely through an increase in load under high moisture content conditions without complete inundation of these sands.

It has become clear from the relative density tests that the condition of the collapsing sand subgrades in the natural state approaches the loosest state of packing that can be achieved and that this condition persists for considerable depth. In regard to the depth to which compaction should extend, the early work of Burmister (8) suggested limits for roads and airport runways, which have been indicated in Figure 4. Further, it would appear that the recommendations made by Foster and Ahlvin (11) would also be applicable to collapsing sands and, in fact, are readily achieved by treatment with the impact roller.

CONCLUSIONS

From this and previous experience it would appear that specifications should cover the state of compaction to be achieved to depths of at least 1.4 meters (4 ft 6 in.) below final road level. With reference to the actual conditions on site, it is apparent that the impact roller has given a relative density that is higher than that achieved after many years in the field when a road has been exposed both to heavy wheel loads and to conditions of wetting and drying through the normal climatic seasons. Further, the impact roller does offer economic advantages over other methods for achieving great depth of compaction in collapsing sands.

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