# The behaviour of concrete block paving under repetitive loading

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## Introduction

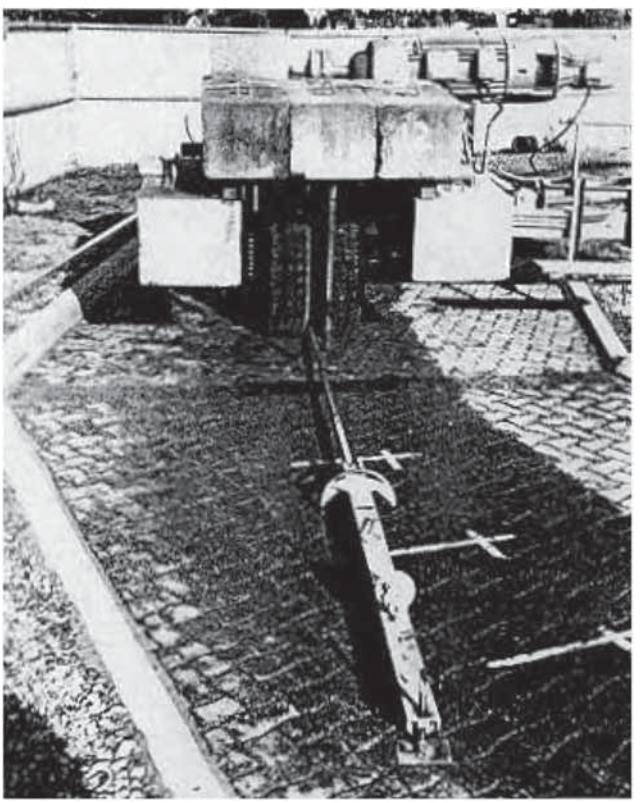
INTERLOCKING concrete block paving is being introduced in New Zealand, at present, mainly in private access roads and heavy goods areas The NZ Portland Cement Association[1] [2] has introduced a block specification and a code of practice, the background to which is discussed by Tait[3].

In 1977, the NZ National Roads Board sponsored a project to study the effects of particle shape and grading on the performance of unbound granular base material. The project was carried out on the University of Canterbury's Test Track at Shipley's Road, Christchurch and reported by Seddon[4]. The early failure of one section of basecourse and its replacement by concrete blocks gave the opportunity to study this material's performance alongside conventional basecourses.

The Canterbury Test Track has been described in detail by William and Paterson' so a brief summary is all that is required here, The two test vehicles consist of twin 9.00 x 20 truck wheels, loaded to 40 kN, travelling at approximately 16 km/h round a circular track of mean radius 9.2 m (Figure 1). One vehicle is driven by an 8.0 kW asynchronous electric motor, the other by the radial arm. Acceleration to full speed is achieved in about 40 s in three stages through a stator/rotor unit, and power is supplied to the motor through sliprings at the central pivot from the switchboard in the control room. An inching motor allows the vehicles to be moved slowly back and forth for Benkelman Beam tests and the like. A spring loaded brake is automatically applied when the power is switched off. bringing the vehicles to rest in approximately 20 s. At the central pivot, a 17 tooth pinion travels round a 79 tooth gear with 0.305 m eccentricity causing each vehicle to follow a curtate epicycloidal path repeating itself after 96 revolutions and thus trafficking an annular path approximately 1.25 m wide. The lateral distribution of wheel passes is approximately sinusoidal, which is a satisfactory simulation of that found on a public highway.

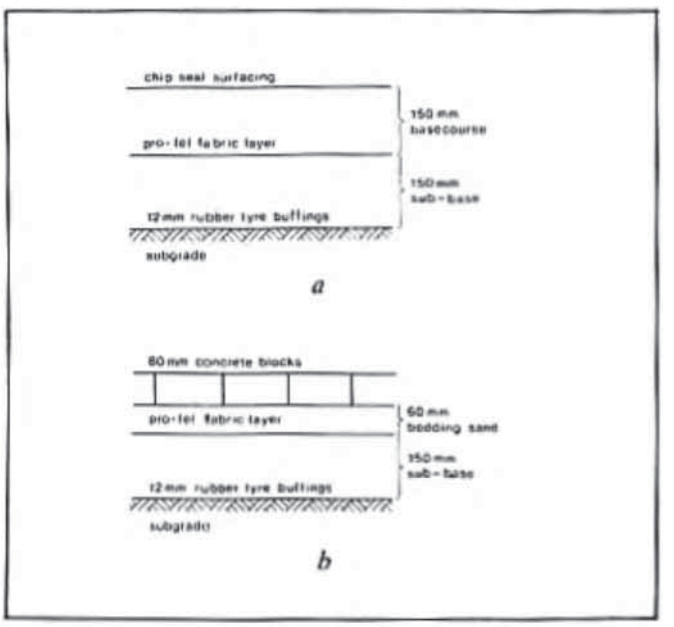
## Experimental work

At the start of this experiment, the basecoursc project was in progress with a cross section illustrated in Figure 2a. The subgrade at the test track site consists of greywacke gravel deposited by the Waimakeriri River which flows across the Canterbury Plains. In its natural state this material has a modulus much too high for rapid assessment of pavement constructions. Consequently, it had been covered by a layer of Pro-Fel (a non-woven filter fabric), 12 mm of rubber tyre buffings and another layer of Pro-Fel. These tyre buffings, a waste product of the tyre re-tread industry, arc tyre rubber in dusty flakes with the consistency of pipe tobacco. Although they achieve the objective of giving the overlying layers a high deflection, the disadvantages are that the shape of the deflection bowl is unrealistically steep and that it is not easy to assess the effective modulus for design purposes. The Pro-Fel layers were used solely to retain the rubber fianicies and to mark the boundaries for subsequent investigations. Overlying this was 150 mm of gravel subbase, 20 mm maximum size and very little passing the 75 mm sieve. Another layer of Pro-Fel to mark the boundary, 150 mm of nine different



*Left: Figure 1: Measurement of surface deflection using the Benkelman beam*

*Below: Figure 2: a, structure of unbound granular pavements; b, structure of concrete block pavements*



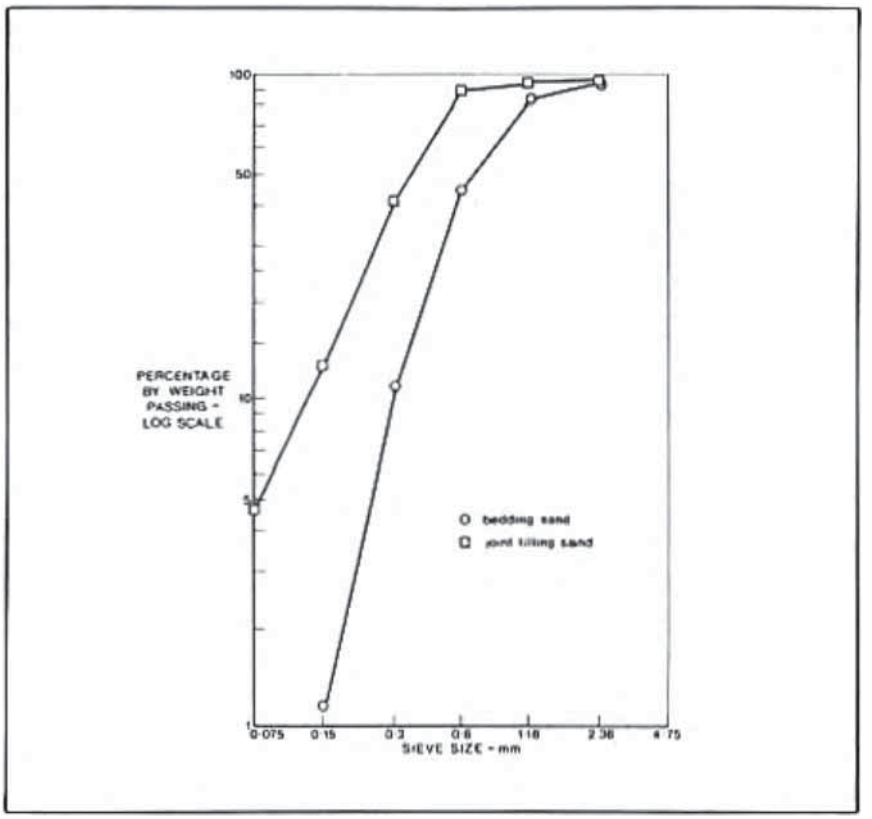



*Figure 3: Grading curves for bedding sand (Birdling's Flat) and joint-filling sand (Mount Somers)*

unbound granular basecourse materials, and a single coat surface dressing completed the pavement. The nine sections were 6.4 m long with a total construction width of 3.0 m.

At 150,000 wheel passes, one of the sections became totally unserviceable, so the basecourse and surface dressing were removed and replaced by two types of 80 mm thick concrete blocks on 60 mm of bedding sand as illustrated in Figure 2b. One block was of the Pavelock pattern manufactured by Winstone Ltd and the other a Unipavc manufactured by Firth Industries Ltd. both of which can be seen in Figure 1 The bedding sand was a beach sand from Bird lings flat, south of Christchurch, with the grading shown in Figure 3. The blocks were laid by the manufacturers in the conventional manner by loosely screeding the bedding sand, laying the blocks in herringbone pattern and compacting with a vibrating plate compactor (Figure 4). edge restraint was provided with precast concrete lighting columns bedded and backed with surplus basecourse aggregate from the site. The sand from Mount Somers, with a grading shown in Figure 3. was brushed and vibrated into the gaps between the blocks.

At frequent intervals during the trafficking, deflections (ie. resilient deformations) under the 40 kN wheel loads were measured by Benkelman Beam at three points on the centreline of the trafficked path for each type of block.

At the same time and at the same points as the deflection measurements, the transverse profiles were recorded. This was achieved by measuring the depth to fixed points on the surface from a straight edge resting on rigid bench marks located outside the edge restraints.

When trafficking was terminated at 100,000 wheel passes, the shape of the deflection bowl was measured and compared to those for the basecourse pavement. For these, a Benkelman Beam was used which had the dial gauge supplemented by a linear variable displacement transducer connected to a chart recorder. Movement of the vehicle was measured by a line wound round a drum, the rotation of which was also recorded on the chart.

At the same time, some of the blocks were carefully removed, and the in-situ dry densities of the bedding sand were measured at two places under the trafficked path and others outside this area. A balloon densometer was used for these tests. In addition, checks were made for permanent settlement of the subbase.

In view of the substantial permanent rutting of the block pavement, which appeared to be solely due to settlement or movement of the bedding sand, a series of laboratory tests was carried out on this material. These included:

(a) Optimum moisture content. For this, a 150 mm diameter mould was used with sufficient spacers to give a thickness equal to that under the blocks in the pavement. A Kango vibrating hammer with a 150 mm diameter foot was used for compaction.

(b) Triaxial shear strength. Standard laboratory apparatus was used on 38 mm diameter by 76 mm high cylindrical samples with cell pressures of 50, 100 and 150 kPa.

Examination of the concrete blocks revealed that there was no distress evident whatsoever although they had been subject to substantial transient and permanent movement. It was found extremely difficult to raise the blocks at the end of trafficking because, of course, disturbance of the bedding sand was to

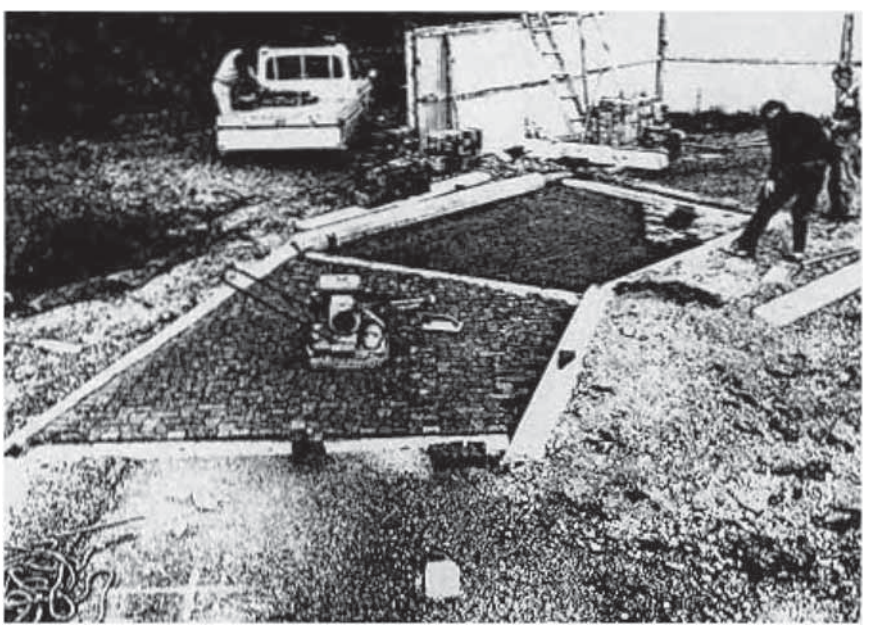


*Figure 4: Constructing the block pavements*

be avoided. The fine joint filling sand appeared to have become almost cemented to the blocks, and cleaning of the blocks for re-laying took considerable time and trouble.

Finally, an attempt was made to model the pavement structure using the University of California computer program ELSYM described by Ahlborn[6]. With reasonable estimates available of modulus and Poisson's ratio for the sub-grade, the rubber and the sub-base, these parameters were varied for the blocks and bedding sand until a reasonable match between the predicted and observed deflection bowl shapes was obtained.

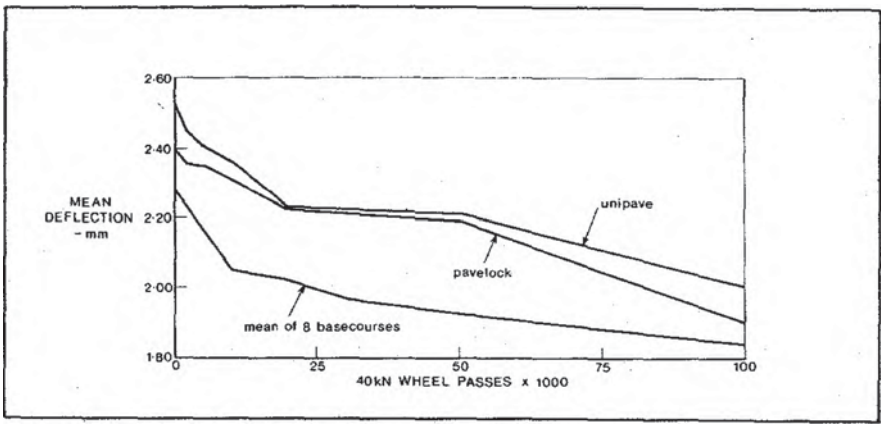



*Figure 5: Benkelman beam deflections for concrete block and unbound granular basecourse pavements*

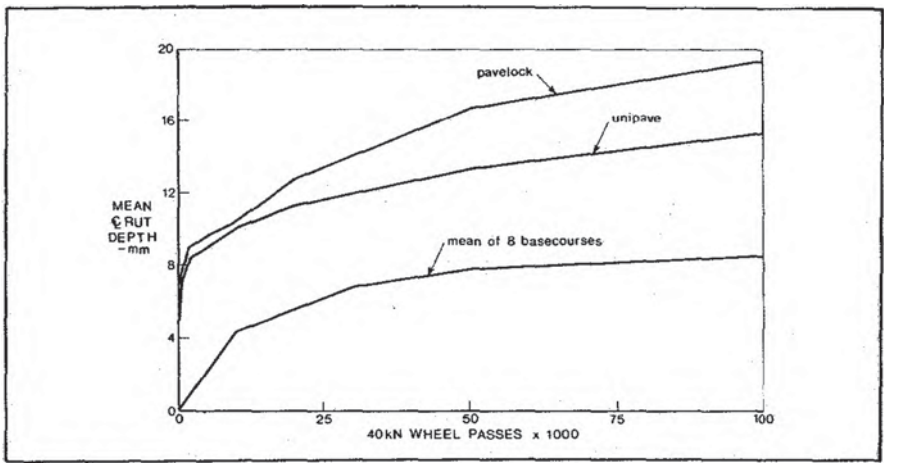



*Figure 6: Mean centreline rut depths for concrete block and unbound granular basecourse pavements*

**Results**

The deflections, i.e. resilient deformations, for the two block types together with the mean of the eight satisfactory basecourses are shown in Figure 5. It is clear that the block pavements gave significantly higher values than the conventional flexible ones, particularly at the early stages of the tests. These results do not follow the pattern of Shackel[7] who observed that

> '. . . in respect of resilient behaviour the block pavements appeared to attain a stable (shake-down) condition virtually from the onset of loading. By contrast, the flexible dolerite pavements required a substantial period of trafficking . . .'

In comparing these results, however, there are two aspects to be considered:

(a) the influence of the artificial rubber subgrade is not fully understood, and

(b) the sub-base had received 150,000 wheel passes (under a granular basecourse) before the concrete blocks were laid. Other observations suggest that it was quite stable at that time.

At the end of trafficking, i.e. 100,000 wheel passes, the block pavements were giving a deflection similar to the granular basecourses and, as the blocks themselves had no indication of distress, it appeared that they still had a good long life in front of them.

The mean rut depths, measured on the centreline, of the two block types, together with the mean of the eight satisfactory granular basecourses, are shown in Figure 6. Clearly, the block pavements gave substantially higher permanent deformation than the granular bases. Other observations on the sub-base, which had previously received 150,000 wheel passes, confirmed that this deformation must have taken place in the sand layer. There is agreement with Shackel[7] that, under trafficking, the block pavements tend to lock up and become progressively stiffer, but in this case the final deformation was much higher and the locking up did not really start to take effect until 50,000 wheel passes.

Transverse profiles of average rut depth for the block pavements and the granular basecourses at 100,000 wheel passes are shown in Figure 7. These confirm the higher deformations for the block pavements but also exhibit higher heaving at the edges of the trafficked path. In examining this phenomenon, it should be realised that, although the trafficking machine simulates the lateral distribution of wheel passes on a public road very well, there is a point beyond which the wheels do not pass. So, in this case, if there is any lateral movement of the sand taking place it can be forced out from under the trafficked path but never

TABLE 1: In-situ densities of the bedding sand

| *Test* | *Block type* | *Position (i)* | *Position (ii)* | *Position (Hi)* |
|---|---|---|---|---|
| Dry density (t/m$^3$) | Winstone's Pavelock | 1.868 | 1.852 | 1.713 |
| | Firth's Unipave | 1.835 | 1.880 | 1.677 |
| Moisture content (%) | Winstone's Pavelock | 3.7 | 4.0 | 3.4 |
| | Firth's Unipave | 4.0 | 3.8 | 3.6 |

where Position (i) = on the centreline of trafficked path
Position (ii) = in the outer area of trafficked path
Position (iii) = outside the trafficked path

be forced back. In the real situation, there would always be the occasional vehicle which would travel outside these limits forcing some of the sand back, and so the heaving would not be so severe.

Deflection bowls for the two types of concrete blocks and for the mean of three of the basecourses are shown in Figure 8. In this case, the data for the blocks were taken at 100,000 wheel passes whereas those for the basecourses were at 250,000 passes. Bearing this in mind, it appears that the stiffness of the two types of pavement layer are very similar. This conclusion is in line with Shackel[7] whose study showed that block pavements responded to traffic loadings in a manner qualitatively similar to conventional flexible pavements.

After removal of the concrete blocks, balloon densometer tests on the bedding sand gave the results contained in Table 1. It will be seen that the dry densities under both types of block at both positions in the trafficked path were very similar, with an average of 1.86 t/m$^3$. The lateral distribution of wheel passes actually gives about twice as many passes over Position (i) as Position (ii). The average dry density at Position (iii) outside the trafficked path was 1.69 t/m$^3$. It is clear, therefore, that considerable further densification of the bedding sand took place under the action of traffic loading.

Compressive strength tests were carried out on the concrete blocks according to the NZ Portland Cement Association[1] recommended specification. The Winstone Pavelock blocks gave a mean strength of 37.4 MPa and the Firth Unipave 56.6 MPa. The specification recommends a mean strength of 50 MPa for these 80 mm thick blocks. Neither type of block showed any form of distress after 100,000 wheel passes under severe transient and permanent deformation so, in view of the strength of the Pavelock blocks, it appears that the specification may be demanding too high a value.

In view of the large permanent deformation of the blocks due to densification of the bedding sand, a series of laboratory tests was carried out on this material. The first of these was a compaction test using a 150 mm diameter mould with a vibrating hammer fitted with a circular foot according to the NZ Standard[8]. The results showed that the highest dry density of 1.89 t/m$^3$ was achieved at zero water content, falling to a 'pessimum' of 1.731 / m$^3$ at 4 to 5% and rising to a peak of 1.82 t/m$^3$ at 13% water content. The as-delivered water content of the sand on which the blocks were laid was 3%, i.e. almost the worst possible for

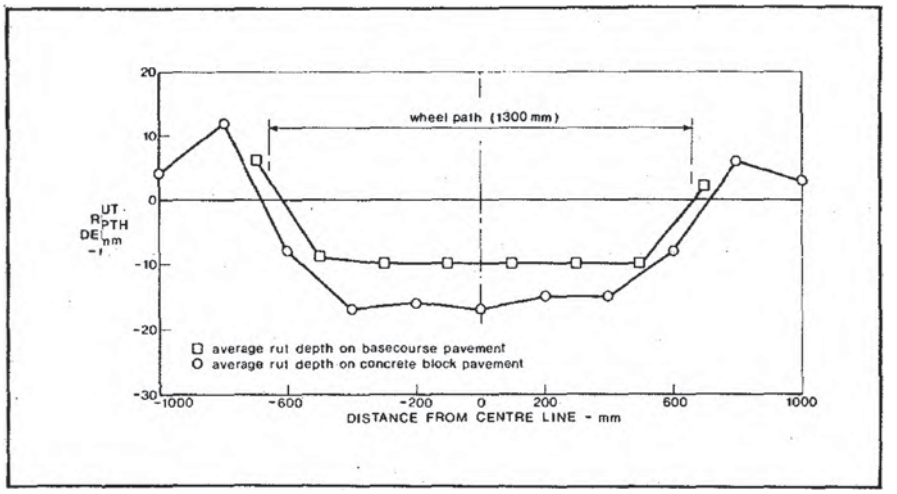



*Figure 7: Transverse profiles of rut depth at 100 000 wheel passes*

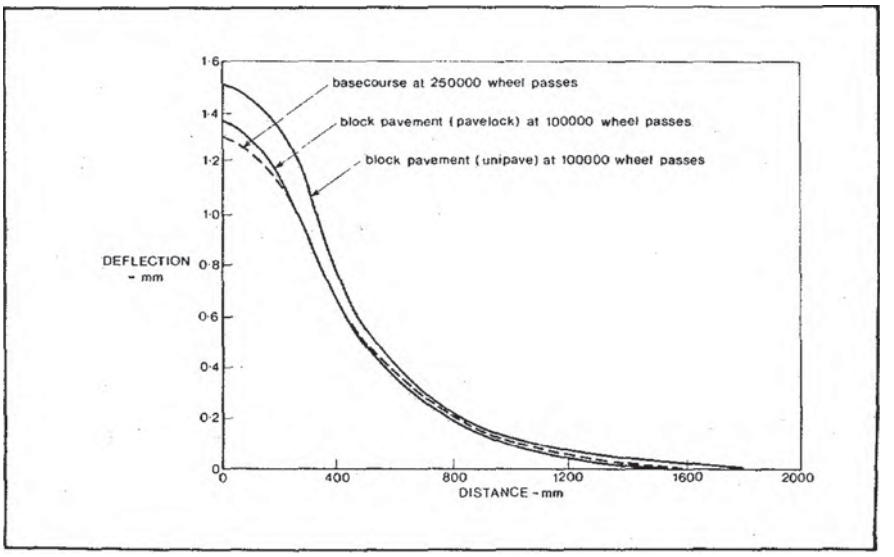



*Figure 8: Deflection bowl shapes*

achieving adequate compaction under the plate vibrator. For this sort of use, the best condition for the sand would be dry, but this would be impossible to achieve in practice. The alternative would be to saturate the sand at some stage before applying the compaction plant.

In order to simulate the sand bedding layer, the standard 150 mm diameter mould was then filled with spacers to give an uncompacted sand thickness of 57.5 mm. The same vibrating hammer and foot was then applied for 180 s and the settlement and dry density of the sand obtained. The results are shown in Table 2. The figure of 1.609 t/m$^3$ for the sand at 3% water content is slightly less than the in-situ densities found under the concrete blocks at Position (iii) which had received no traffic loading, and shows that this laboratory compaction simulated the field compaction quite well.

TABLE 2: Laboratory dry density and settlement of bedding sand

| *Water content (%)* | *A verage settlement (mm)* | *Final dry density (t/m$^3$)* |
|---|---|---|
| *0.2* | 9.4 | 1.872 |
| *3.0* | 16.8 | 1.609 |
| *20.0* | 15.6 | 1.721 |

Laboratory compaction at 0.2% water content gives a figure of 1.872 t/m$^3$ which compares well with the average of 1.86 t/m$^3$ in-situ density under the trafficked path after 100,000 wheel passes. If the average settlement of 16.8 mm for 3% water content is compared to the rut depths at 100,000 wheel passes in Figure 6, it must be realised that the latter rutting had taken place after field compaction and during trafficking. It is thus apparent that, in addition to compaction, some migration of the sand must have, taken place. Finally, if the sand could have been compacted in the field in a dry state to around 1.86 t/m$^3$ much of the permanent rutting could have been avoided.

In view of the apparent lateral movement of the sand beneath the blocks, drained triaxial tests were carried out at zero, 3% and 20% water contents and at cell pressures up to 150 kPa. The zero and 20% water content samples exhibited angles of shearing resistance of 43° compared to 37° for the 3% (in-situ) samples. At 150 kPa cell pressure, the deviator stress at failure for the zero and 20% water content was about 650 kPa compared to 420 kPa for the 3% samples. The stress conditions of a thin bed of sand lying beneath a layer of interlocking concrete blocks and subjected to loading from twin rolling pneumatic tyres is extremely difficult to analyse. However, it is clear that the difference between 37° and 43° in angle of shearing resistance would make a significant difference to the bearing capacity of the sand in a conventional footing, and it is quite likely to make the difference between stability and instability in the circumstances under discussion.

Finally, the modelling of the concrete block pavement using the ELSYM5 computer program was attempted. Previously, this had been carried out for the granular basecourse pavement, and so, with other experience and experimental work, reasonable values of modulus and Poisson's ratio were already available for the sub-grade, the rubber and the sub-base. The program assumes an ideal elastic system, which is manifestly not true for granular layers where the modulus is stress dependent. This makes the modelling of a deflection bowl over the whole of its length not so straight forward. It was decided therefore, like Scala[9] and Gray and Baran[10], to concentrate attention on bowl shape immediately adjacent to the wheel loads. In fact, the match of predicted bowl shape was reasonable up to 600 mm away. By varying the modulus and Poisson's ratio for the concrete block layer until the best fit was obtained, it was concluded that the blocks were behaving as an elastic layer with:

Modulus = 415 MPa and
Poisson's Ratio = 0.25.

This value of modulus is much less than the 900 MPa adopted by Knapton and Barber" in analysing the deflections under repetitive (non-rolling) loads applied by hydraulic jack. Although it was clearly apparent that the choice of sand, its water content, compaction and thickness were incorrect at construction, at the time these deflection bowls were measured the block pavements were in a locked-up condition.

## Conclusions and recommendations

From this experiment, it appears that the concrete block pavement behaved in a manner similar to a traditional flexible unbound granular pavement with a thick surfacing. There is agreement here with Shackel[12] except that the surface deflections were considerably higher in the early stages, becoming comparable with a granular base only at 100,000 wheel passes.

When deflection bowl shapes were compared, it appeared that the concrete block pavement had no greater stiffness than a granular basecourse, even in the fully locked-up condition. This is in contradiction to the results of Knapton and Barber" who used non-rolling loads and concluded that

> '. . . the surfacing can spread very high load concentrations over a large area of the subgrade'.

In this experiment, the concrete blocks were behaving as an elastic layer with a modulus of 415 MPa — cf. 475 MPa for granular basecourse.

There was no cracking or spalling of the concrete blocks at 100,000 wheel passes in spite of the large deflections and permanent rutting. As one type of block had a crushing strength of 37.4 MPa, it appears that the NZ Portland Cement Association[1] specification of 50 MPa for 80 mm thick blocks may be too high.

It is quite clear that the excessive permanent rutting, and possibly the transient deflections at early life, were due to the sand layer used in this experiment. It is, therefore, concluded from the experiment that careful attention be paid to this layer.

In the first place, it appears that the permanent rutting could have been reduced by building up the sub-base instead of using 60 mm of sand in order to match the levels of the existing basecourse layers. This point has been covered by Shackel[7] who showed that permanent rutting was substantially improved by reducing the sand thickness from 50 mm to 30 mm. On the other hand, the Dutch Studie Centrum Wegenbouw[13] states that the sand layer must be elastic and recommends a thickness of 50 + 20 mm. It further states that the layer is proved to take out unevenness in the sub-base and that, after areas which have settled have been re-laid, the sand layer should not be made more than 200 mm thick. Clearly, by keeping the surface tolerance of the sub-base to, say, ± 5 mm there is no reason why the sand thickness could not be reduced to around 20 mm. However, as the thickness of the sand layer does not appear to create problems in Holland, other reasons for the severe rutting must be sought.

It is obvious that the rounded beach sand with the grading shown in Figure 3 was unsuitable for the 80 kN axle loads applied to this pavement. The Studie Centrum Wegenbouw[13] states that for housing, shopping areas and car parks any suitable sand may be used, but that for very heavy traffic, 'broken' sand from crushing plants, well graded between 0 and 8 mm, should be used. A mixture of 3 to 8 mm broken stone with crusher dust was used in the Europe Container Terminus at Rotterdam. This type of material, if adequately compacted, would clearly have a higher shear strength to resist the lateral displacement which must have taken place in this experiment.

The question of compaction must also be considered. Data from Tables 1 and 2 reveal that considerable further densification took place under trafficking. It is apparent that, with the sand used, higher densities before traffic could have been obtained if the moisture content had been either zero or about 13%. Similar figures would also apply if a well graded crushed sand had been used. As zero moisture content would be impossible in most cases, it would seem desirable to saturate the sand at some stage of the operation. Without any experience of this, the author tentatively suggests that water might be sprayed on the blocks after they have been laid on the sand but before the vibrator is applied. The plate vibrator used in this experiment (Figure 4) appeared at the time to be very light for the task expected of it, but a heavier one might have caused damage to the recently-cast blocks. At the end of the experiment the blocks were removed, cleaned and re-laid to form a footpath near the buildings on the site. For compaction, a hand vibrating roller of 500 kg mass was used with no damage to the blocks but at this stage the blocks were at least six months old.

On a recent visit to Amsterdam, the author was informed that vibrating rollers are used in that city, but further experimental work would clearly seem desirable.

Finally, an elastic layer computer program does not seem appropriate for the analysis of this type of pavement, and so the author is currently adapting a finite element program for the purpose.

## Acknowledgements

The author wishes to record his thanks to Winstone Ltd and Firth Industries Ltd for supplying and laying concrete blocks, to the NZ Portland Cement Association for financial assistance, and to

N. W. McDougall and S.T. Cheah for their conscientious work on the project.

This report is based on a paper prepared for the 10th ARRB Conference at Sydney in August 1980.

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