PERFORMANCE OF A TRIAL HEAVY DUTY INTERLOCKING CONCRETE BLOCK PAVEMENT AT THE PORT OF LYTTELTON

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SUMMARY

The results are given of observations on a small test area of interlocking concrete block pavement at the Port of Lyttelton, New Zealand. Rectangular 200 mm x 100 mm x 100 mm blocks were laid in herringbone pattern with 50 mm nominal compacted depth of bedding sand over a granular subbase, and a subgrade with a relatively high CBR. A vehicle detector loop was installed below part of the trial area which, together with the cargo statistics available, has allowed detailed analysis of the traffic across the loop. At 9 years the area around the detector loop was beginning to fail and this has allowed comparisons to be made with design methods. The New Zealand design methods have been shown to produce pavements which are consistent with the performance of the test area up to failure. These include the New Zealand design chart for heavy duty pavements (IB 27), and also the EDA based design charts (IB 67 and NRB RRU TRIO), although these are not for heavy duty pavements. As well the results are compared with the BPA Manual design method. The ability of rectangular blocks to tolerate uneven surface settlement, with relatively little spalling of the top edges, was observed. Different sand gradings, some not complying with recommended grad-envelopes, appear to have performed satisfactorily.

1 Introduction

This paper gives the results from observations, carried out by the Lyttelton Harbour Board (LHB), on a small test area of heavy duty interlocking concrete block pavement (ICBP). The area is located by the Gladstone Pier Linkspan at the Port of Lyttelton, New Zealand, and was subjected to intensive use by forklifts carrying containers, etc., to and from roll-on roll-off vessels over a 9 year period.

At the trial area 200 mm x 100 mm x 100 mm rectangular blocks (200x100x100) were laid in a herringbone pattern with 50 mm nominal compacted depth of bedding sand over a granular subbase and a subgrade with a relatively high CBR (probably with a minimum of 15). Three different gradings of bedding sand were used in separate parts of the area.

A vehicle detector loop was installed below part of the trial area which, together with the cargo statistics available, has allowed detailed analysis of the traffic over the area.

The performance of the test area over the first few years was such that it gave confidence in specifying interlocking concrete blocks for other areas of heavy duty pavement at the Port of Lyttelton **(1)**.

By the end of the 9 year period the most heavily trafficked part of the test area (that around the detector loop) was beginning to fail, and this has allowed the traffic loading actually experienced to be used to calculate the subbase thickness required by the various design methods, and to compare these with the subbase thickness used at the trial area.

2 Background

In 1978 it was decided that a small area of asphaltic concrete pavement behind the Gladstone Pier Linkspan should be relevelled to reinstate grades following settlement beside the retaining wall. As a route was also required across the area for a new 200 mm diameter water main, it was agreed that a trench should be < excavated alongside the retaining wall and a new section of pavement laid.

About this time local concrete block manufacturers offered to manufacture a small batch of 100 mm thick rectangular blocks and absorb the mould costs if LHB was prepared to lay a trial area of ICBP. Therefore it was decided to repave with ICBP, after trenching, and that the area was to act as a trial pavement. The area was considered to be the most heavily trafficked area of heavy duty pavement in the Port, as it was on the direct route of all cargo traffic to the linkspan. This would allow conclusions to be drawn from the trial more rapidly than if the test pavement was located elsewhere.

The Linkspan, and adjacent Union Co. Seacargo Terminal, were leased from LHB by Union Maritime Services Ltd, and serviced the Union Shipping Group roll-on roll-off vessels, primarily those operating on the trans-Tasman trade, but with some coastal cargo, and the Shipping Corporation of NZ's "Coastal Trader". For the initial period of the trial there were 10 vessel calls per fortnight, with most taking a 2 shift day to discharge and load.

Climatic conditions at the test area are "temperate" with approximately 600 mm of annual rainfall. Frosts occur but the ground thaws each day.

3 Materials and Construction

After a review of the literature available, 100 mm thick rectangular blocks were chosen. Winstone (SI) Ltd manufactured the blocks at their Hornby plant in December 1978. The blocks were steam cured for 24 hours and then stacked outside. The specified block strength was 50 MPa (compressive strength determined as set out in Appendix A of IB 019 (2)), the manufacturer advised the strength achieved was 60 MPa, and 5 blocks tested later (in accordance with IB 019 but without soaking prior to testing) gave an average of 52.8 MPa, with a maximum of 63.43 MPa and a minimum of 44.69 MPa. The blocks had 3 mm deep x 22.5° chamfers on the top edges.

LHB arranged backfilling of the trench up to subbase level, and subbase preparation, on a daywork basis, prepared a short specification for the blocklaying (following the recommendations of IB 020 (3)) and obtained prices for this work from experienced blocklayers. Essentially the subgrade and subbase were similar to those existing below the asphaltic concrete prior to the trenching, with additional thickness of subbase (uncrushed AP65, with soaked CBR > 100) to allow matching of surface grades (the nominal subbase thickness was 150 mm but it tapered and the thickness varied up to about 200 mm). No attempt was made to design subbase thickness in terms of wheel loadings and subgrade CBR (although the subgrade CBR is believed to be a minimum of 15, the 10 percentile value used for the asphaltic concrete pavement design). Blocks were laid in a herringbone pattern. The area already had rigid concrete boundaries on all sides (apart from a short length), for reasons other than paving, and did not require any further work to make the boundaries suitable for block paving.

A bedding sand thickness of 50 mm was specified on the basis of the recommendation in IB 020 (3). As no New Zealand guidelines for bedding

sand grading envelopes had been published at that stage, it was decided to include different bedding sands in the test.

Three different bedding sands were used, a naturally occurring beach sand from Birdlings Flat, a cyclone sand from an asphalt aggregate plant and a 50-50 mixture of the other two sands. The cyclone sand particles were angular as it was a crushed material (the beach sand particles were more rounded). The blocklayer preferred the beach sand as a bedding sand. Sub sequent publication of NZS 3116-.1981 (4) showed that the beach sand was within the standard's grading envelope for bedding sands (see Table 1) but that the other two sands were finer. No real difference in block pavement performance was detected which could be attributed to bedding sands.

TABLE 1 - GLADSTONE PIER TEST AREA - SAND GRADINGS

Test Sieve Aperture mm	Percentage Passing Test Sieve				
	Birdlings Flat Sand	Cyclone Sand (Miners Rd)	50-50 Beach/ Cyclone Sand	NZS 3116:1981 Bedding Limits	NRB TRIO Joint Limits
9.5				100	
4.75	100	100	100	95 - 100	
2.36	94	98	96	80 - 100	100
1.18	84	96	90	50 - 95	90 - 100
0.600	45	94	69.5	25 - 60	60 - 90
0.300	11	77	44	10 - 30	30 - 60
0.150	1	34	17.5	0 - 15	10 - 30
0.075		16	8	0 - 10	0 - 10

Trenching began in late April 1979, the first section of blocks was laid in Hay, the second in June, the third in July/August, and the fourth in September/October 1979 to give an area of approximately 770 m^2. Cargo handling operations continued around the construction areas throughout the excavation, aggregate placement and blocklaying, and was the main reason for the lengthy construction period. Settled areas were lifted and relaid in October 1979 and July/August 1981. In February/March 1981 approximately 140 m^2 of additional block pavement was laid (following resiting of an old signal tower) to give a total area of 910 m^2. The blocks were hand laid by a contractor with a 2 man gang. Laying rates were slow but the workmanship was very good.

During blocklaying the blocklayer found some blocks were out of tolerance and these blocks were replaced by the manufacturer. (Approximately 2.1% of the blocks delivered to site required replacement.)

The sand used for the joints was the cyclone sand used as bedding sand for part of the area. Subsequent publication of NZS 3116:1981 showed that the cyclone sand was not within the standard's requirements for joint sands as it had more than 10% passing the 0.075 mm sieve (see Table 1). Also the joint sand was finer than the grading limits given in NRB RRU TRIO (5) al- though it was close to the finer limit (see Table 1). However the joints appear to have performed satisfactorily (it was observed that the joints quickly became full and appeared to be near watertight).

4 Performance

Some settlement did occur behind the retaining wall (above subsoil drain outlets through the wall) and around a sump in the centre of the area. In both cases the problem was not due to the block pavement but it did demonstrate the ability of the blocks to tolerate uneven surface settlement. Some damage did occur to the top edges of the blocks and it is considered that the damage with the rectangular blocks was less than that which would have occurred with other shaped blocks.

At the retaining wall blocks were continued across the top of the wall. This was a mistake because, when settlement occurred behind the wall, the blocks over the back edge of the wall could not rotate to follow the settlement and broke. A similar problem occurred on the other side of the area where the blocks were laid over a lip on a concrete slab containing a railway track. This clearly demonstrated that it is necessary for the blocks to be butted up against the vertical face of a rigid abutment, not continued over a lip.

When the area which had contained the old signal tower was paved an area of unsatisfactory subgrade was left in place by mistake. After only a few passes of heavy forklifts on the first day of operation, on 6 March 1981, the pavement failed by deep rutting (up to 150 mm deep) and heaving. Subsequent testing of the subgrade material showed the CBR to be between 1 and 3. 110 m^2 of ICBP was lifted, the poor subgrade material excavated, and the pavement reconstructed in July/August 1981. However this failure did allow the US Corp of Engineers (USAGE) Equation parameters, used as the basis for the design charts proposed by Tait (6), to be compared with the observed values at failure (see 6.1 below).

Lome blocks were relaid to restore levels after settlement. Lifting the blocks definitely showed that there was considerable interlock between the blocks at the joints. It was necessary to physically smash a row of blocks to allow unzipping of the blocks in the area to be lifted (which was in the centre of the block paved area). When relaying it was necessary to grind the sides of the last row of closures to enable the blocks to be refitted.

Detailed observations of the blocks were carried out periodically and are summarized in Table 2. (In Table 2 the damaged blocks above the retaining wall have not been included as they are considered to have been due to a faulty detail and are not representative of overall pavement performance.)

TABLE 2 - OBSERVATION OF PERFORMANCE OF INTERLOCKING CONCRETE BLOCKS AT THE GLADSTONE PIER TEST AREA

Date of Walk Over	Approximate Number of Blocks Cracked	Damaged on Top Edges	With Surface "Plucked" (a)
June 1982	20 (0.04%)	30 (0.07%)	1 (0.002%)
September 1984	100 (0.22%)	400 (0.88%)	40 (0.09%)
December 1990	275 (0.60%)	700 (1.54%)	100 (0.22%)

Notes: (a) Had stone(s) plucked, or pulled, out of top surface.
(b) Percentages based on a total area of 910 m^2 (less than half of this area was heavily trafficked).

By 30 September 1988 the most heavily trafficked part of the block paved area had essentially failed by rutting (rut depths up to 50 mm) in that substantial repairs were required to restore an adequate surface. However over 50% of the block paved area (i.e. the less heavily trafficked part) was still in very good condition and had substantial further life left. Abrasion, or wear, on the top surface has been less than the chamfer depth.

5 Traffic Counts

A detector loop (similar to those used at traffic signals) was installed, below part of the ICBP, on the direct route to the Gladstone Pier Linkspan. The position of the loop was such that it is considered to have been passed over by all forklifts travelling to, or from, the linkspan. The crossings of the loop were recorded from installation, in August 1979, until September 1988, by which time the roll-on roll-off services using the linkspan had ceased. The traffic, together with the tonnage carried by the forklifts, is shown in Table 3.

TABLE 3 - FORKLIFT PASSES AND MANIFEST TONNES ACROSS THE WHARF PER YEAR FOR THE UNION CO. SEACARGO TERMINAL (TO 30 SEPTEMBER 1988)

Year Ending 30 Sept	Forklift Passes		Manifest tonnes	
	Annual	Cumulative from 1 Oct 79	Annual	Cumulative from 1 Oct 79
1980	25 055	25 055	500 030	500 030
1981	66 205	91 260	507 785	1 007 815
1982	40 062	131 322	534 482	1 542 297
1983	44 354	175 676	560 321	2 102 618
1984	49 920	225 596	421 464	2 524 082
1985	45 865	271 461	455 670	2 979 752
1986	28 065	299 526	284 432	3 264 184
1987	6 750	306 276	113 112	3 377 296
1988	4 179	310 455	0	3 377 296

6 Traffic Counts and Design Methods

The cumulative detector loop counts of forklift passes can be used to compare the actual performance with design methods. However, as the exact relationship between the loop counts and forklift wheel loads is not known, assumptions need to be made on the percentage of counts that are wheel passes, and on the wheel load distribution.

The detector loop, at 2 m x 2 m (with 2.5 m in a line perpendicular to the main traffic path) was considerably smaller than the width of a forklift plus container. Therefore on occasions the loop would have detected a vehicle above when the front and rear wheels had both passed to one side of the loop. As the forklift plus load could be up to 6 m wide it is considered that only 65% to 85% of the total vehicle passes over the loop consisted of a front (and rear) wheel on one side passing over the loop, with 75% giving the best estimate (the front and rear wheels were in line).

The wheel load depends on 3 factors:- the percentage of passes of unladen forklifts, the distribution of the load carried by the forklift, and the forklift wheel loads for the various forklifts and carried loads.

It can be shown that for a container weight distribution such as that given in the BPA Manual (7) for a 20 foot ISO container, the average wheel load for a particular vehicle gives a design load similar to the average design load calculated from the full wheel load distribution for the usual pavement design parameters (i.e. EDA, Shell Standard 80 kN Design Axle, and PAWL - all within approximately 6%). Therefore the average carried load has been used in this analysis.

The average load carried by a forklift is calculated from the total tonnage and total passes in Table 3, (i.e. from 3,377,296/(310,455 x % of Forklifts Laden]), taking 1 manifest tonne = 1000 kg, but depends on assumptions of the percentage of unladen forklift passes. Table 4 indicates that the percentage unladen would have been between 0 and 25%. It is considered that the best estimate of the percentage of unladen forklift passes is 15% with the remaining 85% being of forklifts carrying an average load of 12.80 tonne. This average load allows for the fact that the loads carried by the forklifts were "Seafreighters" (a container flat approximately 4.5 m long by 2.4 m wide, with a maximum gross weight of approximately 15 tonne), timber packs and 20 foot ISO Containers (14.354 tonne average weight for UK ports (7)), and takes into account the increasing percentage of ISO containers, and the increasing weight of the timber packs, in the later years.

TABLE 4 - GLADSTONE PIER TEST AREA - AVERAGE LOAD CARRIED PER FORKLIFT PASS FOR VARYING PERCENTAGES OF UNLADEN FORKLIFTS

% of Forklifts Unladen	50	25	20	15	10	5	0
% of Forklifts Laden	50	75	80	85	90	95	100
Average Load per Pass (tonne)	21.46	14.50	13.60	12.80	12.10	11.45	10.88

The forklifts in use at the Terminal varied with time. From 1980 on the Hyster H360A Forklifts were being replaced by the larger capacity Lees machines, such as the D40, D45 and D50, and, on occasions, by the Lees F55 and D65 (hired from LHB). This gives difficulty in calculating the average front and rear wheel loads. For comparison with design methods the front and rear wheel loads have been calculated for the machines for various loads and an assessment made of the average, and upper and lower bounds.

The cases considered are shown in Table 5 where the lower bound is for the smallest forklift (Hyster H360A) and the upper bound is for the largest (Less D65), (carrying the average loads for the % unladen, and with the % of loop counts being wheel passes, shown in the Table). The 3 mid range cases are for the assessed weighted average forklift wheel load when carrying the average load of 12.80 tonnes.

By 30 September 1988 it is considered that the pavement in the vicinity of the detector loop had essentially reached the end of its life and that major repairs were required. Thus the cumulative design loads calculated from the detector loop are those for the pavement design life. Note that the end of the design life is the stage where repairs to the pavement are required - this is not necessarily full reconstruction of the pavement.

The basis of the comparison of design methods that follows is to calculate the subbase thickness required by the design methods for the respective cumulative design loads obtained from the detector loop. For the comparisons the pavement cross-section at the detector loop is 100 mm thick blocks on 50 mm of bedding sand and 200 mm of granular subbase, with the subgrade CBR » 15.

6.1 Repetitions and USACE Equation Based Design Method

The heavy duty ICBP design chart in IB 27 (8), and first proposed by Tait (6), is based on the bedding sand thickness plus twice the block thickness being subtracted from the USACE Equation thickness above subgrade to give the thickness of subbase required. In the USACE Equation (which was derived for flexible airport pavements consisting of structural asphalt on granular bases)

$$t = (8.71 \times \log R + 5.43) \{W \times 1/(8.1\ CBR) - 1/(450\ p)\}^{0.5}/F \quad ..... \quad (1)$$

where t = total pavement thickness (mm),
R = number of wheel repetitions,
W = wheel load (kg),
CBR = subgrade CBR,
p = tyre pressure (MPa), and
F = factor to allow for CBR > 10 (F = 1 for CBR up to 10, F = 1.005 for CBR - 15).

For the test area W is the Equivalent Single Wheel Load (ESWL) at 350 mm depth (with the ESWL calculated according to Figure 5 of IB 27) and p -- 0.83 MPa. Table 5 shows two subbase thicknesses for the cumulative counts from Table 3, one calculated from Equation (1) when only considering the dual front wheel load (i.e. ignoring rear and unladen wheels loads), and the other using the IB 27 method (which includes rear and unladen wheels as equivalent laden front wheel passes).

TABLE 5 - GLADSTONE PIER TEST AREA - SUBBASE THICKNESS CALCULATED FROM DETECTOR LOOP COUNTS FOR VARIOUS DESIGN METHODS AND PERCENTAGES OF UNLADEN FORKLIFTS

Estimate	Lower Bound	Likely Min.	Best Est.	Likely Max.	Upper Bound
% of Unladen Forklifts	10	15	15	15	20
% of Counts that are Wheel Passes	65	65	75	85	85
No. of Laden Front Wheel Passes ($x10^{-5}$)	1.816	1.715	1.979	2.243	2.111
at ESWL W (tonne) of	12.71	13.45	13.28	13.45	16.57
Subbase thickness from Eqtn (*1*) (mm)	178	188	191	198	244
Ratio Design:Actual	0.89	0.94	0.95	0.99	1.22
IB 27 Subbase Thickness (mm)	175	190	200	210	240
Ratio Design:Actual	0.875	0.95	1.00	1.05	1.20
EDA Calculated from Detector Loop ($x10^{-7}$)	3.37	4.53	5.22	5.92	13.56
IB G7/RRU TR 10 Subbase Thickness (mm)	180	205	215	220	265
Ratio Design:Actual	0.90	1.025	1.075	1.10	1.375
BPA Manual Design Life Passes ($x10^{-5}$>)	1.82	1.73	1.99	2.27	2.15
of Critical Damaging Effect (PAWL)	13.7	23.9	23.9	23.9	61.6
BPA Manual approach Subbase Thickness (mm)	283	276	286	300	297

Also for the initial failure of the area of weak subgrade approximately 20 passes of a 15,000 kg dual wheel load (W - 11,250 kg, p = 0,8 MPa) occurred before failure with a CBR estimated to be 2. Using Equation (1) gives a subbase thickness of 202 mm with 30 mm of bedding sand for 100 mm blocks - these were in fact the actual thicknesses.

Thus use of the IB 27 design chart approach appears to give results consistent with the performance of the test area. The difficulty with this design method appears to be in deciding at what depth to calculate the ESWL.

6.2 EDA Repetitions and Design Methods

In NZ the load parameter used in road pavement design is the equivalent 8.2 tonne design axle (EDA) and the damage rating of other single axle loads (with dual wheels) are related to it by

$$EDA = \{P/8.2\}^{3.85} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

where P is the axle load in tonne (other ratios are used for single wheels, or for tandem and tri-axles (5)), for axle loads up to approximately 20 tonne (i.e. approximately 30 EDA). This equation is not necessarily a suitable method for accounting for repetitions of high axle loads, but it is of interest to apply the equation to the detector loop counts. It should be noted that for NZ roads the lane width is normally 4 m and truck widths are up to 2.4 m which allows for loaded dual wheels up to approximately 600 mm wide covering a strip approximately 2.2 m wide. On this basis the counts of the wider forklift dual wheel passes at the detector loop can be considered equivalent to axle passes on a road lane.

The cumulative forklift passes from Table 3 give the total EDA for the pavement life shown in Table 5. (For the forklifts that operated at the Union Co. Seacargo Terminal it is considered that the average total EDA per per pass of a forklift carrying 12.80 tonne was 256, and 42.6 unladen.) Detailed calculations show that the total EDA for a particular model of forklift is relatively insensitive to the percentage of unladen forklift passes (for the average forklift EDA the total EDA for the period with 25% unladen was only 1.269 times that with 10% unladen). Therefore it would appear that the assumption of 15% of unladen forklift passes is unlikely to lead to gross errors in the calculation of the total EDA for the period.

EDA based design charts for ICBP are given in IB 67 (9) and in NRB RRU TRIO (5). In both cases the charts are derived by computer analysis using multi-layer elastic theory, but only cover 80 mm thick blocks. The charts give similar results.

If the design charts in IB 67, and in NRB RRU TR 10, for 80 mm thick blocks on granular subbases are considered, and 40 mm is subtracted from the thickness of unbound material above the subgrade to allow for 100 mm thick blocks (i.e. using the hlock/basecourse equivalency ratio used in IB 27 (8, 6)), and another 20 mm is subtracted to allow for 50 mm of bedding sand at the test area compared with the 30 mm assumed in the charts, then, by extrapolation, the thicknesses of subbase required for 100 mm thick blocks with a subgrade CBR of 15 and 50 mm of bedding sand would be as shown in Table 5.

The subbase thicknesses in Table 5 show a reasonably good agreement with the 200 mm actually used, even though the EDA approach is not strictly applicable to heavy axle loads. Thus use of the EDA design charts for the design of heavy duty block pavements does appear to give results consistent with the performance of the test area.

6.3 Repetitions and PAWL Design Method

A design method for heavy duty ICBP which allows for varying wheel loads, tyre pressures and repetitions is the BPA Design Manual (7). In this method a Port Area Wheel Load (PAWL) of 12,000 kg, with a tyre pressure of 0.8 MPa, is used as the design parameter and the damage rating of other wheel loads are related to it by

$$D = (W/12{,}000)^{3.75} \; (p/0.8)^{1.25} \qquad \ldots\ldots\ldots (3)$$

where D is the pavement damaging effect in PAWLs,
W is the wheel load in kg, and
p is the tyre pressure in MPa.

Proximity factors (depending on the wheel and axle spacing and the plant's PAWL value) are used to allow for adjacent wheels. Dynamic factors are used to allow for different types of plant (e.g. forklifts, straddle carriers, etc.) and types of operation (i.e. braking, cornering, accelerating and uneven surface). Lane factors are used to allow for the reduced number of wheel passes with wide lanes.

The cumulative forklift passes from Table 3 give the total PAWL for the pavement life shown in Table 5. (For the forklifts that operated at the Union Co. Seacargo Terminal it is considered that the approximate PAWL per pass of the average forklift carrying 12.80 tonne was 23.87, and 1.14 when unladen, with a dynamic factor of 1.0, and a lane factor of 1.0.)

In the design charts for ICBP in the BPA Manual (7) the ICBP is taken as having a layer of lean mix concrete base immediately below the sand bedding and above the granular subbase. In the derivation of the design charts the horizontal tensile strain at the underside of the concrete base is one limiting design parameter (the other being the vertical compressive strain at the surface of the subgrade). The base is considered to provide most of the pavement strength and the surfacing relatively little. This is somewhat different to the situation at the test area where the subbase below the bedding sand would have no tensile strength and the block surfacing provides a substantial portion of the strength.

A method is given in the Manual for converting granular layers into equivalent depths of lean mix concrete base by means of conversion factors. The factor of 0.3 has been used to convert the lean mix concrete base thickness for zero subbase, calculated using the Manual method, to give the equivalent granular subbase thicknesses for the design life passes of the critical PAWLs in Table 5. Also the subbase thickness has been calculated using the Manual method but without any limit on horizontal tensile strain in the base layer, and 40 mm has been subtracted from the calculated thickness to allow for 100 mm thick blocks (the Manual only covers 80 mm blocks on 50 mm of bedding sand). The thicknesses obtained appear overly conservative, but this is probably partly due to the approximations used in the calculations.

6.4 Comments on Design Method Predictions

Although the largest design load per forklift was for the Lees D65, in fact very little use was made of this machine and therefore the figures in Table 5 for the upper bound values would not have been reached. Similarly the lower bound is an extreme case unlikely to have occurred. The more likely maximum and minimum values are shown in Table 5. The likely range predictions are conservative in that they give thicker subbases than that used in all cases except using equation (1) which is not now used in this way.

Some parameters are particularly sensitive and their variation can have a significant effect on the result. In particular variations in CBR have a marked effect on the subbase thickness. The effect of variation on cumulative design load can be seen from Table 5.

7 Conclusions

Rectangular blocks have suffered relatively little spalling of the to edges when subjected to uneven surface settlement due to ground movement.

Bedding and joint sands which do not comply with recommended grading envelopes appear to have performed satisfactorily.

The NZ design chart for heavy duty interlocking concrete block pavements has been found to be consistent with the results at the test area, and are not unconservative for subbase thickness, but the BPA Manual approach appears overly conservative. Also the EDA based design charts published in NZ have been shown to produce interlocking concrete block pavements which are consistent with the performance at the test area up to failure, even though these charts are not for heavy duty pavements. (Both their approaches consider the pavement as a conventional flexible pavement.)

The test area has performed satisfactorily under intensive loading from heavy duty forklifts, with failure beginning in the most heavily trafficked part after 9 years, and has shown that satisfactory heavy duty pavements can be constructed using rectangular interlocking concrete blocks over unbound granular layers provided the subgrade has a reasonable CBR.

8 References

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