PAVE 92

PAVEMENT REHABILITATION WITH CONCRETE BLOCK PAVING: A CASE HISTORY

Moshe Livneh, Karl Ruhm and Noam A. Livneh
Harry Oppenheimer Professor of Engineering, Transportation Research Institute, Technion, Haifa, Israel, Chief Highway Transportation Planning Engineer, Ministry of Housing, Jerusalem, Israel, and Research Engineer, Haifa, Israel, Respectively.

SUMMARY

In a previous paper [1], the authors recommended a CBP solution for the rehabilitation of existing asphalt pavements based on problematic subgrades such as swelling clays or collapsible loess. The passing of time has enabled the authors to follow up on this type of solution at the Beit-She'an urban junction in Israel. The analysis of the field and laboratory tests recently carried out by the authors for this junction, indicates that the causes for failure in the asphaltic pavement, as well as in the rehabilitation solution, stem mainly from the pavement structural sub-design. Also, the CBP pavement's Modulus of Elasticity was found to be lower than that of the asphalt pavement. The reason for this may be the way in which the blocks were laid, which may have been improper, thus preventing the formation of a plausible interlock between the blocks. Despite this finding, the rehabilitation solution of interlocking CBP at the junction is still better than the asphaltic one, due to the basic characteristics of the concrete block paving by which the interlocking blocks create a multi-jointed layer, thus not necessarily making the structural failure a functional one too. In other words, the CBP is the best solution to the problems of variable uneven plastic deformations induced in the pavement. Obviously, in the given case study, had the interlocking block layer been placed well, it would have "built" a higher Modulus of Elasticity and the failure would have thus been avoided. In conclusion, the given case study generates the following recommendations: (a) The CBP rehabilitation solution should continue to be examined in problematic failure areas; (b) The interlocking block solution in these areas will have a higher potential of success if conditions are created for "building" a higher modulus of elasticity in the interlocking block layer.

Introduction

In a previous paper presented at the Third International Conference on Concrete Block Paving in Rome[1], the authors recommended a CBP solution for the rehabilitation of existing asphalt pavements based on problematic subgrades such as swelling clays or collapsible loess. The passing of time has enabled the authors to follow up on this type of solution at the Beit-She'an urban junction in Israel. The follow-up examination is described in this paper.

As reported in the previous paper, a rehabilitation job was carried out at this junction. This rehabilitation job included the removal of the top pavement layers, of a thickness of about 20cm, and the laying of Uni type concrete paving blocks, 8 cm. thick. The blocks were laid in a herringbone pattern over a 3cm. thick layer of sand. The sand layer was placed over a new, relatively thin, subbase layer, 9 cm. thick. This rehabilitation job was completed in June 1985. The rehabilitation solution was adopted after

other rehabilitation solutions (mainly, asphaltic overlays) which were applied at the junction, failed within relatively short service periods. It is important to note that the asphaltic overlay occasionally attained thicknesses of 12-16cm., yet despite such great thickness, the carrying capacity of the pavement was not improved. This was visually evident in the failure spots along the asphaltic pavements leading to the said junction.
As compared with these solutions of asphaltic overlays, the actual in-situ behavior of the Concrete Block Paving (CBP) solution displayed a more positive potential during the first three years of use. Visual inspection during these years indicated that there were no functional failure points discernible in the paved area, and the only marks evident were minute rutting lines in the heavy traffic lane. It is important to emphasize in this context that because of the CBP's basic quality of constituting a multi-jointed layer, these rutting lines do not necessarily indicate functional failure lines. However, it is important to note in this context that a visual inspection conducted in February of 1989 indicated more severe signs of rutting in the CBP area. (See Fig. 1 and for comparison with asphalt paving, see Fig. 2). This behavior of the rehabilitation solution was the reason for the investigation described in the present report, the objectives of which are detailed below.
Despite the fact that the junction lies at the very heart of the city of Beit She'an, it also services the regional traffic passing through the city (to Tiberias, the Jordan Valley, Affula and the Jordan Basin). Thus, this junction is exposed to a great deal of traffic and, more importantly, to heavy traffic. These traffic characteristics are presented in Table 1 and Table 2.
It should be noted that the data for the year 1989 was obtained through a sample count which was carried out at the site on the 11/1/89.

Table No. 1: Traffic Intensities in the Junction

No. of vehicles in 24hr. day (in one direction only)	The Year					
	1982	1984	1985	1962	1987	1989
	2500	3880	4800	5100	3700	5216

Table No. 2: Traffic Composition

Percentage of vehicles					
Private	Taxies	Vans	Busses	Trucks	Total
38	3	28	9	22	100

The above tables indicate the following facts: (a) There is a considerable percentage of heavy vehicles (trucks and busses), about 31%, as compared with a lower percentage in urban areas. The above percentage is more appropriate to inter-urban traffic (see Fig. 3); (b) The decrease in the intensity of the traffic which took place in 1987 may have been occasioned by the opening of the by-pass road which lies to the north of the junction and leads directly from Tiberias to Afula. However, this decrease seems to be temporary as indicated by the traffic counts carried out at the junction on the 11/1/1989.

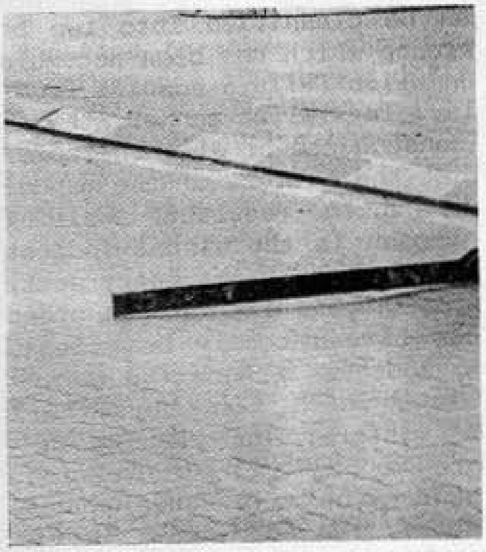

Fig. 1: A picture of rutting in the concrete block pavement.

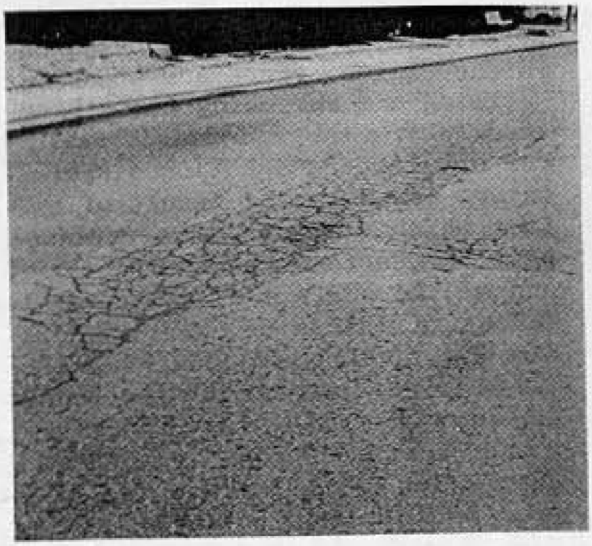

Fig. 2: A picture of alligator cracks and rutting in the asphaltic pavement.

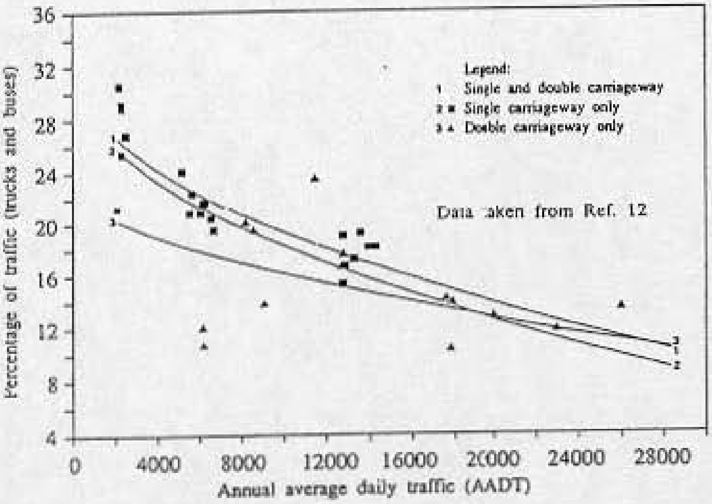

Fig. 3: The relationship between percentage of heavy traffic (trucks and buses) and inter-urban AADT.

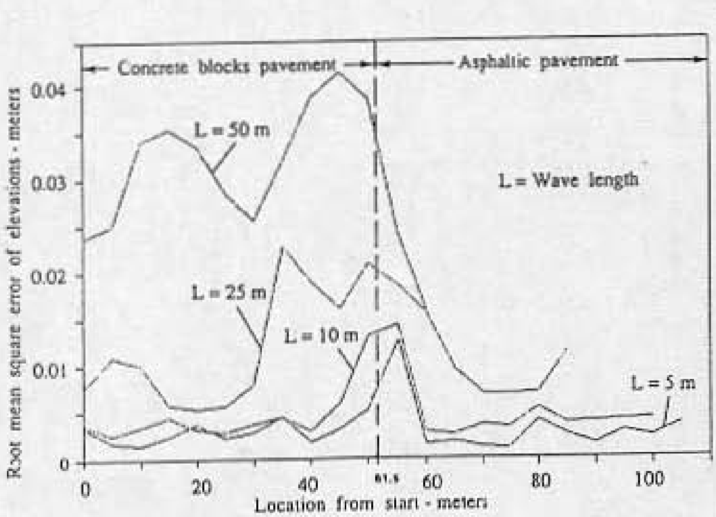

Fig. 4: RMSE of elavations as a function of the wave-length across both the asphaltic and concrete blocks pavements.

In addition to these destructive tests, the testing program also included the performance of rebound deflection tests by means of the Benkleman beam. The results of all these tests are presented in two reports issued by the Soil and Road Testing Laboratory of the Technion Institution for Research and Development [3;4]and the analysis of these test results is presented in the following sections of the present paper.

Identification and Classification Tests

The classification results obtained for pit A excavated in the CBP area (for the location of the pits and the bore-holes see map in Fig. 5), and the classification results obtained for pit B excavated in the asphalt pavement area, indicate that the total thickness of the pavement in both cases is in the range of 50-55 cm, and comprises a base-course layer 25-30 cm. thick, made up of various sizes of basaltic rock in the range of 1/2 inch to 8 inches with a silty sand filling of about 20%. This material was classified as GC (clayey gravels) and indicates its <u>great inferiority</u> in its performance as the base-course of the pavement. The thin graded granular aggregate layer (about 9-15 cm.) is also identical in nature in both pits and its classification too indicates the mediocre qualities of this material. It is important to emphasize in this context that in addition to the classification of the material, the low density of the above aggregate layer at the site considerably detracts from its engineering qualities.

The thickness of the pavement structure was also derived by means of the various drilling activities carried out at the site. According to these tests, the structural thickness (of both the asphalt and the CBP) fluctuates drastically from 30 cm. to 90 cm. (see also Fig. 5). Obviously, this fluctuation too, contributes to the formation of the various weak points in the asphalt pavement. In addition to this, the thickness of the asphalt layers should be presented as derived from the thickness of the asphalt cores. From these results it can be determined that the thickness of the asphalt in the area of the junction, ranges between 10 and 16 cm.

As compared with the equality of the base-course layer, the subgrade of the two pits described above is not identical. In pit A (CBP), the subgrade is comprised of thin sandy silt (ML) while in pit B (asphalt) the subgrade is comprised of fat clay with some silt (CH). This change in the subgrade is even more evident through the various bore-holes. These results indicate the following picture: (a) In the west asphaltic branch of the junction, the subgrade is comprised of fat clay with some silt (CH); (b) In the east asphaltic branch of the junction, the subgrade is comprised of thin sandy clay with some gravel (CL); (c) In the south asphaltic branch of the junction, the subgrade is comprised of thin sandy clay with a little gravel (CL); (d) In junction itself, in the CBP area, the subgrade is comprised of the following range of materials, from thin clay to silty sand, as follows: (1) Thin sandy clay with some gravel (CL); (2) Thin sandy silt with some gravel (ML) and (3) Thin sandy silt as above, but at a quantity of less than 50% which makes it into sandy material (SM).

In summary, for the purpose of evaluation and design, the subgrade can be classified as follows: From the edge of the CBP toward the western branch of the asphalt, the subgrade is <u>clayey</u> (L.L.= 57-68% and PI=37-46%) and from the above edge to the eastern and southern branch of the asphalt (including the CBP area), the subgrade is silty (L.L.=37-39% and PI=14-17%). The interface between the clayey subgrade and the silty subgrade is presented in Fig. 6.

In-Situ Strength Tests

The first series of strength tests was carried out by means of the Dynamic Cone Pentrometer (DCP). Graphic illustrations of a number of sample tests are presented in Fig. 7 and Fig. 8. The test results indicate the following facts: (a) In general, there is a correspondence between the structural thickness obtained through analysis of the DCP test and the thickness derived from classification of the remolded material extracted from the bore-holes. In this context, the only deviation is the result obtained in bore-hole (6). It is interesting to note that in pit A there is also a difference of about 17 cm. between the thickness obtained from the excavated pit and the thickness obtained from the analysis of the drilling results; (b) The subgrade CBR is usually higher than 10%, except for bore-hole 6 (asphalt pavement) and pit A (CBP) in which the subgrade CBR strength is about 4%. It is interesting to note that in pit A (as compared with pit B) the picture is not unequivocal as can be seen from Table 3; (c) The strength of pavement's granular materials is usually higher than a CBR of 80%. Nevertheless, it seems that the strength of the granular materials in the western branch of the entry to the junction (asphaltic structure) is lower than the strength of parallel materials in the junction square itself (CBP structure).

Table No. 3: Subgrade CBR values in pits A and B

Pit	Pavement type	CBR value according to DCP test carried out at the bottom of the pit. (in %)	CBR value according to DCP test carried out through the structural layers at a depth parallel to the bottom of the pit, before opening the pit.	type of subgrade
A	CBP	3.6 (2.2)* 3.7 (2.2)	17.0 (11.8)* 17.0 (9.6)	Silty
B	Asphalt	9.1 (12.7) 10.0 (12.5)	9.6 (11.2) 14.9 (10.6)	Clayey

* Note: The numbers in parentheses indicate the CBR value obtained immediately under the said subgrade layer.

Other, additional strength tests were carried out in a number of bore-holes as follows: In-situ SPT tets and Shear Vane tests. Some results of these tests are presented in Fig. 7 and Fig. 8, on the background of the DCP test results. All the results of the above tests are presented after their translation into CBR values by means of the correlation equations given in [5].

The above figures indicate the measure of mutual correlation of all the tests conducted, as follows: (a) The repeatability rate of the DCP results in two adjacent tests is more than good (see Fig. 7, Fig. 8, Fig. 9 and Fig. 10); (b) There is an acceptable correspondence between the CBR results obtained from DCP tests and those obtained from the SPT or Vane Shear test (see Fig. 7 and Fig. 8).

Moreover, it should be noted that the variation in the CBR values found in pit A, partially stems from the effect of confinement on the silty subgrade. Obviously, when the penetration test is carried out through the structural layers, the structural subgrade is in a confined state and

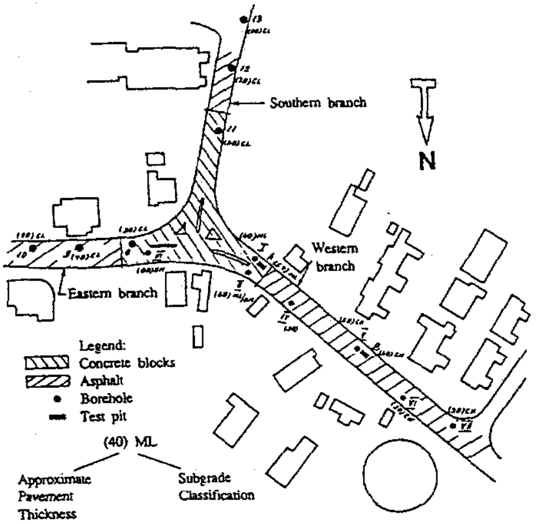

Fig. 5: Layout of boreholes and pits.

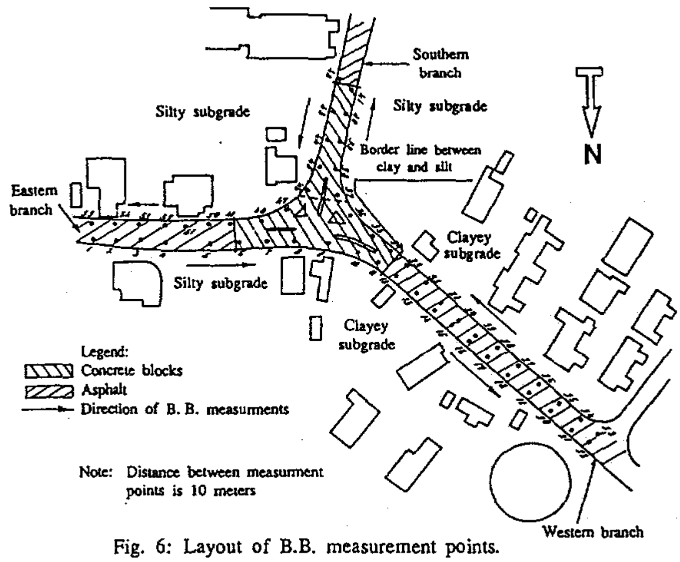

Fig. 6: Layout of B.B. measurement points.

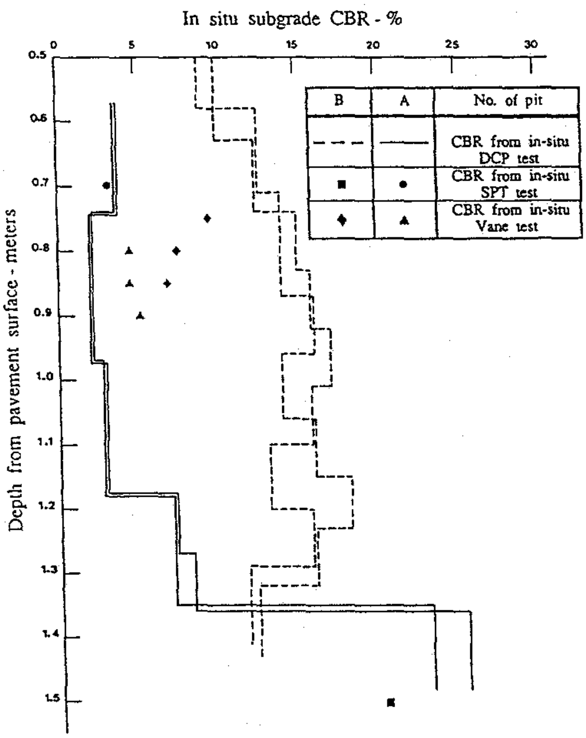

Fig. 7: Strength profile in pit A and B (subgrade only).

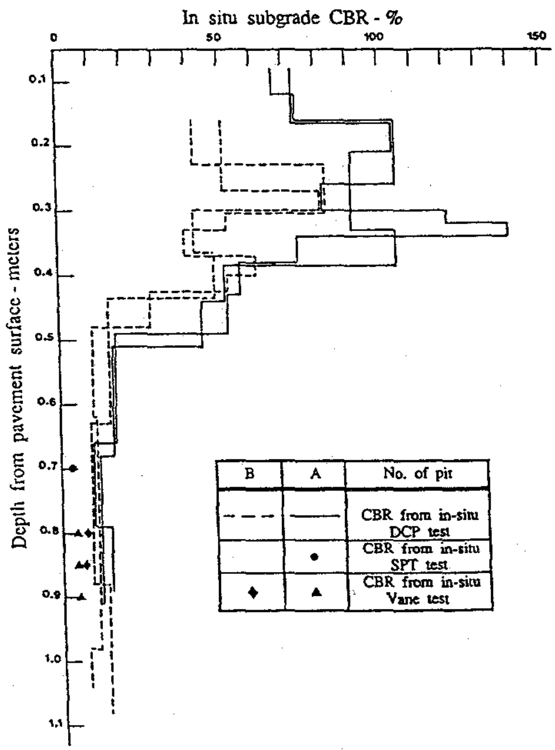

Fig. 8: Strength profile in the neighbourhood of pits A and B (including pavement layers).

therefore stronger. It is important to emphasize that the subgrade strength in a direct surface test attains the highest value at the greatest depth in which the effect of confinement is already noticeable. By comparison, the variation in CBR values found in pit B (Fig. 10) is essentially negligible as the subgrade is comprised of clayey material less effected by confinement. [11]

Analysis of the Laboratory Tests and the Other Field Tests

The CBR tests carried out on the remolded silty material (pit A) and the CBR tests carried out on the remolded clayey material, indicate that the CBR values obtained for the silty material are higher than those obtained for the clayey material. At the same time, one cannot conclude from this finding that the silty subgrade is indeed stronger than the clayey subgrade. Such a conclusion can only be drawn after comparing the CBR results obtained for samples under laboratory density and moisture conditions identical to in-situ conditions.
The in-situ density of the silty material is very low, in the range of 74% to 77% of the maximum laboratory density. The in-situ moisture is also higher than the optimum moisture and may reach a value for which the ratio W/PL equals 1.0. Thes two findings indicate the existence of low CBR values in said subgrade. Moreover, these findings may also indicate that the silty subgrade is in a state of collapse. These findings only correspond to the low CBR values obtained for pit A (A CBR value of 2.2% - 3.7% as compared with a CBR value of 9.6% - 17.0%).

For in-situ density of the clayey material is slightly higher and in the range of 82% to 84% of the maximum laboratory density. The in-situ moisture is also higher than the optimum moisture and equals a value for which the ratio W/PL is 1.1-1.2. These two findings indicate that here too, the values CBR values of said subgrade are low and do not correspond with the data obtained from pit B. The above CBR results are more appropriate to the results obtained in a similar subgrade, but in drill-hole No. 6.

In addition to the CBR tests, unconfined compression tests were carried out on two undisturbed samples. The results of these tests too indicate that the subgrade strength of 5.83 kg/sq.cm. in the vicinity of point B (clay) is higher than that of 0.41 kg/sq.cm. in the vicinity of point A (silt). This difference stems more from the difference in the in-situ density (95% for the clay and 75% for the silt), than from the difference in the classification of these two subgrades.

To conclude, it can be determined that these tests in conjunction with the laboratory CBR tests, indicate that the variation of the in-situ subgrade strength is extreme and random, ranging from very low values (CBR of about 2%) to very high values (CBR of more than 10%).

Finally, swelling tests were carried out on undisturbed clay samples extracted from drill-hole 5 (proximate to test-pit B). The density and moisture data of these samples are: (a) Density in the range of 95% and 97%, and (b) Ratio of moisture and plasticity limit in the range of 0.9 and 1.1. The above density values are particularly high. In addition to that, the testing parameters obtained were: (a) The maximal swelling value (under zero load) 6.5%; and (b) The maximal swelling pressure (under zero movement) 0.62 kg/sq.cm. These two values lead to a calculated percentage of swelling (Sp) under an equivalent counter load of 50cm. of structure (0.11 kg/sq.cm.), as follows:

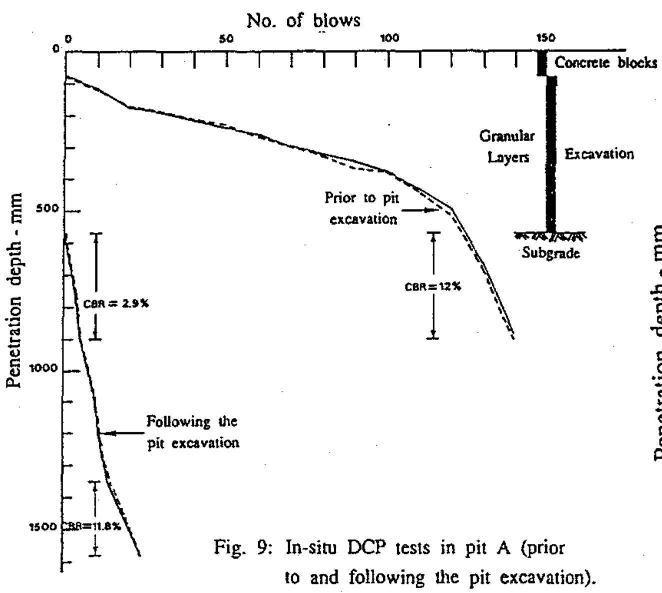

Fig. 9: In-situ DCP tests in pit A (prior to and following the pit excavation).

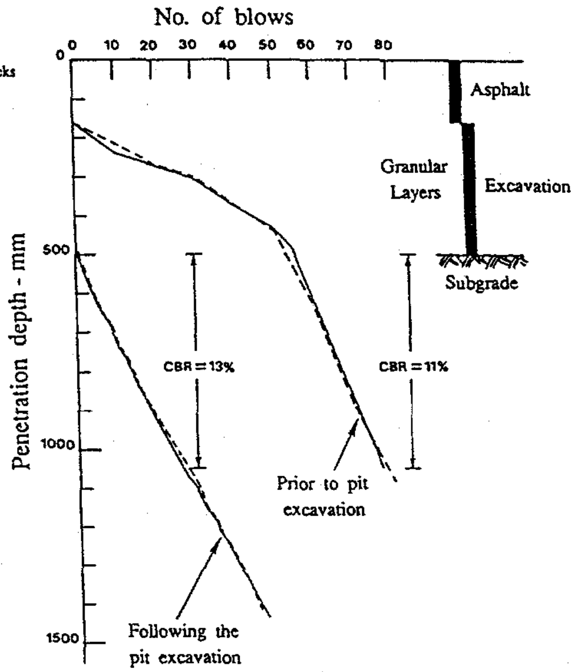

Fig. 10: In-situ DCP tests in pit B (prior to and following the pit excavation).

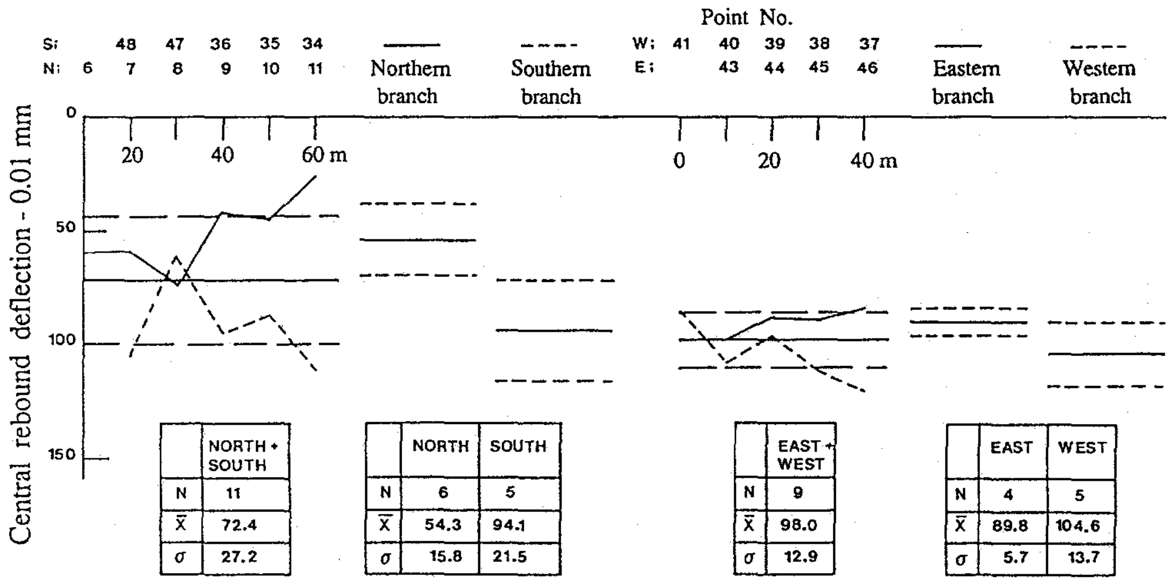

Fig. 11: B.B. central rebound deflections in the concrete blocks section.

$$Sp/6.5 = -0.54\log(0.11/0.63) = 0.41 \; ; \; Sp = 0.41 \times 6.5 = 2.7\%$$

The above swelling value indicates a low to intermediate swelling potential (which essentially also corresponds to the free swelling value obtained, i.e., 80%). The above equation was used in accordance with [6].

Summary of the Laboratory and Field Tests

The results of the various field and laboratory tests may summarized by itemizing the following evaluation parameters:

a. The structural thickness is greater than 30 cm., and 55cm. can be taken as a representative value.
b. The subgrade CBR is relatively high (about 10% and more), but the subgrade may include various weak points which are expressed both in the asphaltic structure and in the CBP structure.
c. The design subgrade CBR for silt is higher than the design subgrade CBR for clay, but in view of the in-situ moisture and density conditions, it is possible that the actual strength profile may be the obverse.
d. The subgrade of the asphalt pavement is partially a silty subgrade and partially a clayey subgrade. The subgrade of the concrete block pavement is a wholly silty subgrade.
e. The silty subgrade is liable to collapse. and the clayey subgrade is liable to swell. However, the swelling potential of the clay subgrade is of a low to intermediate level.
f. The natural subgrade moisture (both the silty and the clayey) is heterogeneously distributed.
g. The granular structure which exists both in the concrete block pavement and in the asphalt pavement is of intermediate quality.

The above parameters find expression in the work described in the following sections.

Analysis of the Benkelman Beam Test Results

In addition to the tests described in the preceding sections, deflection tests were carried out by means of the Benkelman beam. The truck load was the standard one, i.e. an axle weighing 18,000 lbs., and wheel pressure of 80 psi. The location of the measuring points is presented in Fig. 6.
The present section presents an analysis of these B.B. readings, starting with the progression of the central rebound deflection as illustrated in Fig. 11 for the concrete block pavement and in Fig. 12 for the asphalt pavement.

An analysis of the central rebound deflection by means of the ASPHALT INSTITUTE method [7] leads to the results presented in Table 4.

Table No. 4: Carrying capacity values according to the ASPHALT INSITUTE

Points	Average deflection plus two standard deviations in hundredths of mm.	DTN value taken from Eq. (1)	Pavement carrying capacity expressed as the number of applications of an 18,000lb. axle
Entire CBP area	132.9	27	2.0×10^5
Entire Asphalt area	137.3	23	1.7×10^5

It should be noted that in addition to relevant figure given in [7], the value of DTN can be calculated by means of the following expression:

$$\text{DTN} = 86.0 \times \delta^{-4.105} \quad (1)$$

where,

DTN is the design traffic index

δ is the average rebound deflection plus two standard deviations in mm.

The translation of the value of DTN into the carrying capacity of an 18,000lbs. axle, is:

$$W = 20 \times 365 \times \text{DTN} \quad (2)$$

where,

W is the number of applications of a standard 18,000lb. axle.

Table No. 4 indicates that the carrying capacity of the asphaltic pavement is essentially identical to the carrying capacity of a concrete block pavement. Both these carrying capacities are not particularly high and correspond only to light traffic according to the Israeli Public Works Department standards.

In a similar way, Fig. 13 and Fig. 14 illustrate the rebound deflection at a distance of two meters from the truck's dual wheels. Basically, this deflection expresses the strength of the subgrade. According to these figures there is no essential difference between the subgrade strength of the concrete block pavement and the subgrade strength of the asphalt pavement.

Finally, the analysis of the whole deflection bowl was carried out by means of the EVAL-DCP-N software. The analyzed results are concentrated in Table 5. In this table, the subgrade's CBR values were calculated from the subgrade's Modulus of Elasticity values by means of the following equation, [8]:

$$\text{CBR} = [\text{Es}/k]^{1.41} \quad (3)$$

where,

Es is the subgrade's Modulus of Elasticity in kg/cm^2.

CBR is the subgrade's CBR in %.

k is the regression coefficient obtained, and its value is 176.

Table No. 5: Subgrade's CBR values calculated from the subgrade's Modulus of Elasticity values

No. of points	Points No.	Average CBR value in %	Average CBR value minus one SD in %
	Concrete Block Pavement		
11	6-11; 47-48; 34-36	11.1	3.0
9	37-41; 43-46	5.1	3.7
	Asphalt Pavement		
12	49-55; 1-5	9.2	Negative value
22	23-33; 12-22	3.7	1.3

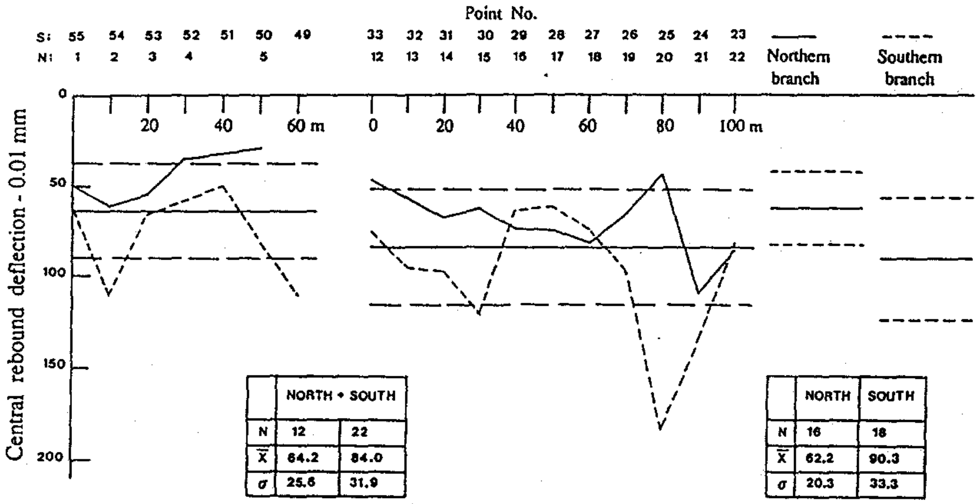

Fig. 12: B.B. central rebound deflections in the asphaltic section.

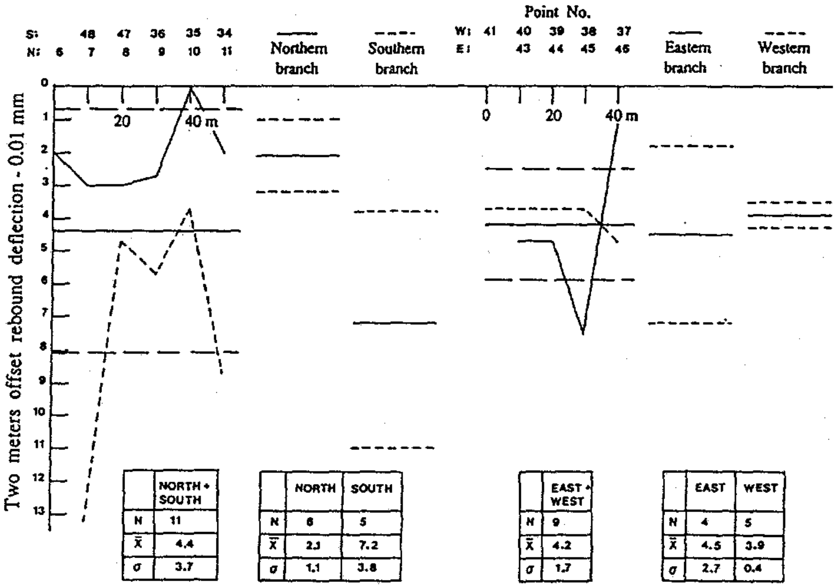

Fig. 13: B.B. 2.0 meters offset rebound deflection in the concrete blocks section.

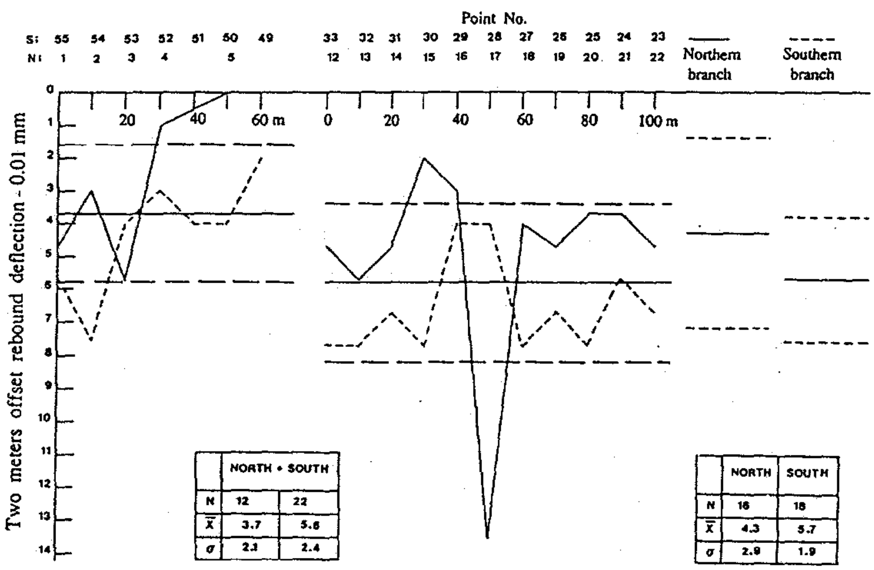

Fig. 14: B.B. 2.0 meters offset rebound deflections in the asphaltic section.

Obviously, Table No. 5 too indicates that the asphalt pavement subgrade's CBR value is lower than that of the concrete block pavement. This fact is also possible according to the strength tests described in preceding sections and reiterated in Table No. 6.

Table No. 6: Subgrade's CBR values calculated from DCP values

Points No.	Average CBR value in %	Average CBR value minus one SD in %
Concrete Block Pavement		
7, 10, 34	12.5	2.7
Asphalt Pavement		
19, 23, 28	8.2	3.8

To conclude, the calculation of the structural Modulus of Elasticity, Ep, indicates that the Modulus of Elasticity of the asphalt pavement is greater than the Modulus of Elasticity of the concrete block pavement. The results of the ratio Ep/Es also indicate that the asphaltic structure is stronger than the CBP structure. These results are given in Table No. 7. This fact goes contrary to expectations, as the purpose of the CBP solution is to increase the structural Modulus of Elasticity as compared with an asphaltic structure.

Table No. 7: Structural Modulus of Elasticity values and the ration of Ep to Es.

N No. of points	Points	Average Structural Modulus of Elasticity in kg/sq.cm.	Avg. Ep/Es ratio	Average Structural Modulus of Elasticity minus 1 SD in kg/sq.cm.	Avg. Ep/Es ratio minus 1 SD
20	The entire CBP area	1135	1.7	574	0.9
34	The entire asphalt area	2002	4.4	1114	1.8

Table No. 7 once more emphasizes the fact that the Modulus of Elasticity of the concrete block structure is inferior to the Modulus of Elasticity of the asphalt pavement structure. According to the SHELL Company method the proper Ep/Es ratio for 550mm thickness should be 3.4.

Number of Applications and Summary of B.B. Tests

By calculating the subgrade's maximal strain value, it is possible to determine the number of applications, as follows:

$$\epsilon c = 1.05 \times 10^{-2} \times [1/W]^{0.223} \qquad (4)$$

where,

- ϵc is the maximum strain pressure on the surface of the subgrade (when loaded by a standard dual-wheel axle).
- W is the number of movements made by a standard axle (18,0001b.).

The above equation is taken from CHEVRON (or the ASPHALT INSTITUTE) and is described in [9] or in [10]. It was used to obtain the results of the log average number of applications, being 4.9-5.9 for the concrete block pavement and 5.2-5.9 for the asphlat pavement. The log average number of applications minus one standard deviation is 4.7-4.8 and 4.5-4.7 respectively.

These results indicate that the carrying capacity of the concrete block pavement is essentially identical to the carrying capacity of the asphalt pavement and corresponds to the Israeli PWD light traffic criterion. The equality of the carrying capacities stems from the fact that the concrete block pavement has a stronger subgrade and a weaker structure, while the asphalt pavement has a weaker subgrade but a stronger structure.

Finally, all the Benkelman beam measuremnts lead to the following evaluation parameters:

a. The subgrade CBR of the concrete block pavement is higher than the subgrade CBR of the asphalt pavement.
b. The equivalent structural Modulus of Elasticity of the asphalt pavement is higher than that of the concrete block pavement.
c. The equivalent structural Modulus of Elasticity of the asphalt pavement and the concrete block pavement, indicates the inferiority of both structures.
d. Because of the above combinations, the carrying capacity of the asphalt pavement is essentially identical to the carrying capacity of the concrete block pavement and its value is equal to the Israeli PWD light-traffic criterion, (according to both evaluation methods).

Summary, Conclusions and Recommendations

In a previous paper [1], the authors recommended a CBP solution for the rehabilitation of existing asphalt pavements based on problematic subgrades such as swelling clays or collapsible loess. The passing of time has enabled the authors to follow up on this type of solution at the Beit-She'an junction in Israel. The follow up examination was accompanied by field and laboratory tests, including, among others, the digging of test pits and the execution of strength tests such as DCP, Vane and SPT. In addition to that, structural evaluation was carried out by means of back calculations using measured deflection bowls, resulting from the standard Benkelman beam procedure.

The analysis of the above field and laboratory tests indicates that the reasons for failure in the asphaltic pavement, prior to the CBP solution, stemmed mainly from the structural sub-design. This sub-design includes (a) the presence of weak points in the subgrade, as expressed in low CBR values originating during construction or later as a consequence of swelling, (b) the existence of weak points in the granular structure which are expressed in low Modulus of Elasticity values, and (c) having structural sub-thicknesses at a number of random points. At this juncture it should also be mentioned that the above sub-design points made the application of asphaltic overlays from time to time inappropriate for the rehabilitation solution. The existence of these weak points in the entire structure (including the subgrade), dictates a sort of "root canal" rehabilitation solution for the existing structaure which would include the weak layers and at least the upper layers and the base-course. Obviously, such a solution should also make up for decreased pavement thickness in those places where such thickness is lacking.

Similarly to the asphaltic pavement, the field and laboratory tests reveal that the CBP structure does not provide a fully structural solution for the intensity of the given traffic laod. This is evidenced by the rutting tracks which occured along the wheel tracks of the heavy vehicles on the surface of the CBP. The reasons for the development of these rutting tracks are identical to those cited for the failure of the asphaltic pavement. However, at the same time, it is important to emphasize that, contrary to expectations, the CBP pavement's Modulus of Elasticity was found to be lower than that of the asphaltic pavement. This may stem from the way in which the blocks were laid, which may have been improper, thus preventing the formation of a plausible interlocking between the blocks. Despite this, the solution of interlocking CBP at the junction is still better than the asphaltic one, even considering the structural weakness of the CBP solution revealed in the tests. The above statement is due to the basic characteristics of the CBP pavement, by which the interlocking blocks create a multi-joint layer, thus not necessarily making the structural failure a functional one too. In other words, the CBP pavement has the best ability to cope with problems of variable uneven plastic deformations induced in the pavement. Obviously, in the given case study, had the interlocking block layer been placed well, it would have "built" a higher Modulus of Elasticity and the failure would have thus been avoided.

In this matter of the basic quality of CBP, the following excerpt from [8] constitutes a good description:

" (a) This pavement has the best capacity to contend with problems of irregular plastic deformations in the pavement. Plastic deformations which are expressed in rutting phenomena in flexible pavements stem from the over-sensitivity of the concrete asphalt to temperature and/or to subgrade sinkage. The first problem does not exist at all in the concrete blocks which are impervious to temperature, while the second problem can be very easily dealt with when it takes place, by removing the blocks, remedially treating the subgrade and reinstalling the blocks in their proper place."

(b) The phenomena of the various types of cracks and breaks which exist in both flexible pavements (in the asphaltic layer) and rigid pavements (in concrete slabs), are almost nonexistent in the concrete blocks. The reason for this is that the latter type of pavement is in fact a jointed road comprised of small links, which, on account of their short length, hardly develop any of the tensile stresses which are the prime cause of cracking phenomena in other types of pavement. Moreover, even if occasional cracking or breakage do occur in the concrete block paving, the replacement of a faulty block with a new one does not present a practical problem"

In conclusion, the given case study indicates the following recommendations: (a) The CBP rehabilitation solution should continue to be examined in problematic failure areas. (b) The interlocking block solution in these areas will have a higher potential of success if conditions are created for "building" a higher Modulus of Elasticity in the interlocking block layer.

REFERENCES

1. Livneh, M., "An Example of Pavement Rehabilitation by Concrete Block Paving in Expansive or Collapsible Soil Regions," 3rd Int. Conf. on Concrete Block Paving, Rome, May 1988.

2. L.C.I. - Transportation Engineers Consultants, "The ROUGH-3 Software for the Evaluation of Runway Surface Roughness and the Selection of Rehabilitation Alternatives - User's Manual," prepared for the Civil Engineering Department of the IAF, March 1988. (in Hebrew).

3. The Technion Institute for Research and Development Ltd., The Soil and Road Testing Laboratory, "Soil tests and Deflection Bowl Measurements at the Sha'ul Hamelech, Hahistadruth junction in Beit She'an," Job No. 4370/88, July 1988 (in Hebrew).

4. The Technion Institute for Research and Development Ltd., The Soil and Road Testing Laboratory, "Continued In-situ and laboratory tests of the Sha'ul Hamelech, Hahistadruth junction in Beit She'an," Job No. 4370/88, August 1988 (in Hebrew).

5. Livneh, M., "In-Situ CBR Testing by Indirect Methods", Proc. XIIth Int. Conf. on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Brazil, August 1989.

6. Wiseman, G., Komornik, A. and Greenstein, J., "Experience with Roads and Buildings on Expansive Clays," Transportation Research Record No. 1032, Transportation Research Board, 1985.

7. The Asphalt Institute, "Asphalt Overlays and Pavement Rehabilitation", Manal Series 17 (MS-17), 1977.

8. Livneh, M., "Design and Evaluation Parameters of Various Pavement Structures", Research Report No. 88-129, Transportation Research Institute, the Technion - Israel Institute of Technology, November 1988, (in Hebrew).

9. Santucci, L.E., "Thickness Design Procedure for Asphalt and Emulsified Asphalt Mixes," Proc., Vol. I., 4th International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, Michigan, 1977.

10. Shook, J.F., Finn, F.N., Witczak, M.W., Monismith, C.L., "Thickness Design of Asphalt Pavements - The Asphalt Institute Method," 5th Int. Conf. on the Structural Design of Asphalt Pavements, Delft, Holland, August 1982.

11. Ishai, I., Livneh, M., and Livneh N.A., "The Effect of Vertical Confinement on the DCP Values in Subgrade and Pavement Evaluation," submitted to the 1992 Annual Meeting of the TRB.

12. L.C.I. - Transportation Engineers Consultants, Cambridge Systematics, Beny Allenbogen, "Pavement Management System - Stage A, An Economic and Engineering Survey of the Feasibility of Investing in the Maintenance of the Inter-Urban Road Network", prepared for the Public Works Department, Department of Maintenance and Operation, 1987, (in Hebrew).