Concrete Pavements With Continuous Reinforcement and Elastic Joints

BENGT O. E. PERSSON, Consulting Engineer, Täby, Sweden; and
BENGT F. FRIBERG, Consulting Engineer, St. Louis, Mo.

Attempts have been made to obtain the benefits of continuous pavement designs without the large amounts of steel in continuously reinforced and randomly cracked roads. The designs use weakened plane joints with steel ties crossing the joints (here called elastic joints) to form transverse hinges across the pavement. In common practice, contraction joints have been used intermittently as every fourth or fifth joint to avoid excessive stresses in the continuous steel in between.

Continuous pavements using elastic joints exclusively have been explored in Sweden. Controlled slab tension tests and joint-width measurements have given data on joint-cracking forces and movements and on full-section cracking. The effectiveness of different bar bond breakers and corresponding elastic joint forces was observed. An initial short highway installation indicated that 0.2 percent continuous steel and elastic joints could be used, with the bars coated with asphalt to prevent bond for a short distance under the elastic joints.

The highway installation was inspected after three years of use and found to be in excellent condition. No random cracks or other deterioration could be seen. Although traffic was light compared to that on American divided highways, a large percentage was trucks.

•CONTINUOUSLY reinforced concrete pavements have demonstrated that transverse cracks need not be detrimental to pavement performance. As usually built, the continuous steel is depended upon to cause closely spaced transverse cracks in pavement portions away from free ends. The tension forces transmitted by the steel across the cracks to the concrete must exceed the concrete tension strength to be effective for cracking at some critical early age; over 0.5 percent of steel section has been found to be necessary.

Some continuity across cracks is necessary, especially with respect to shear transfer, for satisfactory highway pavement performance. If cracks, or joints, become too wide, aggregate interlock may be lost, and faulting is a common and objectionable result in jointed pavements without doweling. In continuously reinforced pavements, a reasonably equal width of all cracks is assured by the uniformity of steel tension forces at adjacent cracks.

Continuous reinforcement, as generally used, maintains even crack widths because it has tension force capacity many times greater than the small forces needed to overcome variations in subgrade friction and crack-face tensions between short pavement elements. Any continuing functions of the large amounts of continuous steel are insignificant compared to its initial early-age objective—to promote cracking across the pavement section. In the mature pavement, crack widths at different temperatures appear

Paper sponsored by Committee on Rigid Pavement Design and presented at the 48th Annual Meeting.

to be directly related to crack spacing and temperature but not to the amount of steel. This indicates that cracks might be caused by less steel through the use of weakened planes across the pavement, resulting in substantial savings in steel quantities and possibly equally satisfactory performance. Elastic joint pavements are intended to meet this design objective.

ELASTIC JOINT PAVEMENTS

The term "elastic joint", as used in this report, refers to transverse joints across which steel ties connect the two pavement elements. As the elastic joint opens on decreasing temperature, the ties are strained in tension. Overstressing the steel at low temperatures is avoided by treating the parts nearest the joint to prevent bonding to the concrete. The steel then serves as an elastic tie across the joint.

Limited applications of elastic joint pavements have appeared in the United States, Switzerland, and Germany; in each case, free-moving contraction joints are employed at intervals. In Sweden elastic joint pavements have been investigated and applied to highway construction as fully continuous construction.

Limited forms of elastic joint pavements have been used in highway pavements in the United States. The Arkansas standard pavement design, in use since about 1955, consists of 45-ft reinforced pavement slabs between contraction joints, with two intermediate weakened-plane joints forming three 15-ft panels tied together by continuous wire mesh for the full 45-ft slab length. Examination of many miles of this pavement shows nearly complete absence of cracking and excellent performance.

A similar design used on Connecticut highways during the 1950's was less successful. It consisted of 100-ft mesh reinforced pavement slabs between expansion joints, with three intermediate weakened-plane elastic joints forming four 25-ft panels tied together by the continuous wire mesh. Some of the elastic joints have opened wide due to tension failure of the steel wires. The reason was determined to be increasing resistance to pavement contraction at the expansion joints as a result of dowels that rusted after some 5 to 10 years, and induced excessive tension in the longitudinal pavement reinforcement when temperatures decreased. Many expansion joints were immobilized at the narrow joint widths during summer temperatures, and the contraction and expansion functions were transferred to the failed elastic joints, which thereafter performed as undoweled expansion joints.

Elastic-joint pavements have been built as standard construction in Switzerland since about 1960, and in 1961 a test road using the design was laid in West Germany near Offenburg, between Karlsruhe and Baden-Baden. The pavements consist of 30-m (99-ft) slabs between contraction joints with three intermediate elastic joints forming four 7.5-m (25-ft) elements. All the joints are formed by a narrow strip of corrugated asbestos board on edge as base and a vibrated-in fiberboard joint crack guide at the top surface, including a 45-deg surface kerf cut. The elastic joints are tied across with 12-mm ($\frac{1}{2}$-in.) plain bars 1m (40 in.) long with end hooks spaced 40 cm (16 in.); the 70 cm (28 in.) at the center of each tie bar are coated with asphalt cutback. On the German test road of similar design, the observed maximum elastic joint openings were 0.8 mm (0.03 in.) average and 1.5 mm (0.06 in.) extreme width. The observed residual elastic joint openings averaged 0.02 mm (0.008 in.). The observed maximum openings at the contraction joints were 10 mm (0.40 in.).

CONTINUOUSLY REINFORCED PAVEMENTS WITH ELASTIC JOINTS

The investigations in Sweden are of particular interest because their objective is the development of fully continuous pavements (1, 2, 3). The investigations covered:

1. Pullout bond tests of plain bars with different treatments;
2. Controlled tension tests of narrow slabs on subgrades, with different dimensions of weakened-plane crack starters to determine the joint-cracking and slab-cracking forces and the relationship between tension forces and crack openings for bars variously treated at the elastic joints; and

3. Performance of an exploratory highway pavement installation after two years of use, consisting of 340 ft of 6.3-in. thick pavement with elastic joints spaced 13.5 to 20 ft and reinforced with 0.2 to 0.4 percent of continuous plain bars, including joint width measurements of the elastic joints.

The controlled tension tests were made at a site near Stockholm. The exploratory highway was installed in 1964 on European expressway E4/E6 in southern Sweden. A second longer test road is in the planning stage. The tests were made from 1964 to 1966. All work is financed by the State Research Council.

PULLOUT TESTS

Initial data on slip resistance of plain round steel bars with different coatings were obtained from pullout tests on 32 concrete specimens with embedded bars, tested in tension. Concrete specimens 16 by 16 cm (6 by 6 in.) in cross section and 1 cm (40 in.) long were used. The 12-mm or 16-mm bar (0.47 in. or 0.63 in.) to be tested was embedded 50 cm (20 in.) in the center of the cross section and was pulled from one end. The concrete specimen was anchored by a heavy deformed bar similarly embedded and extended from the opposite end. Loads were observed for beginning slip, maximum slip resistance, and continuing sliding. The tests included bars uncoated and bars coated with cutback asphalt (RMA 15), epoxy, epoxy and sand, and an epoxy-tar formulation. Table 1 gives the test results in summary. Bar stress is given for initial slip and maximum resistance; bond stress for continuing slip is based on 50-cm (20-in.) embedment.

TENSION TESTS ON JOINTED SLABS ON SUBGRADE

An extensive series of applied-tension tests was performed on narrow slabs reinforced with continuous steel and provided with weakened-plane type joints placed on prepared subgrade. The slabs were 16 cm (6.3 in.) thick and 50 cm (19.7 in.) wide; 46 slabs were 5 m (16.4 ft) long with one weakened plane at midlength of most, and 6 slabs were 20 m (65.6 ft) long with three weakened planes at the quarter points. All slabs were placed on a sand base covered with a thin plastic film. The slabs were enclosed under temporary shelters as protection against weather and frost and were tested after about one-half year.

The continuous reinforcement consisted of two plain bars at a center depth of 12.5 cm (4.9 in.) from the sides, with diameters of 12, 16, and 19 mm (0.47, 0.63, and 0.75 in.), equivalent to 0.27, 0.47, and 0.67 percent reinforcement. In a majority of the slabs, the bars were coated with RMA 15 asphalt over a distance of 75 or 150 cm (29 or 59 in.)

TABLE 1

SUMMARY OF PULLOUT TESTS ON PLAIN BARS
(Average Values)

Coating and Bar Size	Initial Slip		Maximum Resistance		Continuing Slip			
	Bar Stress		Bar Stress		Bar Stress		Bond Stress	
	kp/cm²	psi	kp/cm²	psi	kp/cm²	psi	kp/cm²	psi
Uncoated bars								
12-mm (0.47 in.)	2,580	36,700	3,600	51,200	2,170	30,800	13	180
16-mm (0.63 in.)	1,390	19,800	2,760	39,200	2,140	30,400	17	240
Asphalt coated, RMA 15								
12-mm	150	2,130	450	6,400	300	4,720	1.8	26
16-mm	140	2,000	580	8,250	460	6,550	3.7	52
Various epoxy coatings (average)[a]								
12-mm	2,950	42,000	3,780	53,800	2,740	39,000	16	230
16-mm	1,860	26,500	2,850	40,500	2,320	33,000	19	260

[a]Two bars, which were coated with epoxy tar, are not included. They had bond values about one-half of the values for uncoated bars.

centered under the weakened planes. Other details in some tests, such as deformed bars, wire mesh between the joints, and other coatings, did not provide sufficient information for appraisal.

The weakened-plane joints consisted of 3-mm ($^1/_8$-in.) Masonite strips as crack starters, vibrated into place at the midlength of the 5-m slabs and at the three-quarter points in the 20-m slabs. In most slabs the crack-starter depth was 5 cm (2 in.). Different depths and shapes from 1.9 to 8 cm ($^3/_4$ to 3 in.) were tried; the $^3/_4$-in. depth was insufficient to localize the crack, and the $^3/_4$-in. crack starters were deepened by saw cuts. In 14 slabs no crack starters were used; instead, coating on the continuous bars was varied to serve as crack localizers. Results of these tests proved inconclusive and are not included in the discussion.

Longitudinal tension on the slabs was applied through header beams beyond both ends. Four deformed bars were embedded in each slab end and protruded through holes in the steel header beams where they were anchored by wedges. Force against the header beams was applied by two hydraulic jacks, one on each side of the slab, bearing against 6 by 6-in. timbers as reaction columns that extended between the header beams. Changes in joint width and subsequent full-section cracks were measured for increasing jack loads with a 10-in. Huggenberg gage between points on the top surface. The longitudinal tension force was centered at middepth of the 16-cm slabs.

Cracking at the joints below the crack starters was visible at the sides of the slabs. Cracked joints were observed before tension application in only two 5-m slabs; however, such initial cracks were observed at most of the joints in the 20-m slabs. A distinct increase in the rate of extension across the joint with increasing tension force was considered as a joint-cracking load. Figure 1 shows typical joint cracking for the three joints in a 20-m slab. Figure 1 also shows cracking at joints in two 5-m slabs that showed no indications of cracks at the sides of the slabs before testing.

Figure 2, prepared from tabulated data, shows the relationship between joint opening (as measured by extension on the 10-in. gage length) at cracking and the average stress on the joint net section below the crack starter for slabs reinforced with 12-, 16-, and 19-mm bars. The data show two groupings of the cracking or joint activation: group 1, averaging below 0.002-in. extension, apparently applicable to joints uncracked before tension application, and group 2, averaging above 0.005-in. extension, believed to apply for joints cracked before tension application, even if not visible. In the latter group the change in rate of extension, rated as cracking, might be due to bond slip. With reference to Figure 2, the force at which a substantial rate of joint widening began increased with bar size, from 6.1 to 9.4 to 11.8 kp/cm^2 average stress on the joint net section for 12-, 16-, and 19-mm bars. Table 2 gives average net section stress and extension at cracking for the two groups with typical joint behavior. The joint cracking force was lower for the joints with 150-cm coated bars, but the joint cracking width was nearly the same for 75- and 150-cm coatings with the cutback asphalt.

Full-section cracks occurred in nearly all slabs tested. The force required for cracking away from the joints averaged 3.8, 2.7, and 2.3 times that required for joint activation for the slabs reinforced with 12-, 16-, and 19-mm bars respectively. The even stress distribution on the concrete section, as well as bars in bond, would account for the great increase in force necessary for full-section cracks. The use of crack starters of different depths in some slabs with 16-mm bars caused the relationship between cracking at the weakened plane and full section, as shown in Figure 3. The effect of eccentric loading on the joint net section is apparent in the disproportionate decrease in cracking force for the net section.

Longitudinal deformation measurements in connection with the tension tests (3) show the value of asphalt coatings on the bars for keeping steel stresses at elastic joints within limits. Typical relationships between steel stress and joint opening are shown in Figure 4 for 12-mm and 16-mm bars. For example, asphalt coating of 29 in. at the elastic joint resulted in steel stress of 55 and 35 ksi respectively for a 1.5-mm joint opening.

Without asphalt coating the steel stress in 12-mm bars reached yield-point stress of about 85 ksi at a joint opening of 0.05 in. Residual joint openings after tension removal are shown on the left side of Figure 4. They increased linearly in proportion to joint

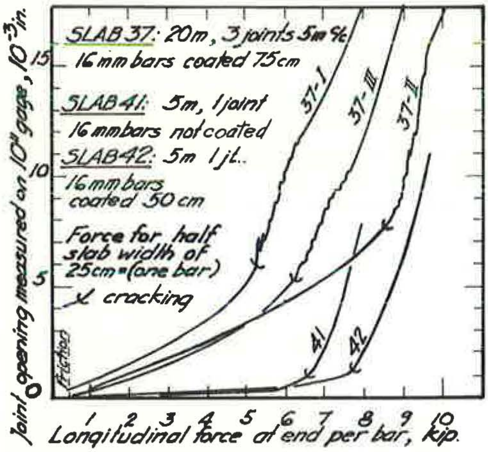


Figure 1. Relationship between tension force and length change across elastic joints for typical conditions. The joint was apparently uncracked before tensioning in slabs 41 and 42. The three joints in slab 37 were apparently cracked before tension application at the two ends of the slab.

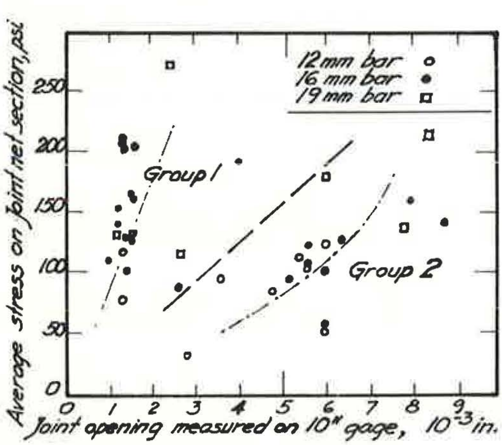


Figure 2. Average stress on the joint net section at which a large increase in the rate of joint opening took place, as measured on the 10-in. gage length across the crack starter, for different slabs. The results on different slabs fall into two groups (separated by the dashed line). Activation of the joint in the slabs of group 1 apparently coincided with cracking of the concrete below the crack starter. The joints in group 2 slabs apparently were cracked before tension application, and the change in rate of joint widening with tension was related to extended bond slip.

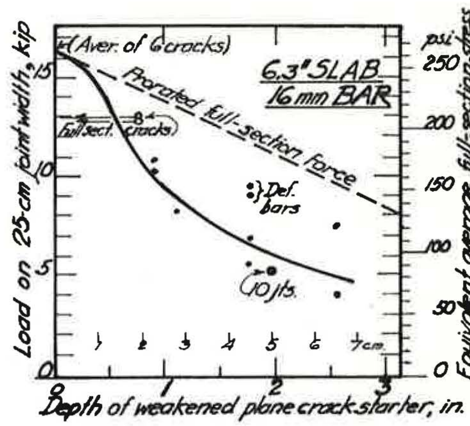
TABLE 2

STRESSES AND EXTENSION ACROSS JOINTS AT JOINT CRACKING

Plain Bar Size, mm	Coated Bar Length, cm	Group 1 Small Extension Across Joint: No. of Tests	Net Section Stress, kp/cm²	Ext. Across Joint at Cracking, mm	Group 2 Large Extension Across Joint: No. of Test	Net Section Stress, kp/cm²	Ext. Across Joint at Cracking, mm
12	75	1	9.1	0.035	7	5.4	0.121
	150	2	6.9	0.033			
16	75	5	11.0	0.031	7	8.6	0.165
	150	3	8.4	0.046			
19	75	2	14.1	0.049	2	13.8	0.180
	150	2	8.7	0.049	1	9.7	0.196

Data are from Table 6 of Persson (1).
Extension values are for 10-in. gage length across joint.
Net section stress is the total force, deducting friction between the slab end and the joint, divided by the joint net section area below the crack starter, without consideration of transformed steel area. It is an average value.

Figure 3. Cracking force on the weakened-plane joints in relation to the depth of the crack starters. Cracking stress lower than that prorated from full-section cracking stress can be explained by eccentric force on the joint net section.



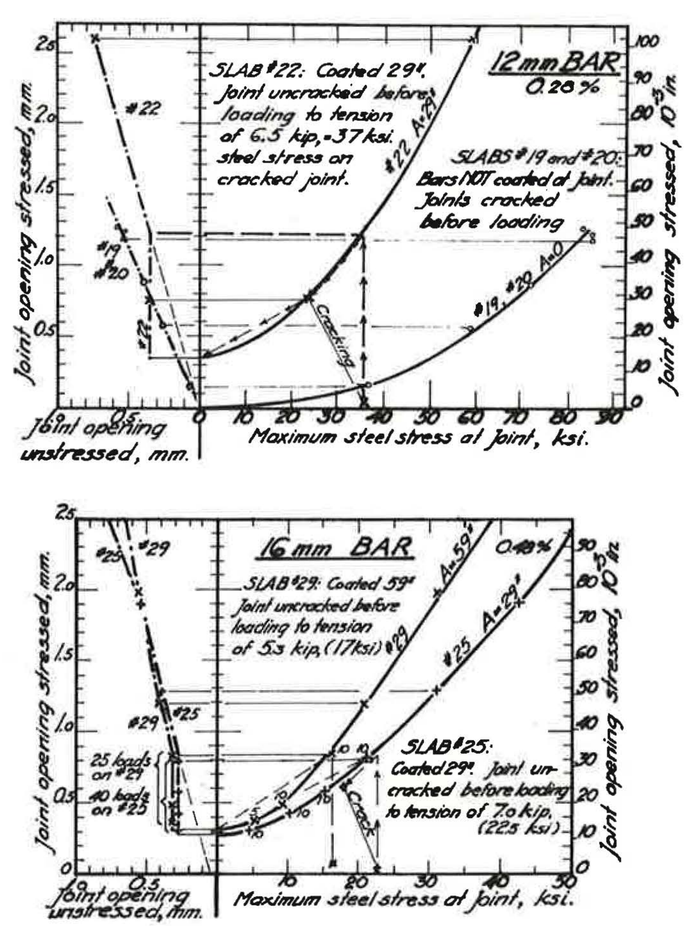


Figure 4. Relationship between joint opening and increasing bar stress in the continuous steel with asphalt coating. Data reveal the joint widening incident to joint cracking under tension stress as a permanent set joint width.

openings under tension; however, joints that cracked during tension testing assumed a constant residual opening, that did not change, even for many repeated tension applications, until the tension force exceeded the cracking force.

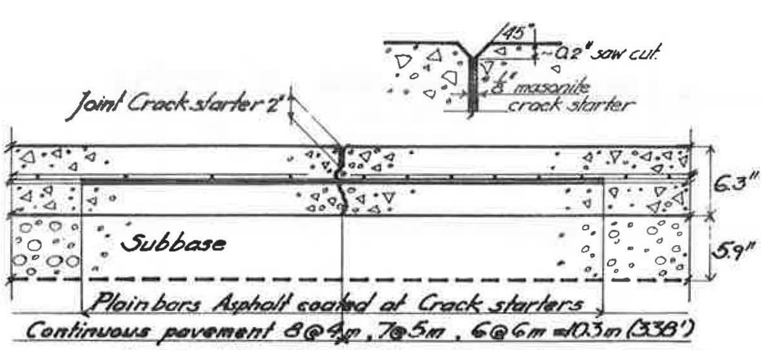
EXPLORATORY PAVEMENT AT HOFTERUP AFTER TWO YEARS OF USE

The exploratory concrete pavement (2) was placed on September 29, 1964, on the divided highway E4/E6 in the southern part of Sweden about 30 km north of the city of Malmö. It is 8 m (26 ft) wide and 103 m (340 ft) long, and consists of eight 4-m, seven 5-m, and six 6-m slabs (13.2 ft, 16.5 ft, and 19.7 ft), all 16 cm (6.3 in.) thick. The continuous reinforcement was 16-mm (5/8-in.) plain bars of 75,000-psi yield strength steel, spaced 40 cm (16 in.) in the 4- and 5-m slabs and 30 cm (12 in.) in the 6-m slabs, giving 0.31 and 0.42 percent steel, except that four of the 4-m slabs had 12-mm (0.47-in.) bars spaced 35-cm (14-in.) or 0.2 percent steel. Fifteen of the 19 slabs also were individually reinforced with wire mesh with from 0.04 to 0.16 percent longitudinal steel which stopped back from the transverse joints. The experimental pavement was placed on 15 cm (6 in.) of open-graded granular base and was separated at both ends from the 8-in. thick conventional pavement of 33-ft slabs on cement-treated base by contraction joints with 5/8-in. dowels spaced 16 in. center-to-center. Construction temperatures averaged 15 C (60 F).

The continuous bars were at middepth and were coated with RMA cutback asphalt of low viscosity for a distance of 1.4, 1.7, and 2.0 m (55, 67, and 79 in.) centered at the elastic joint locations between the 4-, 5-, and 6-m slabs respectively. The joints consisted of 1/8-in. bitumen-coated hard Masonite crack starters 5 cm (2 in.) deep, placed in cut grooves and troweled smooth. At three weeks of age a 5-mm (0.2-in.) deep 45-deg kerf was sawed in the surface at each transverse joint line. Construction details are shown in Figure 7.

Points for joint-width measurements with a 10-in. Huggenberg gage were set in the plastic concrete. Figure 6 shows joint widening at 3 and 20 days as measured with the gage. Of the 20 joints, 9 showed clear crack indications at 3 days. At 20 days all joints appeared to be working except the first three joints between 4-m slabs, at which the bars were displaced by the paving operations, partially locking those joints.

Figure 7 shows joint movements from the lowest to the highest observed temperatures, February to July 1966, in relationship to joint widths at one year, September 1965. The extreme air temperatures, -11 C (12 F) and 38 C (100 F) correspond to a change of approximately 70 F in average slab temperature, equivalent to unrestrained



Group	Slabs No.	Slabs Length	Reinforcement Size	Reinforcement Spac.	Reinforcement %	Reinforcement Coated	
I	4	13'-2"	.63"	15.8"	.31	55"	Light mesh of varied size and spacing between joints
	4	-"-	.47"	13.8"	.20	55"	
II	7	16'-5"	.63"	15.8"	.31	67"	
III	6	19'-8"	.63"	11.8"	.42	79"	

Figure 5. Design details of joints in the exploratory pavement with elastic joints and continuous reinforcement, built in 1964, with 0.20 to 0.42 percent steel and 13- to 20-ft joint spacing.

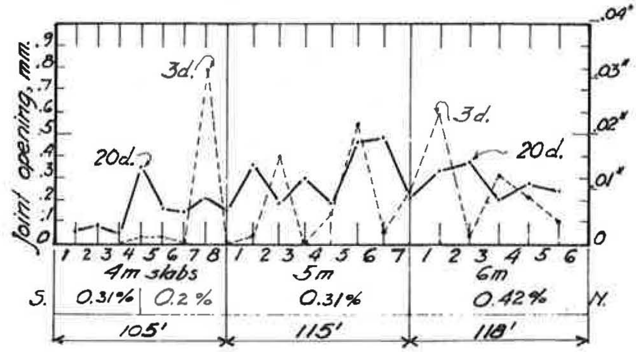


Figure 6. Elastic joint widths at 3 and 20 days of age. Air temperature at construction 59 F, at 20 days 50 F. About half of the joints were uncracked at 3 days; at 20 days, all joints appeared to be working as intended, except the first three joints at the south end, where the continuous bars were misaligned by paving operations.

length changes and average observed length changes as follows (coefficient of thermal expansion 0.0000055 in./in./deg F):

Slab	Unrestrained Change	Observed Change
4-m (13.2 ft)	0.060 in.	0.03 in.
5-m (16.5 ft)	0.075 in.	0.03 in.
6-m (19.7 ft)	0.090 in.	0.02 in.

No observations were made of width changes of the joints at both ends where large compensating movements may have taken place during seasonal temperature changes.

The measurements of joint openings on the test road show maximum joint opening at low temperatures of about 0.9 mm for slabs reinforced with 12- and 16-mm continuous bars coated at the joints for distances of 1.4 m (55 in.) and 1.7 m (67 in.) respectively. Assuming steel stress and joint opening relationships on the test road to be

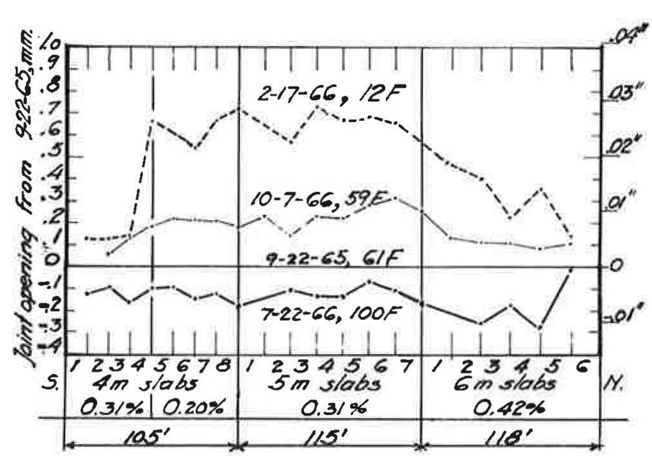


Figure 7. Changes in elastic joint widths on the exploratory pavement from one to two years of age. The measurements include the low and high air temperatures, 12 and 100 F. Gage points set on the day of construction were lost during the first year.

the same as illustrated for the tension tests in Figure 4, the maximum tension stress in the 12-mm bars (0.47 in.) would have been at most 25,000 psi, and in the 16-mm bars (⅝ in.), perhaps 15,000 psi. The approximate corresponding concrete stresses away from the joints, based on 0.20 and 0.31 percent reinforcement, would be 50 and 45 psi respectively maximum at the extreme low temperature. The increase in the low-temperature joint widths from the north end toward midlength according to Figure 7 indicates that subgrade friction was a primary stress-producing factor. At midlength, about 150 ft from each end, a concrete stress of 45 psi would correspond to an average friction coefficient of about 0.3 (assuming a concrete weight of 1 lb/ft length per in^2 cross section).

Steel stresses in the continuous reinforcement in the test road were quite low for the largest observed joint width, in accordance with the indicated stresses for up to 1-mm joint openings in Figure 4. The joints were even; 0.2 percent steel was apparently sufficient to provide uniform joint movements. The bond prevention by asphalt coating of the continuous bars through the joints worked as intended and limited steel stresses effectively to very low maximums.

No random cracks have occurred to date in the test road. Traffic is heavy, including continental traffic to Norway and to central Sweden. The surface of the test road, as of adjacent conventional pavement, is excellent.

REFERENCES

1. Persson, B. O. E. Concrete Pavements With Continuous Reinforcement and Elastic Joints: Pullout Tests and Study of Crack Formation and Joint Opening. Cement och Betong, No. 2, pp. 3-29, 1967.
2. Persson, B. O. E. Concrete Pavements With Continuous Reinforcement and Elastic Joints: Test Road at Hofterup After Two Years Traffic. Cement och Betong, No. 2, pp. 1-16, 1967.
3. Persson, B. O. E. Concrete Pavements With Continuous Reinforcement and Elastic Joints: Longitudinal Deformation Measurements. Cement och Betong, No. 3, pp. 1-24, 1967.