NON-CONVENTIONAL VS. CONVENTIONAL CONCRETE PAVEMENTS IN ARIZONA

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Pavement faulting or step-off has been one of the major problems on Arizona's Portland Cement Concrete Pavements. Other problems have included warping and curling, texture loss, and shoulder joint failures. In 1974 work was initiated in an attempt to solve some or all of these problems by building and studying "non-conventional" PCCP sections on State Route 360, The Superstition Freeway. These projects are adjacent to a conventional section, i.e., 23 cm (9 in.) PCC, 15 cm (6 in.) CTB, and 10 cm (4 in.) aggregate base. To date, full-depth 33 and 28 cm (13 and 11 in.) PCCP on subgrade and prestressed 15 cm (6 in.) PCCP on a 10 cm (4 in.) lean concrete base have been built. A 23 cm (9 in.) PCCP on 10 cm (4 in.) of Lean Concrete Base is under construction. The main items of study are joint performance and ride quality. Full depth PCCP results in lower joint deflections in comparison to conventional PCCP. The extra slab thickness and joint surface area do not overcome the effects of joint separation from shrinkage due to curing and temperature changes. Loss of rideability is somewhat less on full depth versus conventional PCCP.

Arizona built its first major portland cement concrete pavement (PCCP) in the mid 50's. In total approximately 650 lane-kilometers (400 lane-miles) have been built. The major problems that have occurred are joint faulting, slab warping and curling, surface texture loss, and shoulder joint failures.

Shoulder joint failures have been corrected on existing pavements by creating a 1.25 cm (1/2 in.) wide joint between the asphaltic concrete shoulder and filling it with an asphalt-rubber sealant. On new construction, concrete shoulders are used to eliminate the problem area. The loss of surface texture has been slowed by specifying a transverse wire-tine texture. Research projects, both within and without Arizona have shown this texture more durable than any other tried.

At one time, our construction specifications called for metal insert weakened plane transverse control joints every 15.24 m (60 ft.). Intermediate weakened plane transverse joints, at a 3.8 m (15 ft.) average spacing, were sawed at a later time. The end result was a fairly rough joint every 15.24 m (60 ft.) due to either poor finishing at that joint, accentuation of the warping and/or curling due to that joint being the first joint formed, or some other unknown cause. The specifications now require all joints to be saw joints at an average spacing of 3.8 m (15 ft.) (13 ft., 15 ft., 17 ft., repeat). This has eliminated problems occurring at the insert joints.

The main problem remaining is joint faulting. Our experience has been similar to other states in that our PCC Pavements begin to fault after 8 to 12 years of traffic. We have found this to occur both on PCCP having an aggregate base material and on PCCP having a cement-treated base material. The mechanism for this faulting is believed to be the migration of loose material to the area under the "approach" slab (1,2). The build-up of this material eventually results in permanent faulting at the joint. It is believed that the faulting could be prevented either by eliminating the source(s) of loose material, by preventing differential movement of the slabs forming the joint or by reducing the deflections, both differential and total, to an amount too low for the faulting mechansim to work.

In 1975 in an attempt to solve these problems we began what would become a series of experimental projects on the Superstition Freeway.

This freeway runs from its junction with Interstate 10 East toward Apache Junction where it will eventually join U.S. 89. Although not an Interstate route, it has been designed to interstate standards and carries similar traffic. When completed the route will be 42 km (26 mi.) long; currently 6 projects have been placed for a total of 15 km (9 mi.). The first project was a conventional pavement, built in 1972. It consisted of 23 cm (9 in.) of PCC on 15 cm (6 in.) of cement treated base and a 10 cm (4 in.) gravel subbase. The second project was of similar original design, however the ADOT Research Section proposed a change order of 33 cm (13 in.) PCCP (including shoulders) on subgrade.

There are many theoretical reasons to support the concept of full depth PCCP, such as reduced warping and curling, lower deflections, and larger joint surface all of which were expected to improve joint performance and consequently reduce joint faulting. Hopefully, this reduced warping and curling would result in more complete subgrade contact with a concurrent reduction in the pumping potential at the joint. Also, the thicker slab was expected to have reduced deflections due to its greater section modulus. In addition, the greater mass could result in a reduction of dynamic deflection due to its higher inertia (2). It was also anticipated that the greater cross-sectional area of the joint would provide more interlock and hence better joint performance. Another advantage would be the simplification of the construction process by eliminating the base material. (The contractor claimed that 60 days construction time was saved.) In view of these possible benefits, it was agreed to accept the proposed change order.

Following completion of the second project in 1975, considerable interest developed in the concept of a pre-stressed, post-tensioned concrete pavement. It was envisioned that this concept would result in a substantial saving of concrete. The prestress concept would also allow joints to be spaced 120 to 215 meters (400 to 700 ft.) apart which, at least in theory, would provide a superior ride due to the absence of intermediate joints plus reduced maintenance since most maintenance is at the joints. Consequently, the third project on the Superstition Freeway was made into a formal research project and a pre-stressed, post-tensioned PCCP was constructed and was completed in 1977.

Discussions then turned to the design for the next section. In view of the problems of joint faulting on conventional PCCP the ADOT did not wish to return to that concept, nor did we want to build another pre-stressed pavement until we had a chance to evaluate the one already built. It was decided to build another 33 cm (13 in.) PCCP section as a formal research project. Since the original choice of 33 cm (13 in.) as the design thickness was somewhat arbitrary from the standpoint of structural analysis, it was suggested and agreed that a portion of the project be 28 cm (11 in.) PCCP so that a comparsion could be made between the performance of the 28 and 33 cm (11 and 13 in.) thicknesses. This fourth project was completed in 1979. Currently two additional projects are under construction. One is 33 cm (13 in.) PCCP on subgrade and the other is 23 cm (9 in.) PCCP on 10 cm (4 in.) Lean Concrete Base (LCB).

The 23 cm (9 in.) PCCP on 10 cm (4 in.) LCB is based on work done by California (1). The theory behind this design is that by using LCB (a non-erodible base) and concrete shoulders there will be no source of the loose material that builds up under the slabs and causes faulting.

The previous projects using 33 cm (13 in.) and 28 cm (11 in.) PCCP on subgrade were designed to prevent the build-up of loose material by preventing differential slab movement of the joint and by reducing the magnitude of the deflections that did occur.

Although it had not originally been intended as such, the Superstition Freeway thus evolved into a PCCP test road. Table 1 summarizes the various project designs and subgrade properties. Figure 1 shows the typical sections for the projects.

Test Program

Beginning in 1974, a Ride (Mays-meter) survey has been made annually on this highway. Starting in late 1976 and continuing thereafter additional ride testing was scheduled for various times throughout the year to obtain a better idea of the change in ride versus time then could be obtained with just once a year testing.

On February 27, 1980 deflection measurements were made on the most recently completed project on SR 360, the 1979 33 cm (13 in.) and 28 cm (11 in.) full depth PCCP section. Joint load tests were made in the WB high speed lane by moving an 80.1 kn (18 kip) single axle load across the joint from one slab to the next and measuring the deflections. Initial readings were made with a benkleman beam but because of their relatively low precision and the fact that only the deflection on one side of the joint could be measured at a time, the use of the benkleman beam was discontinued. Two dial gauges were then mounted on a wooden beam so that deflection could be measured adjacent to the outside of the dual wheels on both sides of the joint simultaneously. Concurrent with this, deflection measurements were taken with a Dynaflect with sensors 1 and 2 located equal distance on either side of the joint and with the force wheels at the #1 sensor location. Dynaflect readings were also taken at the midpoint of some slabs. Testing with the Dynaflect proceeded smoothly. Testing with the dial gauges was somewhat slow as it was difficult to maintain a zero setting, possibly due to the somewhat windy conditions.

On February 28, 1980 the same measurements were made on the 1975 33 cm (13 in.) PCCP section and the 1972 conventional 23 cm (9 in.) section. Measurements proceeded smoothly with both the Dynaflect and the dial gauges. On March 5, 1980 additional Dynaflect testing was performed on the 1975 33 cm (13 in.) section, further east than the tests done on February 28, 1980. The results were similar to the February 28, 1980 results.

On March 26, 1980 15 cm (6 in.) diameter cores were taken from 5 locations on the 1975 33 cm (13 in.) PCCP project both from the center of the slabs and from the joints. None of the cores was retrieved intact, all broke horizontally in one or more locations in addition to the crack forming the joint. On March 27, 1980, two additional cores were taken at the far east end of the WB roadway. This area is in the 1979 33 cm (13 in.) PCCP project. One core was retrieved intact. The other broke due to the joint having formed at an angle which nearly intercepted the core cut. On March 28, 1980, two additional cores were taken in mid-slab in the far east end of the 1979 33 cm (13 in.) PCCP. One core was recovered intact and the other broke horizontally 5 cm (2 in.) from the top of the core.

On April 14 through 16, 1980, additional Dynaflect testing was performed at numerous locations, and additional cores were taken at mid-slab. The cores were recovered intact.

On February 29, 1980 joint widths at the surface were measured and on the 28 cm (11 in.) and both 33 cm (13 in.) sections were found to be between 1.27 cm and 1.4 cm in average width. They were originally sawn to 1.27 ± .32 cm. On the

FIGURE 1.

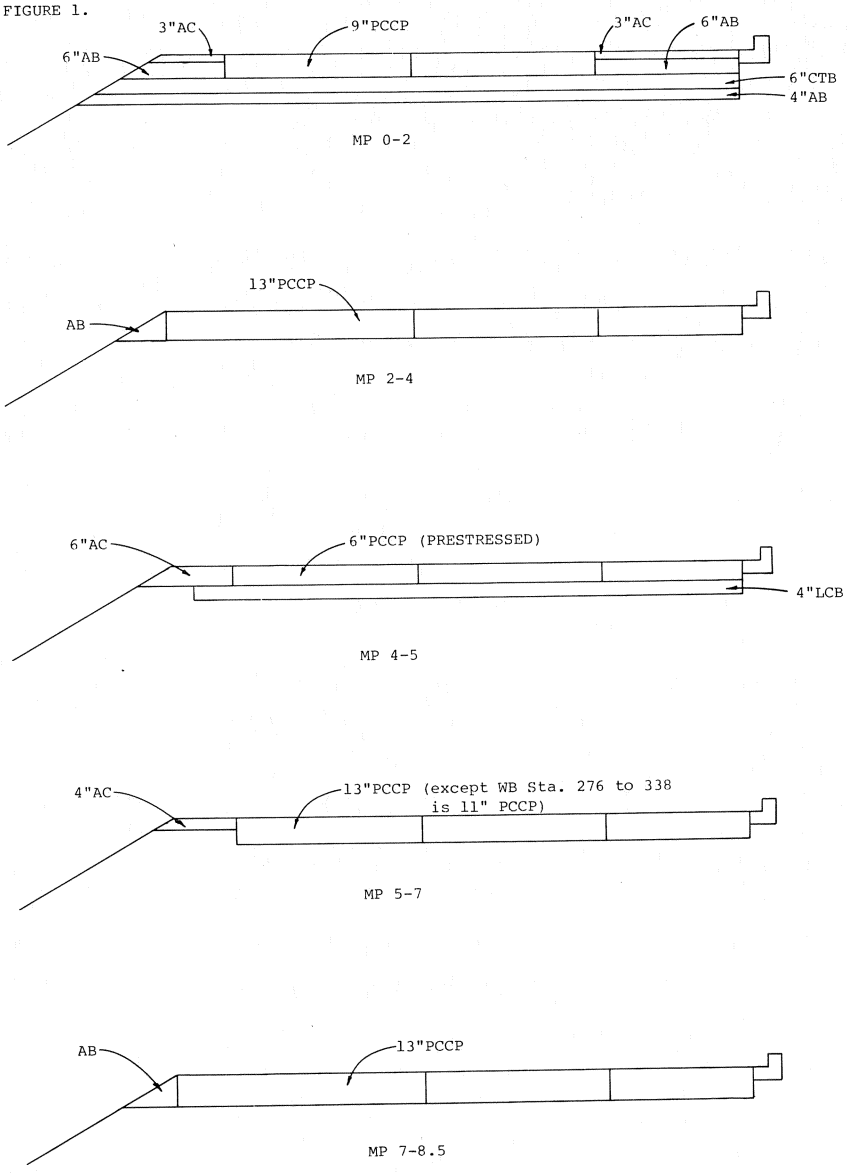


TABLE 1. PAVEMENT STRUCTURE & SUBGRADE PROPERTIES

	STRUCTURE				SUBGRADE PROPERTIES						
							% Passing				
MP	PCC	CTB	AB	SM	R-VALUE	PI	.074mm	.42mm	2.4mm	6.35mm	19mm
0-2 (1972)	23cm(9")	15cm(6")	10cm(4")	0	-	36	71	84	90	92	97
2-4 (1975)	33cm(13")	0	0	0	-	28	71	92	99	100	
4-5 (1977)	15cm(6")*	10cm(4")**	0	0	12	18	75	94	99	100	
5-7 (1979)	33cm(13")	0	0	0	14	21	73	93	98	100	

*Prestressed, post-tensioned

**Lean concrete base

23 cm (9 in.) section the joint width at the surface of those joints with good load transfer was .5 to .64 cm. On one joint in the 23 cm (9 in.) section that had poor load transfer, the surface joint width was 0.89 cm. The joints were originally sawn to a 0.4762 cm nominal width.

Results

Mays Ride Meter

The results of Mays-meter testing show a trend of increasing roughness with increasing time and traffic. (See Figures 2, 3, 4 and 5) The 1975 33 cm (13 in.) section is measurably smoother than the conventional section and is increasing in roughness at a somewhat lower rate. The 1972 conventional 23 cm (9 in.) section, and all the full depth sections are considered to have good rideability. Unexpectedly, the 1979 33 cm (13 in.) section had a significantly higher initial roughness than the 1975 33 cm (13 in.) section. The 1977 pre-stressed section has exceptionally high roughness as well as wide variations in its roughness. This roughness occurs between the joints and is generally attributed to the repeated equipment breakdowns that occurred during construction. The wide variation is unexplained.

A review of Figures 2 and 3 indicates that within the general trend of increasing roughness there are significant short term changes of roughness. In some cases the relative roughness reverses with the smoother section becoming the rougher while the rougher section becomes the smoother. In other cases the relative standing remains the same even though the trend on many sections reverses. The short term changes in roughness are believed to be due primarily to temperature and moisture changes which result in curling and warping of the slabs and in "locking" and "unlocking" of the joints. The long term changes are believed to be a reflection of the serviceability loss due to time and traffic.

The 1979 33 cm (13 in.) sections' fairly high roughness values were somewhat surprising, particularly between milepost 5 and 6, which was not only the roughest full depth section, but which also became even rougher during a time period when the other full depth sections, and 3 of the 4 conventional sections, became smoother.

Two of the possible reasons for the relatively high roughness of the 1979 33 cm (13 in.) section are: first, a systematic change in the measuring system (Mays-meter) which results in higher roughness. A second possibility is the more effective texture (3) specification used on the 1979 33 cm (13 in.) section has produced a higher "background" roughness. The 1975 section had a longitudinal nylon broom texture, much of which has worn off, whereas the 1979 section has a transverse wire tine texture.

Faulting and Pumping

Joint faulting was first noticed in early 1979 on the west end of the 1972 conventional section. This is the oldest, and hence, most heavily traffic-

FIGURE 2. MAYS ROUGHNESS HISTORY S.R. 360 EASTBOUND

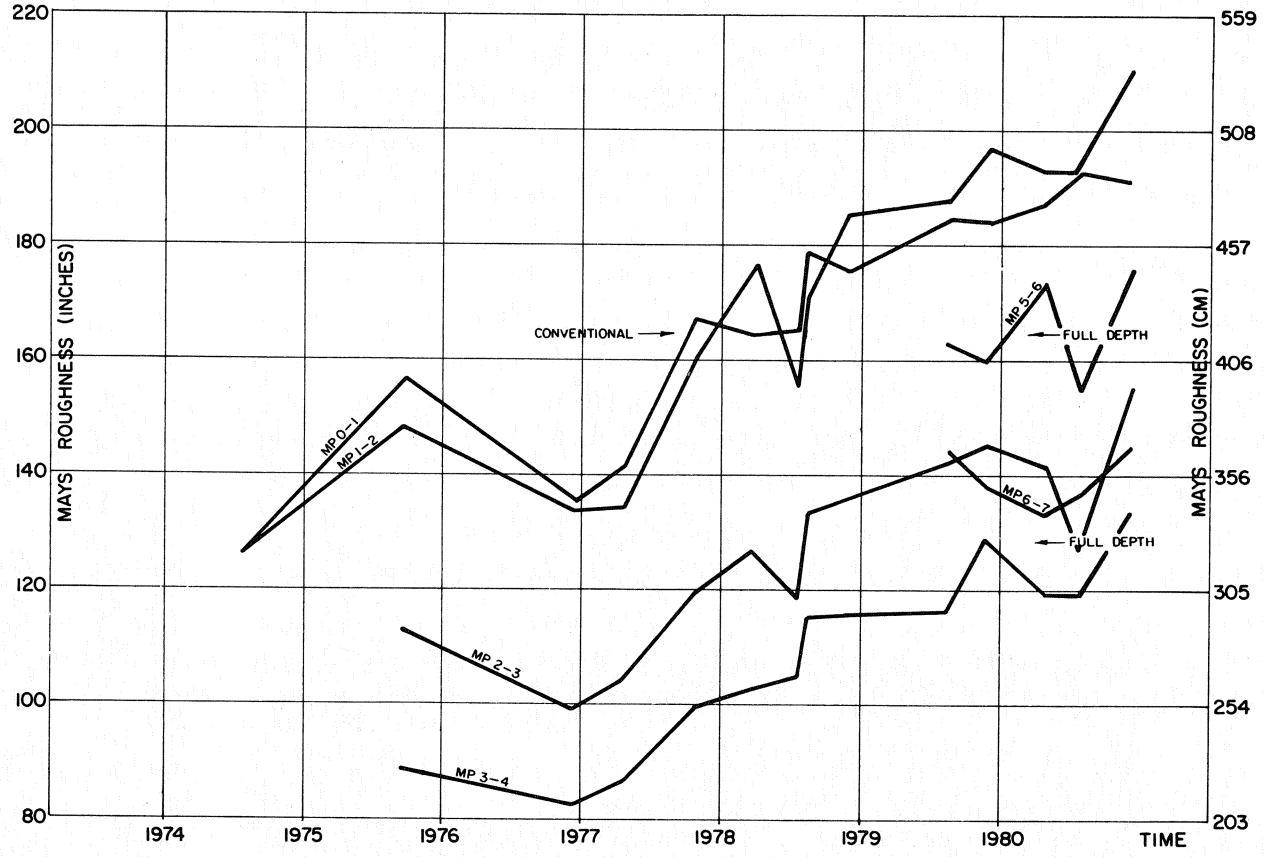


FIGURE 3. MAYS ROUGHNESS HISTORY S.R. 360 WESTBOUND

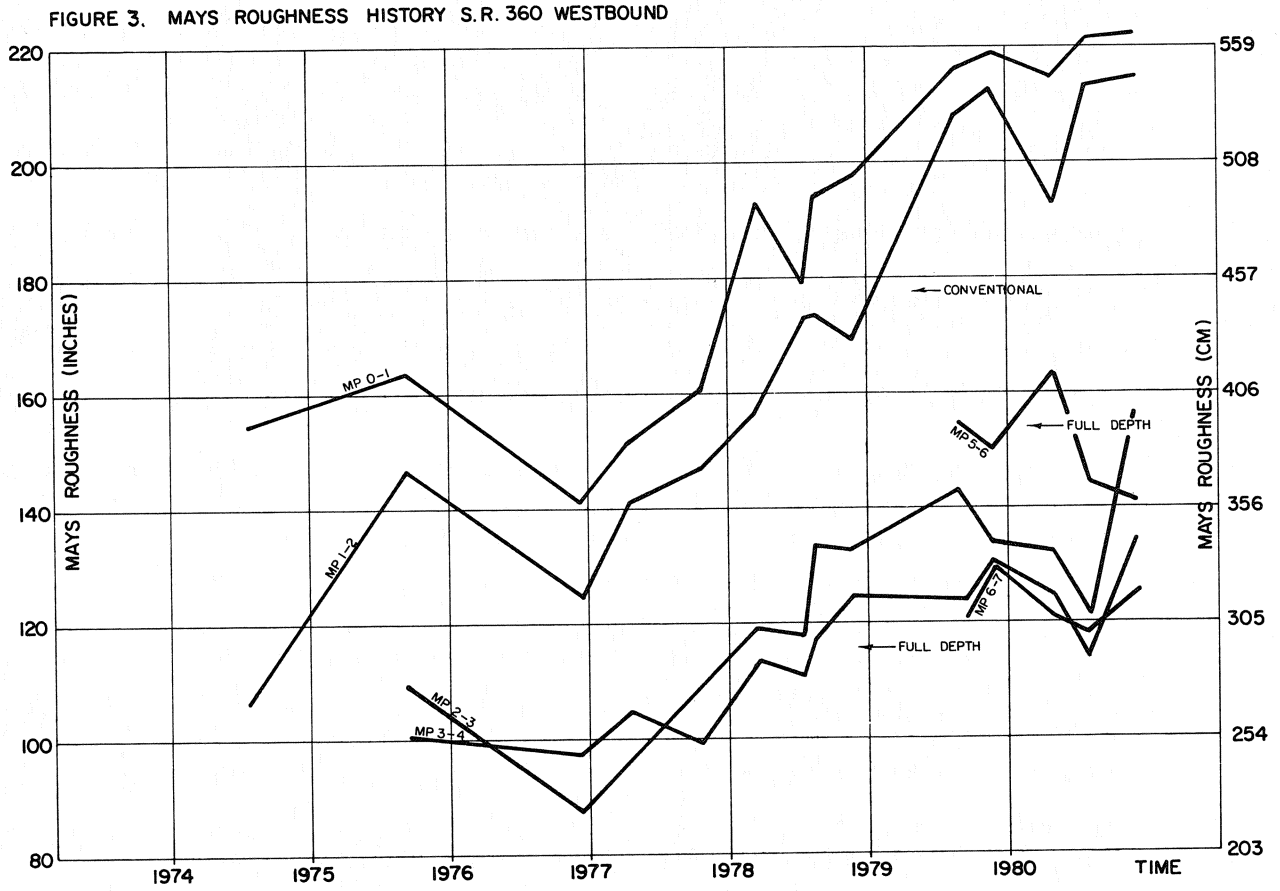


FIGURE 4. MAYS ROUGHNESS HISTORY S.R. 360 EASTBOUND

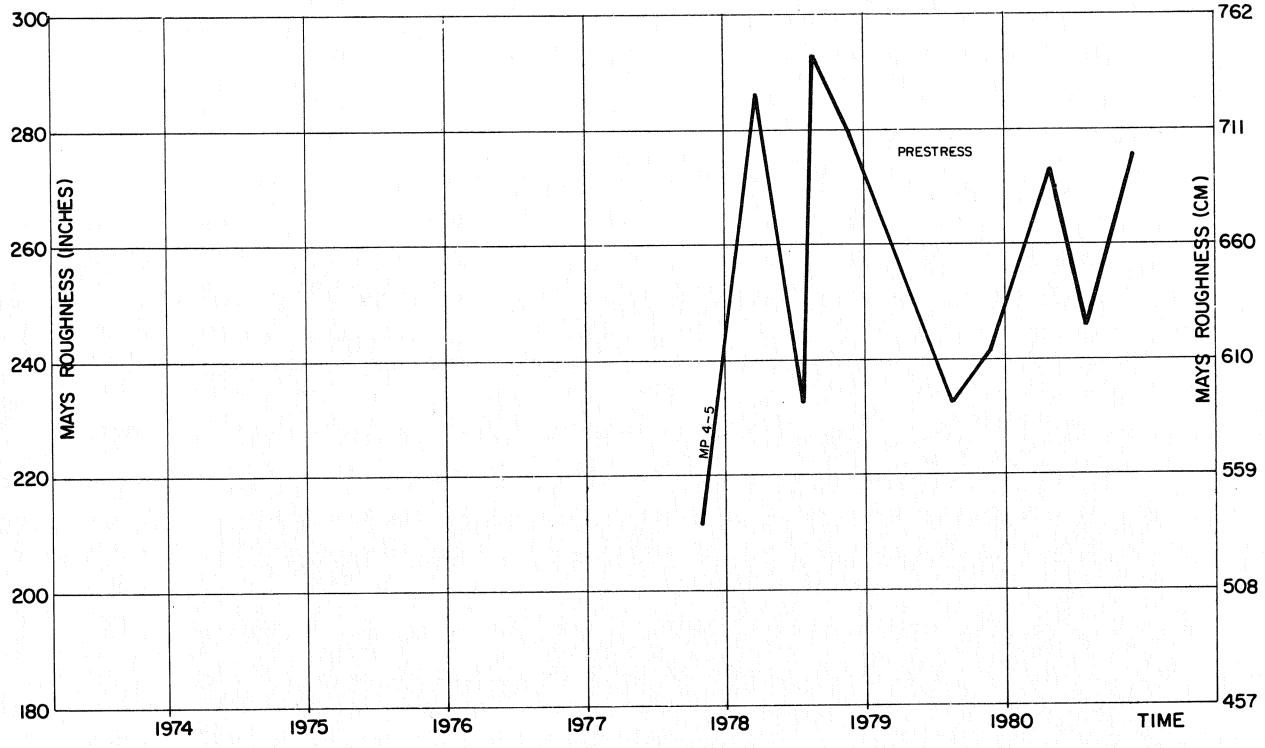
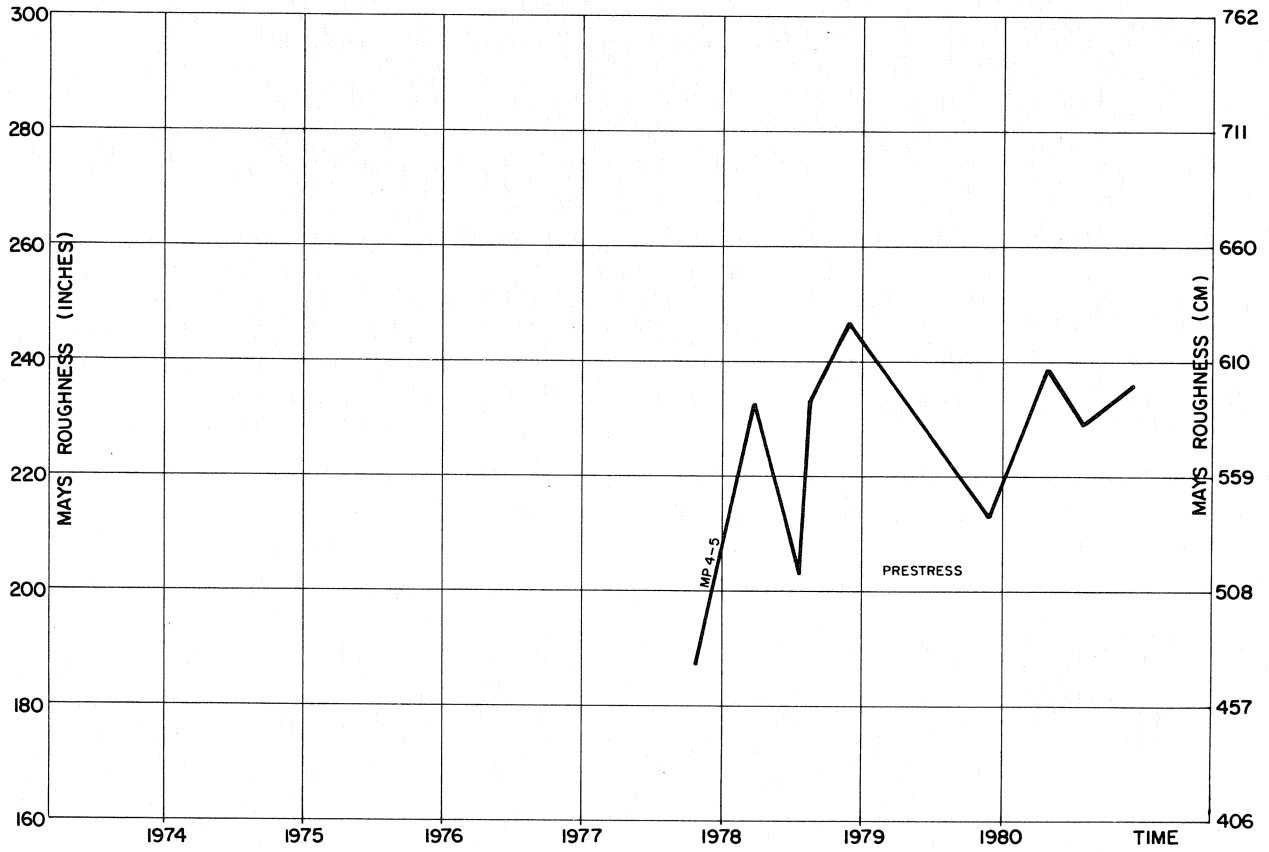


FIGURE 5. MAYS ROUGHNESS HISTORY WESTBOUND S.R. 360



ed area. No faulting has been found in any other areas. No pumping has been noted in any of the sections.

Joint Deflections

The joint deflection study was undertaken in an attempt to obtain data which could be used to predict future joint performance and to enable comparisons to be made between the joint load transfer characteristics of the various sections.

The results of the deflection tests are summarized in Table 2. The Load Transfer Efficiency (LTE) represents the effectiveness of the joint at transfering deflections from the loaded slab to the non-loaded slab. An analysis of Dynaflect mid-slab deflections suggest a perfect joint would be 88% effective. Testing of tied longitudinal joints on an unopened section of Interstate 10 in Phoenix, Arizona produce a LTE of 78 as well as the lowest average deflection of any joints or mid-slab areas tested both in adjacent areas on I-10 or on the Superstition Freeway. As can be seen from the table the LTE based on the dial gauge readings compares very favorably with the LTE based on the Dynaflect readings.

Dial gauge measurements on February 27, 1980 on the 1979 28 cm (11 in.) and 33 cm (13 in.) PCCP indicated good load transfer across the joint. Because the deflection on these sections were very low, particularly the 33 cm (13 in.) section, it was difficult to "follow" the gauges.

Dial gauge measurements on February 28, 1980 on the 1975 33 cm (13 in.) PCCP indicated poor load transfer across the joints. There was a very definite break in the dial gauge readings as the load crossed the joint. The movement at the joint was visible to the eye. On the 1972 23 cm (9 in.) section there was virtually no break in the readings indicating good load transfer.

Because the differential deflections (the difference between the deflection of the loaded and non-loaded slab) were very low on the 1972 23 cm (9 in.) section it was though that possibly the measurements were at joints that had never cracked thru and consequently were not working joints. To test this theory, dial gauge measurements were made at two locations of uncontrolled cracks. Even at these locations the amount of differential deflection was exceedingly low.

Testing on the 1972 23 cm (9 in.) PCCP also indicated the effect of joint separation on joint deflection and load transfer efficiency. At station 112 the joint width at the surface was 0.89 cm (.35 in.), which is an increase of 0.41 cm over the original sawn width of 0.4762 cm. At this joint the Load Transfer Efficiency was about 15% compared to a LTE of about 80% at those joints that were 0.635 cm wide at the surface, or 0.1524 cm over their original width. As would be expected, there were also a considerable increase in the deflection of the loaded slab at this open joint, the deflection being approximately 50% higher than the average for loaded joints on the 1972 23 cm (9 in.) section.

TABLE 2. DEFLECTION TEST RESULTS

STATION	PCCP SLAB THICKNESS	SURFACE TEMP° C	DIAL GAUGE TESTS (in mm) DEFLECTION	DIFFERENTIAL	LOAD* TRANSFER EFFICIENCY	DYNAFLECT JOINT DEFLECTION (in mm) SENSOR 1	SENSOR 2	LOAD** TRANSFER EFFICIENCY
			- WB -					
102-113	23cm(9")	21	0.0635	0.0127	80	0.0123	0.0086	70
96-113	23cm(9")	47				0.0128	0.0102	80
114-117	33cm(13")	17	0.0762	0.0635	17	0.0151	0.0048	32
161-164	33cm(13")	17				0.0107	0.0059	55
168-192	33cm(13")	35				0.0079	0.0068	87
285-297	28cm(11")	49				0.0062	0.0049	79
322-337	28cm(11")	32	0.0635	0.0127	80	0.0074	0.0062	84
339-354	33cm(13")	27	0.0381	0.0127	67	0.0062	0.0042	67
			- EB -					
115-122	33cm(13")	50				0.0085	0.0072	85
128-146	33cm(13")	52				0.0081	0.0071	88
277-333	33cm(13")	44				0.0059	0.0049	83

*LTE = 100 - $\left(\frac{\text{Differential}}{\text{Deflection}} \times 100\right)$

**LTE = 100 - $\left[\left(\frac{\text{Sensor 1 - Sensor 2}}{\text{Sensor 1}}\right) \times 100\right]$

NOTE: Analysis of the dynaflect center of slab deflections indicate a LTE of 88 is equivalent to a 100% interlocked joint.

Ultimate Strength & Modulus

The mid-slab cores were tested for modulus and ultimate strength. The average values for the sections were:

		Modulus		Ultimate Strength	
		$\bar{x}$	s	$\bar{x}$	s
1975	33 cm (13 in.)	14483 MPa	2965 MPa	39.6 MPa	5.76 MPa
1979	33 cm (13 in.)	17241 MPa	1310 MPa	46.3 MPa	4.81 MPa

Note: 1 MPa = 145 psi

These modulus values were lower then expected. Typically, expected values would range between 14000 MPa to 42000 MPa at 28 days (4). Since the age of the cores tested was 1 to 4 years a value of over 20000 MPa was expected.

Discussion

The results of the concrete coring and subsequent laboratory testing for strength and modulus indicate that the concrete in the 1975 33 cm (13 in.) section is more variable and of lower quality than that of the 1979 33 cm (13 in.) section. This lower quality became apparent during the coring operations as it was difficult to recover the cores intact. In the worst areas, the concrete appeared to consist of 3 layers. Possibly, this was a result of poor vibration during construction.

A comparison of the results of the dynaflect testing at different dates, and various pavement surface temperatures, indicate that the degree of aggregate interlock varied widely. The most likely explanation is that the variation in interlock is directly related to changes in the temperature. Higher temperatures would cause slab expansion and cause the joints to interlock. Lower temperatures cause contraction and loss of interlock. If the slab expansion results in movement of some of the slabs then pavement "growth" may occur and aggregate interlock will be lost at progressively higher temperatures. If the joints are kept sealed to prevent debris from entering the "growth" should be minimized. The data available in this study suggest that aggregate interlock is being lost below about 16 degrees C on the 1975 33 cm (13 in.) section.

A related phenomenon to "growth", that of downhill creep, has been suggested as a mechanism that would cause a loss of interlock at the top of grades. A review of the LTE at the individual joint tests does not support this theory. Surface measurement of the saw joint width indicated very little if any joint separation.

A comparison of the dynaflect test results for the various sections shows the influence of both aggregate interlock and slab thickness. For interlocked joints, increased thickness resulted in re-

duced deflections. The results also suggest that higher modulus in the 1979 33 cm (13 in.) section yields lower deflections.

After 5 years of traffic, the 1975 33 cm (13 in.) section is not faulted; however, the joint deflection study indicate that differential deflection that might lead to faulting is occurring during the colder months of the year. It remains to be seen if the magnitude of this differential deflection is sufficient to lead to the build-up of material that will result in faulting.

Past experience with thinner pavements would indicate that faulting will occur; however, there is a significant difference in this pavement structure which may preclude faulting. The typical faulted pavement has loose material consisting of silt, sand, and fine gravel trapped under the faulted slab and above either a dense graded aggregate base or a stabilized base layer. During periods when water is present this loose material is pumped laterally toward the approach slab from under the leave slab since the water velocity is higher in that direction. (It might be possible that fine material could be pumped by air without the presence of water, but this has not been demonstrated). The result is a net increase in the quantity of material under the approach slab which raises the slab and causes faulting or step-off. The thick PCCP on subgrade has, at least in the areas we have used this design, virtually no source of particles larger than 2.4 mm (#8 sieve), combined with a large amount of material finer then .074 mm (minus #200's). During periods when water is present and movement of loose material could occur the only material available would be a clay, silt, sand slurry. Even if differential joint deflections and longitudinal pumping occurs it is conceivable that faulting will not occur because the slurry could behave as a viscous fluid and not remain under the approach slab but flow back under both the approach slab and the leave slab. Vertical pumping could possibly occur but none has been noted to date on any of our sections.

The 1979 33 cm (13 in.) section had good joint performance at the temperatures encountered in this study, however, after a few more yearly temperatures cycles it may begin to display the same loss of interlock as the 1975 33 cm (13 in.) section.

The 1972 23 cm (9 in.) section, although faulting in some areas, is performing satisfactorily, but does have fairly high deflections (compared to the 1975 33 cm (13 in.) section when interlocked, and to the 1979 33 cm (13 in.) section).

The full depth sections have reduced deflections but, at least on the 1975 project, considerable differential deflection is occurring during the cool months. The loss of interlock and resultant moderate total and high differential deflections during the cold months is likely to be detrimental. The almost parallel increase in roughness for the 1972 conventional and 1975 full depth section does not indicate a very large reduction in the rate of serviceability loss. Although it may be that the 1975 full depth section is not typical, the 1979 full depth section has not been in service long enough to judge whether its performance will differ significantly from that of the 1975 section. It has a better predicted performance at this time since it has excellent load transfer and low deflections at the joints.

When comparing the results of the 1972 23 cm (9 in.) conventional pavement and the 1975 33 cm (13 in.) section one might be tempted to conclude that the CTB is assisting the load transfer since at roughly the same temperatures (21 for the 23 cm (9 in.) and 16 for the 33 cm (13 in.) the 23 cm (9 in.) section maintained good load transfer whereas the 33 cm (13 in.) section did not. As was pointed out earlier, at one joint in the 23 cm (9 in.) section that has widened appreciably the load transfer did drop drastically. It is likely that had this testing been done at lower temperatures a number of the joints on the 23 cm (9 in.) section would have opened sufficiently to lose interlock. The same might be true of the 1979 33 cm (13 in.) section, that is, at lower temperatures than were encountered during this study, aggregate interlock might have been lost.

Conclusions

1) Dynaflect deflection measurements can be used in lieu of direct dial gauge measurements to evaluate the load transfer characteristics of joints on PCCP.

2) Loss of aggregate interlock at the joints of thick (33 cm (13 in.)) PCCP placed on subgrade results in moderately high joint deflections and high differential joint deflections.

3) When aggregate interlock is present at the joints, 28 and 33 cm (11 in. and 13 in.) concrete pavements on subgrade have lower total deflections then conventional pavements, i.e., 23 cm (9 in.) concrete slab on 15 cm (6 in.) cement treated base and 10 cm (4 in.) aggregate base.

4) A 33 cm (13 in.) concrete pavement (on subgrade) that does not have joint interlock, has lower total deflections then a conventional 23 cm (9 in.) concrete pavement (with 15 cm cement-treated base and 10 cm aggregate base) that does have joint interlock.

5) Other factors being equal, 33 cm (13 in.) sections have measureably lower deflections than 28 cm (11 in.) sections.

6. A 23 cm (9 in.) slab on cement-treated base provides sufficient joint surface area to ensure load transfer when the joint remains interlocked.

References

1. D. L. Spelman, J. H. Wodstrom & B. F. Neal, "Faulting of Portland Cement Concrete Pavements", Highway Research Record 407, 1972.
2. G. Gluais & J. L. Nissoux, "Thick Concrete Slabs Without Foundation", 1978. (Translated by John L. Walkinshaw, 1979).
3. S. M. Law & M. Rasoulian, "Texturing of Concrete Pavements", Louisiana Department of Transportation, FHWA/LA-79/134, 1979.
4. F. S. Merritt, Editor, "Standard Handbook for Civil Engineers", McGraw-Hill, 1968.