CHAPTER 3

RIGID PAVEMENT

U.S. Department of Transportation
Federal Highway Administration

Technical Advisory

Subject

CONCRETE PAVEMENT JOINTS

Classification Code	Date
T 5040.30	November 30, 1990

1. PURPOSE. To provide guidance and recommendations relating to the design and construction of joints in jointed portland cement concrete pavements.

2. CANCELLATION. Technical Advisory T 5140.18, Rigid Pavement Joints, dated December 15, 1980, is canceled.

3. BACKGROUND

a. The performance of concrete pavements depends to a large extent upon the satisfactory performance of the joints. Most jointed concrete pavement failures can be attributed to failures at the joint, as opposed to inadequate structural capacity. Distresses that may result from joint failure include faulting, pumping, spalling, corner breaks, blow-ups, and mid-panel cracking. Characteristics that contribute to satisfactory joint performance, such as adequate load transfer and proper concrete consolidation, have been identified through research and field experience. The incorporation of these characteristics into the design, construction, and maintenance of concrete pavements should result in joints capable of performing satisfactorily over the life of the pavement. Regardless of the joint sealant material used, periodic resealing will be required to ensure satisfactory joint performance throughout the life of the pavement. Satisfactory joint performance also depends on appropriate pavement design standards, quality construction materials, and good construction and maintenance procedures.

DISTRIBUTION

OPI HNG-42

b. The most common types of pavement joints, which are defined by their function, are as follows:

(1) Transverse Contraction Joint - a sawed, formed, or tooled groove in a concrete slab that creates a weakened vertical plane. It regulates the location of the cracking caused by dimensional changes in the slab, and is by far the most common type of joint in concrete pavements.

(2) Longitudinal Joint - a joint between two slabs which allows slab warping without appreciable separation or cracking of the slabs.

(3) Construction Joint - a joint between slabs that results when concrete is placed at different times. This type of joint can be further broken down into transverse and longitudinal joints.

(4) Expansion Joint - a joint placed at a specific location to allow the pavement to expand without damaging adjacent structures or the pavement itself.

4. TRANSVERSE CONTRACTION JOINTS. The primary purpose of transverse contraction joints is to control the cracking that results from the tensile and bending stresses in concrete slabs caused by the cement hydration process, traffic loadings, and the environment. Because these joints are so numerous, their performance significantly impacts pavement performance. A distressed joint typically exhibits faulting and/or spalling. Poor joint performance frequently leads to further distresses such as corner breaks, blow-ups, and mid-panel cracks. Such cracks may themselves begin to function as joints and develop similar distresses. The performance of transverse contraction joints is related to three major factors:

a. Joint Spacing. Joint spacing varies throughout the country because of considerations of initial costs, type of slab (reinforced or plain), type of load transfer, and local conditions. Design considerations should include: the effect of longitudinal slab movement on sealant and load transfer performance; the maximum slab length which will not develop transverse cracks in a plain concrete pavement; the amount of cracking which can be tolerated in a jointed reinforced concrete pavement; and the use of random joint spacings.

(1) The amount of longitudinal slab movement that a joint experiences is primarily a function of joint spacing and temperature changes. Expansion characteristics of the aggregates used in the concrete and the friction between the bottom of the slab and the base also have an effect on slab movement.

(a) Joint movement can be estimated by the following equation:

$$\Delta L = CL(\alpha \Delta T + \epsilon)$$

where:

ΔL = the expected change in slab length, in inches.
C = the base/slab frictional restraint factor (0.65 for stabilized bases, 0.8 for granular bases).
L = the slab length, in inches.
α = the PCC coefficient of thermal expansion (see Table 1 for typical values).
ΔT = the maximum temperature range (generally the temperature of the concrete at the time of placement minus the average daily minimum temperature in January, in °F).
ϵ = the shrinkage coefficient of concrete (see Table 2 for typical values). This factor should be omitted on rehabilitation projects, as shrinkage is no longer a factor.

TABLE 1. TYPICAL VALUES FOR PCC COEFFICIENT OF THERMAL EXPANSION (α) [1]

Type of Coarse Aggregate	PCC Coeff. of Thermal Expansion (10^{-6}/°F)
Quartz	6.6
Sandstone	6.5
Gravel	6.0
Granite	5.3
Basalt	4.8
Limestone	3.8

(b) While the above equation can be used to estimate anticipated joint movements, it may be worthwhile to physically measure joint movements in existing pavements. These measurements could provide the designer with more realistic design inputs.

TABLE 2. TYPICAL VALUES FOR PCC COEFFICIENT OF SHRINKAGE (ϵ) [1]

Indirect Tensile Strength (psi)	PCC Coeff. of Shrinkage (in./in.)
300 (or less)	0.0008
400	0.0006
500	0.00045
600	0.0003
700 (or greater)	0.0002

(2) For plain concrete slabs, a maximum joint spacing of 15 feet is recommended. Longer slabs frequently develop transverse cracks. It is recognized that in certain areas, joint spacings greater than 15 feet have performed satisfactorily. The importance of taking local experience into account when selecting joint spacing (and designing pavements in general) cannot be overstated. Studies have shown that pavement thickness, base stiffness, and climate also affect the maximum anticipated joint spacing beyond which transverse cracking can be expected. Research indicates that there is a general relationship between the ratio of slab length (L) to the radius of relative stiffness (ℓ) and the amount of transverse cracking [2]. This research shows that there is an increase in transverse cracking when the ratio L/ℓ exceeds 5.0. Further discussion is provided in Attachment 1.

(3) For reinforced concrete slabs, a maximum joint spacing of 30 feet is recommended. Longer slab lengths have a greater tendency to develop working mid-panel cracks caused by the rupture of the steel reinforcement. Studies have also shown that, as the joint spacing increases above 30 feet, the rate of faulting increases and joint sealant performance decreases [4].

(4) Random joint spacings have been successfully used in plain undoweled pavements to minimize resonant vehicle responses. When using random joint spacings, the longest slab should be no greater than 15 feet, to reduce the potential for transverse cracking. Some States are successfully using a spacing of 12′-15′-13′-14′. Large differences in slab lengths should be avoided.

(5) While they do not affect joint spacing, skewed joints have been used in plain pavements to provide a smoother ride. A skew of 2 feet in 12 feet is recommended, with the skew placed so that the inside wheel crosses the joint ahead of the outside wheel. Only one wheel crosses the joint at a time, which minimizes vehicle response and decreases stresses within the slab. Skewed joints are most commonly used when load transfer devices are not present. While skewed joints may be used in conjunction with load transfer devices, studies have not substantiated that skewing doweled joints improves pavement performance and are not recommended. Dowels in skewed joints must be placed parallel to the roadway and not perpendicular to the joints.

b. Load Transfer across the joint. Loads applied by traffic must be effectively transferred from one slab to the next in order to minimize vertical deflections at the joint. Reduced deflections decrease the potential for pumping of the base/subbase material and faulting. The two principal methods used to develop load transfer across a joint are: aggregate interlock; and load transfer devices, such as dowel bars. It is recommended that dowel bars be used.

(1) Aggregate Interlock. Aggregate interlock is achieved through shearing friction at the irregular faces of the crack that forms beneath the saw cut. Climate, and aggregate hardness have an impact on load transfer efficiency. It can be improved by using aggregate that is large, angular, and durable. Stabilized bases have also been shown to improve load transfer efficiency [14]. However, the efficiency of aggregate interlock decreases rapidly with increased crack width and the frequent application of heavy loads to the point that pavement performance may be effected. Therefore, it is recommended that aggregate interlock for load transfer be considered only on local roads and streets which carry a low volume of heavy trucks.

(2) Dowel Bars. Dowel bars should be used on all routes carrying more than a low volume of heavy trucks. The purpose of dowels is to transfer loads across a joint without restricting joint movement due to thermal contraction and expansion of the concrete. Studies have shown that larger dowels are more effective in transferring

loads and in reducing faulting. It is recommended that the minimum dowel diameter be D/8, where D is the thickness of the pavement. However, the dowel diameter should not be less than 1¼ inches. It is also recommended that 18-inch long dowels be used at 12-inch spacings. Dowels should be placed mid-depth in the slab. Dowels should be corrosion-resistant to prevent dowel seizure, which causes the joint to lock up. Epoxy-coated and stainless steel dowels have been shown to adequately prevent corrosion.

c. Joint Shape and Sealant Properties

(1) The purpose of a joint sealant is to deter the entry of water and incompressible material into the joint and the pavement structure. It is recognized that it is not possible to construct and maintain a watertight joint. However, the sealant should be capable of minimizing the amount of water that enters the pavement structure, thus reducing moisture-related distresses such as pumping and faulting. Incompressibles should be kept out of the joint. These incompressibles prevent the joint from closing normally during slab expansion and lead to spalling and blow-ups.

(2) Sealant behavior has a significant influence on joint performance. High-type sealant materials, such as silicone and preformed compression seals, are recommended for sealing all contraction, longitudinal, and construction joints. While these materials are more expensive, they provide a better seal and a longer service life. Careful attention should be given to the manufacturer's recommended installation procedures. Joint preparation and sealant installation are very important to the successful performance of the joint. It is therefore strongly recommended that particular attention be given to both the construction of the joint and installation of the sealant material.

(3) When using silicone sealants, a minimum shape factor (ratio of sealant depth to width) of 1:2 is recommended. The maximum shape factor should not exceed 1:1. For best results, the minimum width of the sealant should be 3/8-inch. The surface of the sealant should be recessed 1/4- to 3/8-inch below the pavement surface to prevent abrasion caused by traffic. The use of a backer rod is necessary to provide the proper shape factor and to prevent the sealant from bonding to the bottom of the joint reservoir. This backer rod should be a closed-cell polyurethane foam rod having a diameter approximately 25 percent greater than the width of the joint to ensure a tight fit.

(4) When using preformed compression seals, the joint should be designed so that the seal will be in 20 to 50 percent compression at all times. The surface of the seal should be recessed 1/8- to 3/8-inch to protect it from traffic. Additional information can be obtained from FHWA Technical Paper 89-04, "Preformed Compression Seals" [5] for PCC pavement joints."

5. LONGITUDINAL JOINTS

a. Longitudinal joints are used to relieve warping stresses and are generally needed when slab widths exceed 15 feet. Widths up to and including 15 feet have performed satisfactorily without a longitudinal joint, although there is the possibility of some longitudinal cracking. Longitudinal joints should coincide with pavement lane lines whenever possible, to improve traffic operations. The paint stripe on widened lanes should be at 12 feet and the use of a rumble strip on the widened section is recommended.

b. Load transfer at longitudinal joints is achieved through aggregate interlock. Longitudinal joints should be tied with tiebars to prevent lane separation and/or faulting. The tiebars should be mechanically inserted and placed at mid-depth. When using Grade 40 steel, 5/8-inch by 30-inch or 1/2-inch by 24-inch tiebars should be used. When using Grade 60 steel, 5/8-inch by 40-inch or 1/2-inch by 32-inch tiebars should be used. These lengths are necessary to develop the allowable working strength of the tiebar. Tiebar spacing will vary with the thickness of the pavement and the distance from the joint to the nearest free edge. Recommended tiebar spacings are provided in Table 3.

c. Tiebars should not be placed within 15 inches of transverse joints. When using tiebars longer than 32 inches with skewed joints, tiebars should not be placed within 18 inches of the transverse joints.

d. The use of corrosion-resistant tiebars is recommended, as corrosion can reduce the structural adequacy of tiebars.

e. It is recommended that longitudinal joints be sawed and sealed to deter the infiltration of surface water into the pavement structure. A 3/8-inch wide by 1-inch deep sealant reservoir should be sufficient.

TABLE 3. MAXIMUM RECOMMENDED TIEBAR SPACINGS (In.)

Note : 48" maximum spacing recommended.

BAR SIZE		# 4 BAR										# 5 BAR									
GRADE STEEL		GRADE 40					GRADE 60					GRADE 40					GRADE 60				
DIST TO FREE EDGE (ft.)		10	12	16	22	24	10	12	16	22	24	10	12	16	22	24	10	12	16	22	24
PVMT THICKNESS	**TYPE OF JOINT**																				
9"	Warp	37	31	23	17	16	48	47	35	25	23	48	48	36	26	24	48	48	48	40	36
	Butt	26	22	16	12	11	40	34	25	18	16	42	35	26	19	17	48	48	39	29	26
10"	Warp	34	28	22	16	14	48	42	32	23	20	48	44	33	24	22	48	48	48	36	32
	Butt	24	20	16	11	10	36	30	23	16	14	38	31	24	17	16	48	47	35	26	23
11"	Warp	31	25	20	15	13	47	38	29	21	19	48	40	30	22	20	48	48	44	32	30
	Butt	22	18	14	11	9	34	27	21	15	14	34	29	21	16	14	48	43	31	23	21
12"	Warp	28	23	18	13	12	42	35	27	19	18	44	36	28	20	18	48	48	41	30	28
	Butt	20	16	13	9	9	30	25	19	14	13	31	26	20	14	13	47	39	29	21	20

Warp joint: a sawed or construction joint with a keyway

Butt joint: a construction joint with no keyway

6. CONSTRUCTION JOINTS

a. Transverse Construction Joints

(1) Transverse construction joints should normally replace a planned contraction joint. However, they should not be skewed, as satisfactory concrete placement and consolidation are difficult to obtain. Transverse construction joints should be doweled as described in paragraph 4b(2) and butted, as opposed to keyed. Keyed transverse joints tend to spall and are not recommended.

(2) It is recommended that transverse construction joints be sawed and sealed. The reservoir dimensions should be the same as those used for the transverse contraction joints.

b. Longitudinal Construction Joints

(1) The decision to use keyed longitudinal construction joints should be given careful consideration. The top of the slab above the keyway frequently fails in shear. For this reason, it is recommended that keyways not be used when the pavement thickness is less than 10 inches. In these cases, the tiebars should be designed to carry the load transfer.

(2) When the pavement thickness is 10 inches or more, a keyway may be used to provide the necessary load transfer. If a keyway is to be used, the recommended dimensions are shown in Figure 1. Keyways larger than the one shown may reduce the concrete shear strength at the joint and result in joint failures. The keyway should be located at mid-depth of the slab to ensure maximum strength. Tiebars are necessary when using keyways. Consideration should be given to deleting the keyway and increasing the size and/or number of tiebars. The additional steel cost may be more than offset by the potential savings in initial labor and future maintenance costs.

(3) Tiebars should not be placed within 15 inches of transverse joints. When using tiebars longer than 32 inches with skewed joints, tiebars should not be placed within 18 inches of the transverse joints.

(4) It is essential that the tiebars be firmly anchored in the concrete. Tiebars should be either mechanically inserted into the plastic concrete or installed as a two-part threaded tiebar and splice coupler system. It is recommended that periodic pull-out tests be conducted to ensure the tiebars are securely anchored in the concrete. Attachment 2 describes a recommended testing procedure for tiebars.

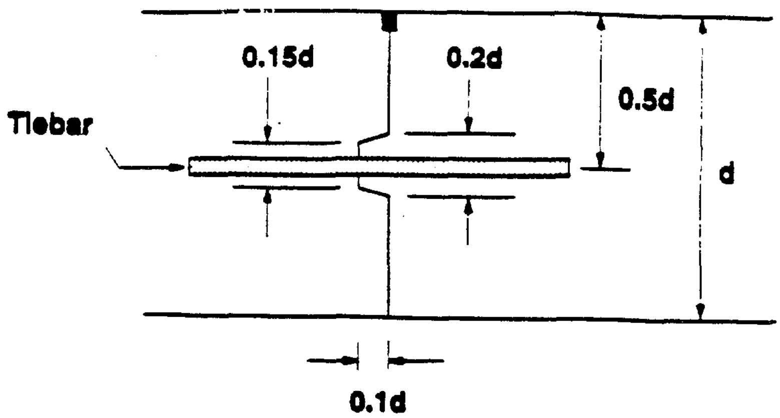


Figure 1. Recommended Keyway Dimensions

(5) Bending of tiebars is not encouraged. Where bending of the tiebars would be necessary, it is recommended that a two-part threaded tiebar and splice coupler system be used in lieu of tiebars. If tiebars must be bent and later straightened during construction, Grade 40 steel should be used, as it better tolerates the bending. It may be necessary to reapply a corrosion-resistant coating to the tiebars after they have been straightened. When pull-out tests are performed, they should be conducted after the tiebars have been straightened.

(6) It is recommended that longitudinal construction joints be sawed and sealed. The reservoir dimensions should be the same as those used for the longitudinal joints.

7. <u>EXPANSION JOINTS</u>

a. Good design and maintenance of contraction joints have virtually eliminated the need for expansion joints, except at fixed objects such as structures. When expansion joints are used, the pavement moves to close the unrestrained expansion joint over a period of a few years. As this happens, several of the adjoining contraction joints may open, effectively destroying their seals and aggregate interlock.

b. The width of an expansion joint is typically 3/4-inch or more. Filler material is commonly placed 3/4- to 1-inch below the slab surface to allow space for sealing material. Smooth dowels are the most widely used method of transferring load across expansion joints. Expansion joint dowels are specially fabricated with a cap on one end of each dowel that creates a void in the slab to accommodate the dowel as the adjacent slab closes the expansion joint, as shown in Figure 2.

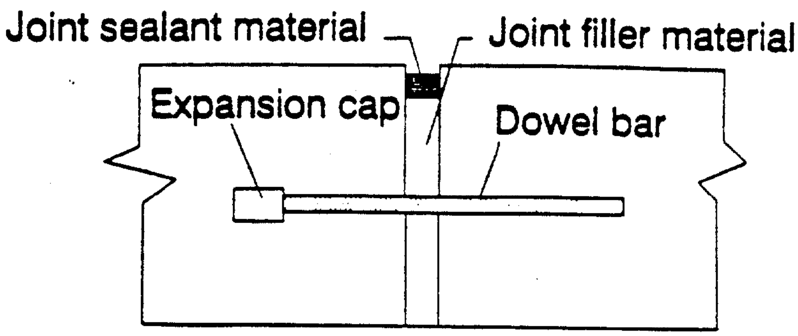


Figure 2. Expansion Joint Detail

c. Pressure relief joints are intended to serve the same purpose as expansion joints, except that they are installed after initial construction to relieve pressure against structures and to alleviate potential pavement blowups. Pressure relief joints are not recommended for routine installations. However, they may be appropriate to relieve imminent structure damage or under conditions where excessive compressive stresses exist. Additional information can be obtained from the FHWA Pavement Rehabilitation Manual, Chapter 9.

8. JOINT CONSTRUCTION

a. Concrete Placement

(1) A prepaving conference should be considered on all major paving projects. This conference should include the project engineer and the paving contractor and should discuss methods for accomplishing all phases of the paving operation. The need for attention to detail cannot be overstated.

(2) When using dowel baskets, the baskets should be checked prior to placing the concrete to ensure that the dowels are properly aligned and that the dowel basket is securely anchored in the base. It is recommended that dowel baskets be secured to the base with steel stakes having a minimum diameter of 0.3-inch. These stakes should be embedded into the base a minimum depth of 4 inches for stabilized dense bases, 6 inches for treated permeable bases, and 10 inches for untreated permeable bases, aggregate bases, or natural subgrade. A minimum of 8 stakes per basket is recommended. All temporary spacer wires extending across the joint should be removed from the basket. Securing the steel stakes to

the top of the dowel basket, as opposed to the bottom, should stabilize the dowel basket once these spacer wires are removed.

(3) Dowels should be lightly coated with grease or other substance over their entire length to prevent bonding of the dowel to the concrete. This coating may be eliminated in the vicinity of the welded end if the dowel is to be coated prior to being welded to the basket. The traditional practice of coating only one-half of the dowel has frequently resulted in problems, primarily caused by insufficient greasing and/or dowel misalignment. The dowel must be free to slide in the concrete so that the two pavement slabs move independently, thus preventing excessive pavement stresses. Only a thin coating should be used, as a thick coating may result in large voids in the concrete around the dowels.

(4) The placement of concrete at construction joints is particularly critical. Therefore, care must be taken to ensure that only quality concrete is used in their construction; i.e., do not use the first concrete down the chute, nor the "roll" from the screed to construct this type of joint. The concrete used to construct these joints should be the same as for the remainder of the slab. The practice of modifying the mix at the joints is not recommended.

(5) Careful and sufficient consolidation of the concrete in the area of the joints is essential to good joint performance. Load transfer across a doweled joint is greatly affected by the quality of concrete consolidation around the dowels. Consolidation also has a direct relationship to concrete strength and durability. Concrete strength, in turn, has a significant effect on the amount of spalling that occurs at the joint.

(6) The placement of dowels should be carefully verified soon after paving begins. If specified tolerances are not being achieved, then an evaluation of the dowel installation, concrete mix design, and placement techniques must be made. Appropriate corrections should be made to the paving process to ensure proper alignment of the load transfer devices.

(7) When paving full-depth full-width, a mechanical prespreader and finishing machine in the paving train can be used to reduce drag and shear forces on the dowels.

(8) In cases where separate concrete placement is made adjacent to previously placed concrete, i.e., truck climbing lanes or concrete shoulders being placed after mainline pavement, it

is important that incompressibles do not enter the previously sawed transverse joint reservoir or crack that typically forms below the transverse joint reservoir. It is recommended that backer rod, tape, or other material be placed on the vertical face of the transverse joint at the edge of the pavement to prevent mortar from intruding into the existing joint. Failure to keep incompressibles out could prevent the joint from closing normally during slab expansion and may lead to delaminations near the edge of the previously placed concrete.

b. Sawing

(1) It is recommended that all joints be sawed. The sawing of transverse contraction and longitudinal joints should be a two-phase operation. The initial sawing is intended to cause the pavement to crack at the intended joint. It should be made to the required depth, as described later, with a 1/8-inch wide blade. The second sawing provides the necessary shape factor for the sealant material. This second sawcut can be made any time prior to the sealant installation. However, the later the sealant reservoir is made, the better the condition of the joint face. Both sawcuts should be periodically checked to ensure proper depth, as saw blades tend to wear, as well as ride up when hard aggregate is encountered. Periodic measurement of blade diameter is an excellent method to monitor random blade wear, particularly when using gang saws.

(2) Time of initial sawing, both in the transverse and longitudinal directions, is critical in preventing uncontrolled shrinkage cracking. It is very important that sawing begin as soon as the concrete is strong enough to both support the sawing equipment and to prevent raveling during the sawing operation. All joints should be sawed within 12 hours of concrete placement. The sawing of concrete constructed on stabilized base must be sawed earlier. This is particularly critical during hot weather. Once sawing begins, it should be a continuous operation and should only be stopped if raveling begins to occur.

(3) For transverse contraction joints, an initial sawcut of D/3 is recommended, particularly for pavements with a thickness greater than 10 inches. In no case should the sawcut depth be less than D/4. Transverse contraction joints should be initially sawed in succession. Skip sawing is not recommended, as this practice results in a wide range of crack widths that form beneath the sawed joints. These varied crack widths affect the shape factors and may cause excessive sealant stresses in those joints initially sawed.

The dimensions of the final sawing should be dependent upon the sealant type and the anticipated longitudinal slab movement.

(4) For longitudinal joints, a minimum initial sawcut depth of D/3 is recommended to ensure cracking at the joint. The maximum sawcut depth should be such that the tiebars are not damaged. A final sawing that provides a 3/8-inch wide by 1-inch deep sealant reservoir should be sufficient.

(5) When a lengthy period is anticipated between the initial sawing of the joint and the final sawing and sealing, consideration should be given to filling the joint with a temporary filler. This filler material should keep incompressibles out of the joint and reduce the potential for spalling.

(6) The use of plastic inserts is not recommended. Although a few States have had success with these inserts, most States no longer allow their use. Improper placement of plastic inserts has been identified as a cause of random longitudinal cracking [2]. It is also very difficult to seal the joint formed by these inserts.

Anthony R. Kane
Associate Administrator for Program Development

Attachments

DESIGN OF LENGTH

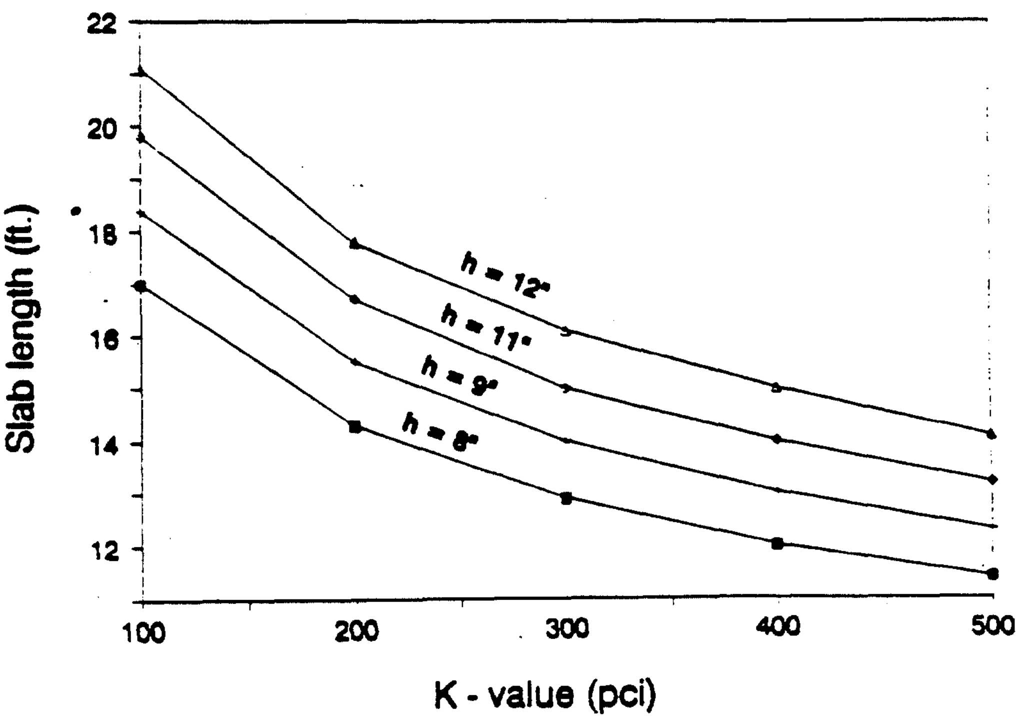
Studies have shown that pavement thickness, base stiffness, and climate affect the maximum anticipated joint spacing beyond which transverse cracking can be expected [2]. Research indicates that there is a general relationship between the ratio of slab length (L) to the radius of relative stiffness (ℓ) and transverse cracking. The radius of relative stiffness is a term defined by Westergaard to quantify the relationship between the stiffness of the foundation and the flexural stiffness of the slab. The radius of relative stiffness has a lineal dimension and is determined by the following equation:

$$\ell = [Eh^3/12k(1-\mu^2)]^{0.25}$$

where

ℓ = radius of relative stiffness (in.)
E = concrete modulus of elasticity (psi.)
h = pavement thickness (in.)
μ = Poisson's ratio of the pavement
k = modulus of subgrade reaction (pci.)

Research data indicates that there is an increase in transverse cracking when the ratio L/ℓ exceeds 5.0. Using the criteria of a maximum L/ℓ ratio of 5.0, the allowable joint spacing would increase with increased slab thickness, but decrease with increased (stiffer) foundation support conditions. The relationship between slab length, slab thickness, and foundation support for a L/ℓ ratio of 5.0 is shown below.



TIEBAR PULL-OUT TESTS

Proper consolidation of the concrete around the tiebars is essential to the performance of longitudinal construction joints. Adjacent lanes should not be constructed until the project engineer has had opportunity to test the pull-out resistance of the tiebars. Acceptance of the tiebars should be based on the results of the tests for resistance to pull-out. The project engineer will select 15 tiebars from the first day's placement, after the concrete has attained a flexural strength of 550 psi. The tiebars will be tested to 12,000 lbs. or to a slippage of 1/32-inch, whichever occurs first. The average of the results of these pull-out tests, divided by the spacing of the tiebars, will be used to determine the pull-out resistance in lbs. per linear foot.

If the test results on the first day's placement are well within the test requirements shown below, additional testing will be at the discretion of the project engineer and will be based on comparison of the installation methods and spacings of the first day's placement with subsequent placements.

If the results of the pull-out tests are less than the minimum requirements specified for the width of concrete being tied, the contractor shall install additional tiebars to provide the minimum average pull-out resistance required, as directed by the project engineer. Testing of the supplemental tiebars will be at the discretion of the Engineer.

Tiebars shall be installed by methods and procedures such that the tiebars will develop the minimum average pull-out resistance specified without any slippage exceeding 1/32-inch in accordance with the following table:

Tied Width of Pavement (Distance from Joint Being Constructed to Nearest Free Edge).	Average Pull-out Resistance of Tiebars, lbs./L.F. of joint, minimum.
12 feet or less	2200
Over 12 feet to 17 feet	3200
Over 17 feet to 24 feet	4500
Over 24 feet to 28 feet	5200
Over 28 feet to 36 feet	6800
Over 36 feet	9000

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U.S. Department
of Transportation

Federal Highway
Administration

Memorandum

Subject: Technical Paper - The Benefits of Using Dowel Bars

Date: MAY 17 1989

From: Director, Office of Highway Operations
Washington, D.C. 20590

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators
Direct Federal Program Administrator

Attached for your use are two copies of a technical paper on the benefits of using dowel bars in PCC pavements. This paper suggests that dowel bars should be used on all pavements, except possibly those with low truck volumes. The paper also points out the need for proper installation if the benefits of the dowels are to be realized. Many States are experiencing premature deterioration of their undoweled PCC pavements. In these cases, we encourage the field offices to work with the States in evaluating the merits of dowel bars.

We suggest that a copy of this paper be forwarded to each division office. We appreciate the efforts of the regional offices in reviewing the draft of this paper. If you have any questions concerning this paper, or wish to offer information relating to recent field experience with the installation of dowel bars, please contact Mr. David Law at FTS 366-1341.

N. J. Van Ness

Norman J. Van Ness

TECHNICAL PAPER 89-03 -- Benefits of Using Dowel Bars

Over the past few years, Pavement Division personnel have reviewed several sections of undoweled PCC pavements. In general, these pavements have experienced a level of deterioration due to faulting that is significantly greater than that found in comparable sections of doweled PCC pavements. This finding has led to a concern over the design and construction of undoweled pavement sections. The purpose of this brief paper is to illustrate the benefits of using dowels on jointed PCC pavements, particularly on those routes carrying a large number of trucks.

For jointed PCC pavements to perform satisfactorily, traffic loads must be effectively transferred from one slab to the next. Without adequate load transfer, the pavement is subjected to a variety of distresses, such as pumping, faulting, and corner breaks. There is considerable disagreement on how load transfer should be obtained. One school of thought is to rely on aggregate interlock in combination with short joint spacings, skewed joints, and stabilized subbases. The other school of thought is to rely on load transfer devices, such as dowel bars.

Aggregate interlock is ineffective at crack widths greater than 0.035 inch. A smaller crack width, generally 0.025 inch, is considered necessary for satisfactory long-term performance of undoweled pavements. An Iowa DOT study(1) of undoweled pavements concluded that "from measurements of joint openings it appears doubtful that aggregate interlock is maintained even by joints spaced at 20 ft." When measured beneath the sawed portion of the joint, over 90% of the joints had crack widths in excess of 0.06 inch. In order to limit crack widths to 0.035 inch over a temperature range of 60-80 Fahrenheit degrees, joint spacings in the range of 6 to 11 feet are needed. Such a spacing is not considered practical. Properly sized dowels, on the other hand, provide effective load transfer at reasonable joint spacings. Maximum joint spacings of 15-20 ft. and 30-40 ft. are recommended for plain and reinforced pavements respectively.

The use of dowels has been shown to reduce faulting. A Florida DOT study(2) concluded that "doweled contraction joints fault less than non-doweled contraction joints." A Georgia DOT study(3) of a project on I-85 found that "dowel bars were effective in reducing the faulting at the contraction joints." The Wisconsin DOT conducted a condition survey of their Interstate system. One of the findings of this study(4) was that "building nonreinforced concrete pavements with additional thickness (2-3 inches) in lieu of using positive load transfer devices (dowel bars) at transverse contraction joints is not successful in preventing or reducing joint faulting to an acceptable level during a pavement's life." Faulting at the joints was notably absent during the AASHO Road Test. One transverse joint faulted seriously, but investigation showed that the joint had been accidently sawed at some distance beyond the end of the dowel. Over the two-year test period, there were no other cases of measurable faulting at the joints, all of which were doweled. Based on road tests performed at the NARDO track in Italy, the XVIII World Road Congress reported that dowels significantly increased the pavement service life[5]

Dowels reduce deflections at the joint, which in turn reduce the magnitude of concrete flexural stresses. These deflections and stresses are reduced due to the load being more effectively shared with the adjoining slab through shear and bending stresses in the dowel itself. Reduced concrete flexural stresses increase the fatigue life of the pavement and thus extend its service life. A theoretical analysis indicates that a 10" doweled slab with 80% load transfer will have the same deflection as a 12" undoweled slab with only 40% load transfer. Dowels can also reduce the potential for premature failure due to corner breaking caused by loss of subgrade support through pumping.

When dowels are properly designed and installed, they can reduce faulting and increase the pavement's service life. When they are not, dowels can cause premature failure of the pavement in the vicinity of the joint. Dowels too small in diameter to handle the necessary stresses have resulted in premature joint failures. Excessive concrete bearing stresses have crushed the concrete around the dowels and allowed faulting to occur. Considerable research has been performed recently which supports the use of larger bars (1-1/4 to 1-1/2 inch). The AASHTO **Guide for Design of Pavement Structures** recommends a dowel diameter of 1/8th the pavement thickness, with the dowels placed near the center of the slab to minimize bending stresses. Most States are currently using, as a minimum, 1-1/4 inch diameter bars, with good results. Most States are using dowels 18 inches long spaced at 12 inches and are reporting no problems.

Dowels should also be corrosion-resistant. The use of epoxy-coated or stainless-steel dowels has been shown to provide the necessary resistance to corrosion. It is important that a bond-breaker be applied to the dowels to allow the slabs to freely and independently expand and contract without developing restraint forces. This bond-breaker should be applied to provide a thin but uniform coating.

Many of the past performance problems associated with doweled joints were the result of excessive joint spacings, ranging from 60 to 100 ft. The trend to plain doweled slabs with joint spacings of 15 to 20 ft. eliminates many of these problems. Shorter joint spacings result in smaller crack widths, which reduce the stresses acting on the dowels. The shorter spacing also reduces slab movement, which makes dowel alignment less critical, as the restraint forces due to misalignment are directly proportional to the amount of slab movement. In addition, the effect of two slabs acting as one as the result of a locked or "frozen" joint is not as severe for the shorter slab lengths.

In order to perform satisfactorily, dowels must also be reasonably aligned. The prevailing practice is to specify dowel alignment tolerances on the order of 1 percent, or roughly 1/8-inch per foot. This frequently results in a high percentage of the dowels being "out of specs" and gives the impression that obtaining proper dowel alignment is very difficult. Studies[(6)(7)] suggest that the alignment tolerances can be relaxed. FHWA now recommends an alignment tolerance of 1/4 inch per foot and will be evaluating the possibility that these tolerances can be further relaxed. It is recognized that the problems with misaligned dowels are generally the result of gross misalignment occurring during concrete placement. Equipment to precisely measure compliance with alignment tolerances after concrete placement are not readily available. However, it is recommended that the completed pavement joints be inspected using a metal detector to verify that no significant dowel misalignment has occurred.

The use of mechanical dowel bar inserters holds promise for the improved installation of dowel bars. Two manufacturers, Guntert-Zimmerman and Gomaco, have developed and are marketing new automatic inserters. It is their claim that these inserters are capable of placing dowels more efficiently and at less cost than basket assemblies without sacrificing placement accuracy.

Construction Technology Laboratories was recently retained to monitor the results of the placement of dowels using these new machines. The Guntert-Zimmerman inserter was evaluated on projects in Texas and Wisconsin and the Gomaco inserter was evaluated on an Idaho project. The placement of dowels using basket assemblies was also monitored in Texas and Wisconsin. Preliminary findings indicate that the inserters placed the dowels with approximately the same accuracy as dowels placed using basket assemblies. Cost figures from the Wisconsin study indicate that a savings of approximately $ 0.35 per sq. yd. of concrete pavement was obtained by using the dowel implanter in lieu of dowel baskets.

The use of dowels is strongly encouraged on all pavements except possibly those with very low truck volumes. Dowels can provide a higher serviceability level over a longer period of time than pavements relying only on aggregate interlock for load transfer. Dowels can minimize pavement distress caused by overloads or heavier loads travelling by permit.

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TECHNICAL PAPER 89-04 -- PREFORMED COMPRESSION SEALS FOR PCC PAVEMENT JOINTS

Joint sealants for jointed concrete pavements are intended to prevent, or at least deter, the intrusion of water and incompressibles into the joint and pavement structure. Water entering a joint can lead to pumping and faulting, while incompressibles in a joint can cause spalling and blow-ups. A joint sealant must be capable of remaining in firm contact with the concrete at the faces of the joint while withstanding repeated expansion and contraction of the pavement slabs due to thermal variations. There are two types of joint sealants which are currently recognized as having the potential for satisfactory long-term performance. These are the preformed compression seals and the low-modulus silicones. The purpose of this technical paper is to discuss key factors in the design and installation of the preformed compression seals.

DESIGN: Preformed compression seals should be designed so that the sealant will be in compression at all times. These seals are typically manufactured from a neoprene compound and factory molded into a web design. The seal is compressed and inserted into the pavement joint. These compressed webs exert an outward force which keeps the seal tightly pressed against the joint faces, thus effectively sealing the joint. As long as these seals are in compression, they will generally be effective. If compression is lost, they will fail. It is essential to maintain a good uniform seal between the joint faces and the compression seal.

Generally, compression seals function best when compressed between 20 percent and 50 percent of their nominal width. This range will vary slightly with manufacturer and the seal dimensions. Compressive forces less than 20 percent may not be sufficient to hold the sealant in place. If the seal is exposed to compressive forces greater than 50 percent for an extended period of time, it may undergo a compression set. Compression set occurs when the seal doesn't recover to its initial position. Once it undergoes compression set, the seal will not expand as the joint opens, resulting in a total loss of compression and joint sealant failure.

When designing joints using compression seals, the anticipated joint movement, the uncompressed width of the compression seal, and the joint width must all be determined. The first step is to determine the anticipated joint movement, using the following equation:

$$\Delta L = CL * (\alpha \Delta T + \epsilon)$$

where:

ΔL is the anticipated amount of joint movement.

C is the slab/subbase frictional restraint factor (0.65 for stabilized subbases, 0.8 for granular subbases).
L is the joint spacing, in inches.
α is the PCC coefficient of thermal expansion (4.5-6.5 $\times 10^{-6}$). This coefficient is primarily affected by aggregate type. It generally ranges from 5×10^{-6}/°F for the carbonaceous aggregate mixes to 6×10^{-6}/°F for the siliceous aggregate mixes. For further information, see Reference #1, pages 151-152.
ΔT is the maximum anticipated temperature range, generally the mean maximum daily temperature for the hottest month minus the mean minimum daily temperature for the coldest month.
ϵ is the shrinkage coefficient of concrete (0.5-2.5 $\times 10^{-4}$). This factor is ignored on rehabilitation projects, as drying shrinkage has already taken place.

The second step is to select the uncompressed width of the compression seal. The sealant manufacturers have information available which should be used in this selection process. If ΔL is the amount of joint movement and W is the width of the uncompressed seal, then $\Delta L \div W$ should be less than or equal to the allowable movement of the compression seal. This range of allowable movement should be obtained from the sealant manufacturer and typically varies from 50 percent maximum compression to 20 percent minimum compression. If $\Delta L \div W$ is too large, then either the amount of joint movement should be reduced by decreasing the joint spacing or the width of the compression seal should be increased. The width of the uncompressed seal can be determined from the following equation:

$$W \geq \Delta L \div (C_{max} - C_{min})$$

where:

W is the width of the uncompressed seal.
ΔL is the anticipated amount of joint movement.
C_{max} is the maximum recommended compression of the seal, as a decimal (typically 0.5).
C_{min} is the minimum recommended compression of the seal, as a decimal (typically 0.2).

The final step is to select the joint width, based on the width of the compression seal and the anticipated temperature of the pavement at the time of sealant installation. (This need only be a rough estimate.) An approximate installation temperature is necessary so that the compression seal can be installed at the proper compression. Warmer installation temperatures necessitate greater initial compression, as the pavement is closer to its maximum expansion. This will allow the seal to remain sufficiently compressed during cold weather. Conversely, cooler temperatures require a lower initial compression, as the pavement is nearer its maximum contraction. This will prevent the seal from undergoing excessive compressive forces during hot weather. The width of the joint sawcut can be determined from the following equation:

$$Sc = (1 - Pc) * W$$

where:

Sc is the width of the joint sawcut.
Pc is the percent compression of seal at installation, expressed as a decimal.
W is the width of the uncompressed seal.

$$Pc = C_{min} + \left[\frac{\text{Install temp - Min temp}}{\text{Maximum temp - Min temp}}\right] * (C_{max} - C_{min})$$

It should be pointed out that this procedure is approximate; saw blades and compression seals are only available in a limited number of widths. This design procedure is not dependent upon precise temperature predictions and minute variations in joint widths.

Since the pavement temperature at the time of seal installation is not known at the design phase, it is recommended that the design be flexible enough to allow for installation of compression seals over a wide range of temperatures. This can best be achieved by reducing the joint spacing, preferably to 30 feet or less, as shorter joint spacings significantly reduce the amount of joint movement. Selecting a compression seal one or two sizes larger than the minimum required by $\Delta L+W$ will reduce the sensitivity of the design to the installation temperature. Regardless, it may still be necessary to either vary the joint width to account for the pavement temperature at the time of seal installation or to prohibit the installation of compression seals during certain temperatures (i.e., less than 45°F).

Differential vertical movements at the joint also affect seal performance. The greater the vertical movement, the greater the potential that the seal will "walk" up and out of the joint. Doweled joints will reduce vertical movements and are recommended when using compression seals.

Compression seals should not be used within 100 feet of expansion joints. Joints near expansion joints can be expected to expand sufficiently to allow these seals to loosen and pop out.

INSTALLATION: Proper construction techniques must be followed to ensure that the compression seals will perform as intended. Improper installation procedures are a primary cause of premature failures of these seals. Close attention must also be paid to the manufacturer's recommendations.

The joint faces must be vertical, so that the seal does not work itself up and out of the joint. Any spalls at the joint should be patched prior to installation of the compression seal. (Spalls less than 1/4-inch may remain; however, the seal should be recessed sufficiently to avoid the spalled area.) Irregularities in the joint width could reduce the pressure on the seal to the point that it would no longer remain in compression.

It is recommended that the concrete surfaces at the joint be dry prior to installation of the compression seal. The joint should be air-blasted to remove any debris. Both the air temperature and the temperature of the pavement should be above freezing. Prior to installing the compression seal, a lubricant-adhesive should be applied to either the joint faces or the seal. This material primarily serves as a lubricant to facilitate the installation process. This material also cures to form a weak adhesive, which helps keep the seal at the proper height. However, it does not provide any tensile strength.

The compression seal should be adequately recessed, so that it won't be damaged by traffic. The joint edge may be beveled to reduce spalling. A 1/4-inch radius bevel or a 1/8-inch straight bevel is sufficient. The compression seal should be recessed approximately 1/8-inch below the bottom of the bevel. When the joint edge is not beveled, the seal should be recessed from 1/8-inch to 3/8-inch beneath the top of the slab. The seal may be recessed up to 1/2-inch if grinding of the concrete pavement is anticipated in the future. While this additional depth should prevent the seal from being damaged by the grinding operation, it may allow incompressibles to accumulate and cause spalling. The joint reservoir should be deep enough to allow the seal to be compressed without extruding to an elevation where it will be exposed to traffic.

Care should be taken to not stretch the seal during the installation process. A stretched seal will not perform as well or as long as a properly installed seal. The seal should also not be twisted, as intimate contact must be maintained between the seal and joint faces over the full length of the seal. Most compression seal manufacturers have developed installation equipment which do not stretch, twist or damage the seals. This type of equipment should be used.

When sealing a width of two lanes or less, splices should not be permitted. When sealing more than two lanes, one splice may be permitted; however, the contractor should closely follow the manufacturer's recommendations for splicing.

Close inspection of the installation procedure is necessary to ensure that the seals will perform as intended. The inspector should verify that the pavement joint is sawed to the proper width and depth, and that the compression seal is the correct width prior to commencing sealing. The tolerance for the joint width should be ± 1/16-inch. During the installation process, the inspector should verify that the seal is not being stretched. This can be done by comparing the distance between two marks on the surface of the seal measured before and after installation. The inspector should also visually inspect the compression seal to ensure that it has not been twisted or damaged, and is adequately recessed.

<u>SUMMARY:</u> If properly designed and installed, preformed compression seals have the potential to provide excellent performance over an extended period. It is not uncommon to find compression seals more than 10 years old still performing as well as newly installed seals. To ensure this type of performance, both the width of the pavement joint and the pavement temperature at the time of installation need to be coordinated with the width of the compression seal. The joint should be designed and constructed so that the compression seal will function entirely within the manufacturer's recommended operating range, generally 20 percent to 50 percent compression of the uncompressed seal width. This may necessitate that seal installation be prohibited during certain extremes in pavement temperature. Satisfactory joint sealant performance is dependent upon good construction procedures and proper inspection.

<u>SAMPLE DESIGN CALCULATIONS:</u> The design of compression seals is a simple procedure. The following examples show how a design process could be used.

EXAMPLE A:

A State wants to use preformed compression seals on all new PCC pavement projects. Their standard design calls for a 60-foot joint spacing on a granular base. The expected temperature range is from 14°F to 90°F.

Step 1 - Determine the anticipated joint movement.

$$\Delta L = CL\ (\alpha\Delta T + \epsilon)$$
$$= (\ 0.8\ )(\ 60 * 12\ )[\ (\ 5.5*10^{-6}\ )\ (\ 76\ ) + (\ 1.0*10^{-4}\ )\ ]$$
$$= 576\ (\ 4.18*10^{-4} + 1.0*10^{-4}\ )$$
$$= 0.3 \text{ inch}$$

Step 2 - Select width of uncompressed seal.

(This particular sealant manufacturer recommends an operating range of 55 percent to 20 percent)

$$W \geq \Delta L \div (C_{max} - C_{min})$$
$$\geq 0.3 \div (\ 0.55 - 0.20\ )$$
$$\geq 0.3 + 0.35$$
$$\geq 0.86 \text{ inch}$$

use W = 1 inch

Step 3 - Select width of sawcut.

$$Pc = C_{min} + \left[\frac{\text{Install temp} - \text{Min temp}}{\text{Maximum temp} - \text{Min temp}}\right] * (C_{max} - C_{min})$$

$$Sc = (\ 1 - Pc\ ) * W$$

Case 1: Installation temperature = 80°F

$$Pc = 0.2 + [\ (\ 66 \div 76\ ) * (\ 0.55 - 0.2\ )\ ]$$
$$= 0.2 + (\ 0.87\ ) * (\ 0.35\ )$$
$$= 0.50$$

$$Sc = (\ 1 - 0.5\ ) * 1$$
$$= 0.5 \text{ inch}$$

Case 2: Installation temperature = 40°F

$$Pc = 0.2 + [\ (\ 26 \div 76\ ) * (\ 0.55 - 0.2\ )\ ]$$
$$= 0.2 + (\ 0.34\ ) * (\ 0.35\ )$$
$$= 0.32$$

$$Sc = (\ 1 - 0.32\ ) * 1$$
$$= 0.68 \text{ inch}$$

EXAMPLE B:

The State elects to change their design by reducing the joint spacing to 30 feet.

Step 1 - Determine the anticipated joint movement.

ΔL is directly proportional to L; decreasing L by 50 percent decreases ΔL by 50 percent.
$\Delta L = 0.15$ inch

Step 2 - Select width of uncompressed seal.

$W \geq \Delta L \div (C_{max} - C_{min})$
$\geq 0.15 \div (0.55 - 0.20)$
$\geq 0.15 \div 0.35$
≥ 0.43 inch
use W = 0.688 inch (a larger size than necessary)

Step 3 - Select width of sawcut.

$$Pc = C_{min} + \left[\frac{\text{Install temp - Min temp}}{\text{Maximum temp - Min temp}}\right] * (C_{max} - C_{min})$$

$Sc = (1 - Pc) * W$

Case 1: Installation temperature = 80°F

Pc = 0.2 + [(66 ÷ 76) * (0.55 - 0.2)]
= 0.2 + (0.87) * (0.35)
= 0.50

Sc = (1 - 0.5) * 0.688
= 0.344 inch

Case 2: Installation temperature = 40°F

Pc = 0.2 + [(26 ÷ 76) * (0.55 - 0.2)]
= 0.2 + (0.34) * (0.35)
= 0.32

Sc = (1 - 0.32) * 0.688
= 0.47 inch

Because a larger seal was used, would a sawcut width of 0.375 inch work regardless of the temperature at installation?

Case 1: Insufficient compression

(Seal width - Max. joint opening (JO_{max})) ÷ Seal width = C_{min}

$(0.688 - JO_{max}) \div 0.688 = 0.2$

$JO_{max} = 0.688 - (0.2) (0.688)$

$JO_{max} = 0.55$ inch

$JO_{max} - Sc$ = allowable movement

0.55 - 0.375 = allowable movement

0.175 inch = allowable movement

0.175 inch $\geq$ 0.15 inch (anticipated joint movement)

The seal will not be undercompressed.

Case 2: Compression set

(Seal width - Min. joint opening (JO_{min})) ÷ Seal width = C_{max}

$(0.688 - JO_{min}) \div 0.688 = 0.55$

$JO_{min} = 0.688 - (0.55) (0.688)$

$JO_{min} = 0.31$ inch

$Sc - JO_{min}$ = allowable movement

0.375 - 0.31 = allowable movement

0.065 inch = allowable movement

0.065 inch $\leq$ 0.15 inch (anticipated joint movement)

The seal may undergo compression set.

The acceptable installation temperature range

= [allowable movement ÷ total movement] * temp range

= (0.065 ÷ 0.15) * 76°

= 33°

90° - 33° = 57°

The seal will not undergo compression set so long as the joint is sawed and sealed when the pavement temperature is greater than 57°F. It is recommended that the specifications be revised to limit the installation operation to temperatures above 57°F or the design revised to provide a shorter joint spacing.

<u>References</u>

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5. "Studies of Factors Governing Contact Pressure Generation in Neoprene Preformed Compression Seals," Highway Research Record No. 200, 1967, F. H. Hall, J. H. Ritzi, and D. D. Brown.

6. "Techniques of Pavement Rehabilitation," Federal Highway Administration, 1987, ERES Consultants.

7. "Standard Specification for Preformed Elastomeric Compression Joint Seals for Concrete (AASHTO Designation: M 220-67)," AASHTO Materials - Part I Specifications, 1982, American Association of State Highway and Transportation Officials.

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: Examination of Cores from Kansas I-70

Date: JUL 25 1989

From: Chief, Pavement Division
Washington, D.C. 20590

Reply to Attn of: HHO-12

To: Mr. Thomas J. Ptak,
Deputy Regional Federal Highway Administrator HEO-07
Kansas City, Missouri

Attached is Dr. Stephen Forster's report on the examination of the concrete cores from Kansas I-70 east of Abilene (Kansas Project No. 70-21 K-2588-01). Dr. Forster did not find any evidence of "D" cracking of the aggregate or alkali-aggregate reactivity. The crack faces appear rough enough to provide load transfer if the cracks remain tight. However, the cracks in these cores have opened to the point where load transfer has been lost and the cracks are working.

We are observing a significant number of jointed reinforced concrete pavements (JRCP) with working cracks. The two factors believed to be the primary cause of working cracks in JRCP are corroded and locked up dowel bars and inadequate reinforcement. The introduction of epoxy coated dowels has reduced the risk of dowel bar corrosion. However, the procedures used to determine the amount of reinforcement in JRCP are not adequate.

Reinforcement for JRCP is designed using the subgrade drag theory. The procedure does not consider the crack aggregate interlock capability or the repeated shear loads from traffic. Also, the subgrade drag theory does not directly consider climatic effects. In the absence of a good design procedure for the reinforcement in JRCP, we believe the following conclusion from an ongoing research study "Performance/Rehabilitation of Rigid Pavements" provides good guidance:

> "The amount of <u>reinforcement</u> appeared to have an effect in controlling the amount of deteriorated transverse cracking. Although often confounded by the presence of corrosion-resistance dowel bars, pavement sections that contained more than 0.1 percent reinforcing steel exhibited less deteriorated transverse cracking; sections with less than that amount often displayed a significant amount of transverse cracking, particularly in cold climates. A <u>minimum</u> of 0.1 percent reinforcing steel is therefore recommended, with larger amounts required for harsher climates and longer slabs."

Based on Kansas Standard Plan 707.2, it appears there was approximately 0.07 percent reinforcing steel in the Kansas I-70 pavement. We recommend that the State consider increasing the amount of reinforcing steel on future projects.

Correction of working cracks is a very costly process. The available alternatives include installation of retrofit dowels and full-depth patching. If the retrofit dowel technique is used, the work should be performed before the cracks start to deteriorate. A minimum of three dowels is required in each wheelpath. The cost per dowel should be in the range of $30 to $70, depending on the quantities, labor costs, and hardness of the aggregate. When the full-depth repair option is selected, work should be performed after the distress begins to have a serious impact on pavement serviceability. The use of full-depth patching after the distress has occurred is generally the preferred alternative for several reasons: (1) It is difficult to predict whether the working cracks will result in a significant reduction in pavement serviceability. (2) If the rate of serviceability loss is low, the full-depth patching can be performed at the same time future rehabilitation needs are addressed.

Attached is a copy of the latest draft of the report for the research project "Performance/Rehabilitation of Rigid Pavements." Also attached is information on retrofit dowel bar installation.

Please contact Mr. John Hallin at 366-1323, if you have any questions concerning these comments.

Louis M. Papet

U.S. Department
of Transportation

Federal Highway
Administration

Memorandum

Subject: Dowel Bar Inserters

Date: February 23, 1996

From: Chief, Pavement Division

Reply to Attn. of: HNG-40

To: Regional Administrators
Federal Lands Highway Program Administrator
Attention: Regional Pavement Engineers

By a March 6, 1990, memorandum, Mr. Louis Papet provided a copy of a Wisconsin Department of Transportation report on "Dowel Bar Placement: Mechanical Insertion Versus Basket Assemblies." Since that time, there appears to have been poor acceptance of the use of dowel bar inserters. A recent draft NCHRP report noted that 8 States allow the use of inserters, 13 States allow it as an acceptable option, and 20 States do not allow their use.

This technique has been used exclusively in some European countries for over 20 years with satisfactory dowel placement results. We believe all States should be encouraged to make this an allowable option in their specifications. We continue to encourage checking of dowel tolerances by probing through the fresh concrete early during the project and periodically as the work progresses. We also continue to recommend that when either baskets or inserters are used, the location of the dowels in the completed pavement be verified using metal detectors, pachometers, and cores.

If you have any comments or questions please contact Mr. John Hallin at (202) 366-1323 or Mr. Roger Larson at (202) 366-1326

T. Paul Teng

T. Paul Teng, P.E.

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: Dowel Bar Inserters

Date: MAR 6 1990

From: Chief, Pavement Division
Washington, D.C. 20590-0001

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Attention: Pavement Engineers

Attached for your information is a copy of a report prepared by the Wisconsin Department of Transportation (WDOT) entitled "Dowel Bar Placement: Mechanical Insertion Versus Basket Assemblies." This study found that dowel placement accuracy achieved with the mechanical inserters equaled or surpassed the accuracy achieved with basket assemblies. As a result, the WDOT now permits the use of mechanical dowel bar inserters on construction projects.

Wisconsin's evaluation of dowel placement accuracy was based on their specification, which permits an alignment tolerance of 1/2-inch per dowel. This is slightly greater than the 1/4-inch per foot (3/8-inch per dowel) recommended in our May 17, 1989, Technical Paper 89-03, Benefits of Using Dowel Bars. Wisconsin is using a joint spacing of 12-13-19-18 feet and has not reported any distress which would indicate dowel alignment problems. As pointed out in Technical Paper 89-03, we are continuing to evaluate the specification tolerances for dowel alignment.

We concur with the WDOT's conclusion that: "The initial set-up of the dowel bar inserter with respect to depth of dowel placement is critical at the start of each project, and dowel depths should be verified by probing through the fresh concrete." We also recommend that when either baskets or inserters are used, the location of the dowels in the completed pavement be verified using metal detectors, pachometers, and cores.

If you have any questions or comments, please contact Mr. John Hallin at 366-1323.

Louis M. Papet

DOWEL BAR PLACEMENT:
MECHANICAL INSERTION VERSUS BASKET ASSEMBLIES

FINAL REPORT

WISCONSIN DEPARTMENT OF TRANSPORTATION

MARCH 1989

3.5.2

Dowel Bar Placement: Mechanical Insertion Versus Basket Assemblies

Final Report
Project I.D. 0624-32-08
Study No. 88-10

Workplan and Project Coordination by:
Ashwani K. Sharma
Research Project Engineer

Analysis and Report by:
James M. Parry, P.E.
Pavement Monitoring Engineer

February 1989

Wisconsin Department of Transportation
Division of Highways and Transportation Services
Central Office Materials
Applied Research and Pavement Management Sections

ABSTRACT

A mechanical dowel bar inserter was used on three highway construction projects in Wisconsin in 1987 and 1988. Coring was performed on these projects, and on three projects where dowel basket assemblies were used, to determine the dowel placement accuracy of both techniques. Study results indicate that the dowel placement accuracy achieved with the mechanical inserter equaled or surpassed the accuracy achieved with basket assemblies. Based on the results of this study, the mechanical dowel bar inserter will be allowed as an equal alternate to basket assemblies on 1989 construction projects in Wisconsin.

There were some problems on the initial projects where the mechanical inserter was used, including occasional missing dowels, improper location of sawed joints with respect to the location of the dowels, and voids in the concrete above the ends of the dowel bars. However, with continued refinement of construction techniques by the contractors and careful inspection by WisDOT construction personnel, it is believed that these problems associated with the new technology can be reduced or eliminated on future projects.

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INTRODUCTION

Traditionally, the dowel bars at Portland Cement Concrete (PCC, transverse pavement joints have been placed using wire basket assemblies, which are staked to the base prior to paving. Dowel basket assemblies are expensive, and their placement is labor intensive. For at least three decades, contractors and equipment manufacturers have been trying to develop a piece of paving equipment which can accurately place dowels in the plastic concrete at transverse joints, to eliminate the need for dowel baskets.

In the days of side form paving, a machine was developed and used in several states that was doing an acceptable job of vibrating dowels in ahead of the final finishing machine, but with the advent of slipform paving this machine proved unsatisfactory. If the dowels were placed between the spreader and the slipform, the dowels tended to settle or move horizontally when the slipform passed over them. If the dowels were vibrated in behind the slipform paver, there was a depression at the joint location which could not be removed by hand finishing with a straightedge.(1)

In recent years many states, including Wisconsin, are specifying dowels in all transverse joints on heavily trafficked PCC pavements, due to excessive faulting which has occurred on existing pavements without dowels. Shorter joint spacings are now also commonly used, which require many more dowel baskets p mile of pavement. These changes in design policy have stimulated even greater interest in developing a mechanical dowel bar inserter that will work with a slipform paver.

For several years a mechanical dowel bar inserter made by Guntert & Zimmerman has been used in Europe with reported success. A Wisconsin paving contractor purchased one of these machines, and was granted an opportunity to use it on an experimental basis on a project on I-90 at Janesville in 1987. Construction Technology Laboratories, Inc., of Skokie, IL (CTL) was retained to conduct a study of the dowel placement accuracy of the inserter versus baskets on the Janesville project, using a ground penetrating radar system.

The report from this study was reviewed by WisDOT staff, and the study results were found to be inconclusive. This was principally due to shortcomings of the ground penetrating radar technology used in the study. The problems included lack of precision of the measurements for some of the dowel placement parameters, and marginal correlation between the radar data and coring results. Consensus opinion of the WisDOT staff was that additional investigation was needed before the dowel inserter could be approved for general use.

Numbers in parentheses denote references given at end of repor

In July of 1988 this need for additional information was addressed when the workplan for the current study was developed. Coring was chosen as the method to be used for evaluation of dowel location despite the destructive nature of the testing, because of the precise and accurate results which could be obtained. Two additional pavements were constructed using the dowel inserter in 1988, so a more broad-scaled investigation was now possible. The purpose of the current study is to determine whether the dowel bar inserter is acceptable as an equal alternate to dowel baskets for future WisDOT paving projects.

The report by Construction Technology Laboratories, entitled "Field Evaluation of Dowel Placement along a Section of I-90 Near Janesville, Wisconsin", which was the final report for the radar study, will be hereafter refered to as the CTL report. (2)

SAMPLING AND CORING PROCEDURES

Number of Projects: Since three dowel inserter projects were available for evaluation, three additional independent projects using dowel baskets were also selected for this study. The comparison section with dowel baskets on the I-90 Janesville dowel inserter project was not included in this study due to reports from field personnel that unusual care was taken in placing the baskets on that project. The six projects that were selected for this study are described in Table 1.

Number of Dowels per Project: To achieve good statistical reliability while keeping costs and coring damage down to a reasonable level, a nominal sample size of 90 dowels per project was selected.

Location of Test Joints: The test joint locations on each project were selected in such a manner so that a representative sample would be collected. The optimum procedure would have been to select joints at random intervals throughout the project, but traffic control cost constraints dictated the need to concentrate the testing in limited test sections. Three test sections were designated for each project. The test sections were located a mile apart, and were located in the central portion of the project. Spreading the test sections out in this manner assured that all test joints would not fall in a single isolated problem area of the project. Central location of the test sections on the project assured that samples would not be taken near the ends of the project, where "start-up" paving problems typically occur. The location of the starting points for each of the test sections on each of the projects are given in Table 2.

A total of 15 test joints were designated in each test section. To assure equal rotation through the 4-joint repeating random skewed joint spacing pattern, every third joint in the test section was designated as a test joint.

Location of Test Dowels in Test Joints: At each test joint, one dowel was tested in the passing lane, and one dowel was tested in the driving lane. This was done to assess the relative placement error as it varied across the test joint. On the South Madison Beltline project, which had three lanes per direction, testing was confined to the two adjacent lanes (closest to the median) which were paved simultaneously with the dowel bar inserter.

The lateral position of each of the dowels across the pavement was identified by numbering the dowels for each lane in ascending order from the shoulder to the longitudinal joint between the two lanes, as shown in Figure 1. The lateral position of the test dowels for all successive test joints on a project was determined by rotating through the random number sequence shown in Table 3. This assured even representation in the sample for all dowel positions across the lanes.

Coring Procedure: A metal detector was used to locate the designated test dowel at the test joint. Partial depth core holes, centered over the ends of the dowel, were then drilled down to the depth of the dowel. The concrete core was then snapped off at the depth of the dowel, exposing the upper portion of the ends of the dowel bar for location measurements.

Measurements Collected for Test Dowels: The following measurements were collected for each end of each test dowel.

Vertical position: The distance from the top of the dowel end to the pavement surface was measured directly with a tape measure, either from the core or the core hole. These measurements were used to determine the average depth and vertical rotation of the dowels.

Lateral position: The distance from the center of the dowel end to the shoulder edge of the lane was measured using the following procedure. A two-foot carpenter's level was fitted with a custom-built tripod with leveling screws. A small black mark was made at the center of the dowel end. To project a vertical line above the dowel center, the level was plumbed vertically and the edge of the level was sited in to line up with the mark on the dowel. An eight-foot straightedge was laid longitudinally at the shoulder edge of the pavement. A tape measure was stretched transversely along the pavement surface from the siting edge of the level to the straightedge at the shoulder, and the resulting measurement was defined as the lateral position. These measurements were used to determine the horizontal rotation of the dowels.

Longitudinal position: The distance from the end of the dowel to the sawn joint was measured using the following procedure. To project a vertical line above the end of the dowel, the same mounted carpenter's level was plumbed and sited in to line up with the end of the dowel. A tape measure was laid on the pavement surface directly over and parallel to the dowel from the siting edge of the level to the center of the sawn joint. The resulting measurement was defined as the longitudinal position. These measurements were used to determine the longitudinal offset from the center of the dowel to the sawn joint.

Dowel Placement Parameters Evaluated: The four dowel placement parameters evaluated in this study are defined below.

Note that lateral spacing of the dowels was not evaluated in this study, because consecutive dowels across a single joint were not cored. Lateral spacing is not an especially sensitive parameter, and previous WisDOT coring on basket projects and the CTL report on the Janesville inserter project both showed no problem with meeting the contract specification (12" plus or minus 1") for lateral spacing.

The dowel placement parameters are illustrated in Figure 2.

<u>Vertical translation</u>: This is defined as the average depth of the dowel, measured from the top of the dowel to the pavement surface.

<u>Vertical rotation</u>: This is defined as the difference in depth (vertical position) between the opposite ends of the dowel.

<u>Horizontal rotation</u>: This is defined as the difference in lateral position between opposite ends of the dowel.

<u>Longitudinal translation</u>: This is defined as the longitudinal offset between the midpoint of the dowel and the sawn joint.

PLACEMENT TOLERANCE SPECIFICATIONS

The exact placement tolerance specifications used for this analysis are not critical, because the principal objective of this study is to compare the relative performance of the two dowel placement techniques. However, tolerance specifications do provide a useful frame of reference for this performance comparison, so the following specifications were used.

<u>Vertical Translation</u>: Specifications for this parameter were included in the construction contracts for all six projects in this study. The specifications consisted of a target depth for the dowels, and an allowable range of deviation. For the two 1987 construction projects, the I-90 Janesville project and the West Madison Beltline project, the target depth specified called for the dowels to be centered 1/2 inch above the mid-depth of the slab. For the four 1988 construction projects, the target depth specified called for the dowels to be centered at the mid-depth of the slab. For the I-90 project, the allowable range of tolerance for dowel depth was plus or minus 1 inch from the target depth. The same tolerance ranges were used in this analysis for the other five projects in the study.

<u>Vertical Rotation</u>: The tolerance specified in the CTL report for the I-90 project allowed 1/2 inch of vertical deviation from the true longitudinal axis of the pavement. This same tolerance was used for analysis of all of the projects.

<u>Horizontal Rotation</u>: The tolerance specified in the CTL report for the I-90 project allowed 1/2 inch of horizontal deviation from the true longitudinal axis of the pavement. This same tolerance was used for analysis for all of the projects.

<u>Longitudinal translation</u>: No specification was established for this placement parameter in the CTL report for the I-90 project. However, it was cited in a recent FHWA publication that it was necessary to have 6 inches of dowel on each side of the joint for effective load transfer and joint life.(<u>3</u>) Dowels longer than 12 inches are used in practice to allow leeway for joint sawing errors. Thus, for 18-inch dowels, a longitudinal offset of 3 inches in either direction is tolerable, and adequate load transfer is still provided. This tolerance was used for all projects in this analysis.

ANALYSIS OF PERFORMANCE

The coring results and statistics are summarized in Table 4 fo all six projects. A detailed listing of all dowel locations a measurements is provided in Appendix A.

The types of analysis that were performed for each of the dowel placement parameters included the following areas. A basic analysis of averages and distribution densities was conducted using means and standard deviations, and these statistics are included in Table 4.

Additionally, analysis was performed on the direction of deviation from the optimum dowel position (e.g. rotated left or right, rotated up or down, longitudinally offset forward or backward; with respect to the direction of paving). For all dowel placement parameters on all projects, the data had approximately balanced normal distributions which were centered at or near the optimum dowel position. The lack of skewed distributions and lack of distributions centered well away from the optimum position indicate that the variation is due to normal random fluctuation of the two dowel placement methods. It indicates that no pervasive systematic problems exist in either of the dowel placement processes which would caused skewed data.

Analysis of directional deviation of the dowel placement parameters versus the lateral position of the dowels across the roadway was also conducted. Again, for all placement paramete on all projects, the lateral position of the dowels across the roadway had no significant effect on the placement parameters of the dowels, indicating no pervasive systematic problems in either of the dowel placement processes.

Vertical Translation: First, the distribution of dowel depths was examined. The standard deviations for the inserter projects (0.20" to 0.46") were comparable or smaller than those for the basket projects (0.35" to 0.57"), indicating that the inserter is capable of consistent depth placement of the dowels. This does not necessarily mean that the inserter is better in this respect than baskets, because the frame of reference is different for the two placement methods. The depth of the dowels was measured from the pavement surface down to the top of the dowels. Dowel baskets are staked to the base course, thus referenced to the bottom of the slab. As the thickness of the pavement varies along the length of the project (typically 1" or more fluctuation), the depth of the dowels, as measured from the surface, would vary directly with the fluctuation of the slab thickness. The placement of dowels by the inserter is referenced to the paver frame, and is thus more closely correlated with the pavement surface. Thus, fluctuation in pavement thickness should have little or no effect on dowel depths for the inserter projects. In conclusion, the dowel bar inserter is capable of placing dowels in a satisfactory close distribution around a target depth.

Second, the average placement depth of the dowels was examined. The mean depth of placement for all of the inserter and basket projects was below the target depth specification. However, all of the projects except two had had at least 90% of the dowels placed within the allowable range of the depth specification. On one inserter project and one basket project, a significant percentage of the dowels were placed too deep to meet the maximum depth specification.

On the South Madison Beltline project, where the dowel bar inserter was used, 39% of the sampled dowels were placed too deep to meet the maximum depth specification. Since the dowel bar inserter is attached to the frame of the paver, dowel placement is therefore referenced to the top of the slab, as discussed previously. Thus, placement depth, as measured from the top of the slab, should not be influenced by fluctuation in overall pavement thickness. It is then very probable that the inserter was set up incorrectly at the start of the project, and that the dowels were consistently placed too deep throughout the project. The target depth for the dowels was 0.8 inches higher than the mean depth measured. In conclusion, great care must be taken to set up the inserter for proper depth placement at the start of each project, and the setup should be verified by checking dowel depths in the fresh concrete during the early stages of paving.

On the West Madison Beltline project, where dowel baskets were used, 39% of the sampled dowels were placed too deep to meet the maximum depth specification. Since the dowel baskets are staked to the base course, dowel placement is therefore referenced to the bottom of the slab, as discussed previously. Thus, placement depth, as measured from the top of the slab, would directly reflect fluctuations in overall slab thickness. The nominal slab thickness for this project, which was used for the analysis, was 10 inches. Based upon the results of the pavement thickness quality control coring which was performed on the project, the average actual slab thickness was 10.6 inches in the southbound lanes, where the dowel test sections were located. The target depth for the dowels was 0.9 inches higher than the mean depth measured. This difference is only slightly greater than the additional slab thickness measured. Also, if this project had been constructed under the 1988 specifications, the target dowel depth would have been 1/2 inch deeper, and only 6% of the dowels would have been too deep, based on nominal slab thickness. If the analysis were based on actual slab thickness, the out of spec figure would have been reduced even further. In conclusion, while the depth of concrete cover over the dowels fluctuates with pavement thickness on basket projects, the depth of concrete cover below the dowels remains predominantly consistent and adequate.

Vertical Rotation: At least 90% of the dowels were placed within the specification for vertical rotation on all inserter and basket projects. The mean rotation varied from 0.14 to 0.25 inches on the individual projects. In conclusion, the dowel bar inserter performed satisfactorily with respect to the vertical rotation parameter.

Horizontal Rotation: At least 90% of the dowels were placed within the specification for horizontal rotation on all of the inserter projects. Somewhat surprisingly, 17% to 22% of the dowels on the three basket projects did not meet the specified limit of 1/2 inch of horizontal rotation. The mean horizontal rotation for the inserter projects ranged from 0.21 to 0.26 inches, while the mean for the basket projects ranged ranged from 0.32 to 0.40 inches. In conclusion, the dowel bar inserter performed satisfactorily with respect to the horizontal rotation parameter, and was superior to basket performance.

Longitudinal Translation: This parameter is more closely associated with the marking and sawing of joints than with the actual performance of the dowel bar inserter itself. However, whether placement is by inserter or baskets, it is imperative that the sawed joint is aligned properly with the midpoint of the dowels. The number of joints which were improperly aligned with the dowels ranged from 1% to 15% on the inserter projects, and from 1% to 22% on the basket projects. Hence, there is definitely room for improvement of the joint locating techniques used both on inserter and basket projects. An economical means by which to improve performance in this area would be to have available a magnetic rebar locator on all doweled PCC construction projects. This could be used to verify the location of the dowels relative to the pre-established locating marks, especially in the early stage of paving on a project, until the joint marking and sawing procedure is refined to an acceptable level. It was the experience of the field crew doing the coring for this study, that the position of the dowel ends could be accurately established (plus or minus 1 inch) using a magnetic rebar locator. In conclusion, performance with respect to longitudinal translation needs improvement on both inserter and basket projects.

Ride Quality: Concern has been expressed over adverse effects on the ride quality of pavements where dowel bar inserters are used. Pavement serviceability index (PSI) is measured on all newly constructed pavements in Wisconsin to assess the ride quality of these projects. These ride quality measurements are collected after diamond grinding has been completed to meet the California Profilograph based WisDOT smoothness specifications which are part of the construction contract. The total amount of original roughness on a project can then be qualitatively assessed as a combination of the final PSI, and the amount of grinding which was done to achieve that level of ride quality. It is important to remember that some or most of the roughness on a project may originate from paving problems which are independent of the dowel bar inserter.

On the first inserter project, I-90 at Janesville, an extensive amount of grinding was performed, and the final project PSI was only 3.6. Field inspection of this project revealed that, especially in the southern portion of the project, some of the existing roughness and grinding was related to the dowel bar inserter. In some unground stretches, cyclical distortion of the longitudinal profile was visually evident in synchronization with the pavement joints. In some ground stretches, cyclical variation of the depth of grinding was visually evident in synchronization with the pavement joints. However, the overall majority of roughness and grinding on the project appeared to be related to longer wavelength profile distortion, which is typically associated with other independent paving problems.

On the second inserter project, USH 18/151 from Dodgeville to Mt. Horeb, a moderate amount of grinding was performed, and the final project PSI was 4.2. Field inspection of this project revealed that no significant amount of roughness or grinding was synchronized with the pavement joints. The roughness and grinding on this project appeared to be all related to longer wavelength profile distortion, associated with other independent paving problems. Some roughness on this project may have been caused by construction problems with the experimental open-graded base course which was used on portions of the project.

On the third inserter project, the South Madison Beltline, a relatively light amount of grinding was performed, and the final project PSI was 4.6. With a PSI that high and the small amount of grinding, there was not much initial roughness built into this project. Field inspection revealed no significant profile distortion associated with the joints. The limited grinding which was done appeared to be related to longer wavelength profile distortion, independent of the joints.

All three of these projects were paved by the same contractor. It is evident that the contractor's ability to produce a smooth riding pavement with this paver/dowel inserter combination has improved dramatically. In conclusion, by the third project, satisfactory performance was achieved with the dowel inserter with respect to ride quality.

Voids: Another concern about the dowel inserter has been the quality of consolidation of the concrete around the dowels when they are inserted. Significant voids (dime-size or larger - small bugholes were not counted) were found immediately above the ends of 22% to 34% of the dowels on the inserter projects. The voids were always very close to the ends of the dowel, within about the last inch of the bar. On two of the basket projects, no voids were found above the dowels, but on the STH 29 Vinton project voids were found above 40% of the dowels.

With the inserter, the concrete flows upward past the dowel as the dowel is vibrated downward, and the logical location for voids is above the dowel, so it is likely that all voids were detected on the inserter projects. However, on the basket projects the concrete flows downward past the dowel, and the logical location for voids is underneath the dowel. Since on_ the top of the dowel was inspected for this study, it is possible that the void problem may be understated for the basket projects due to undetected voids beneath the dowels. Further coring needs to be performed on basket projects to assess the extent of this problem.

Voids of this magnitude could affect the load transfer capacity of the dowels, so improvement in this area is needed. The amount of vibration used on the inserter needs to be increased slightly, but caution must be exercised to avoid using excessive vibration which may damage the concrete. In conclusion, quality control coring should be performed on inserter and basket projects constructed in the near future to assess the progress in solving this problem.

Missing Dowels: Another problem unique to the dowel bar inserter is that sometimes dowel bars are missing completely. This is difficult to inspect for, when the dowels are immediately buried in the concrete, instead of being laid out on the grade ahead of the paver. At one test joint on the USH 18/151 project, all of the dowels for the joint were missing. Using the magnetic rebar locator, it was determined that all of the dowels were present in the two adjacent joints, but the 24 missing dowels were not found anywhere between the adjacent joints. On the South Madison Beltline project, three dowels were missing from one test joi and one dowel was missing from another test joint. The three missing dowels were located at the edge of the pavement on the opposite side of the road from where the inserter distribution carriage is loaded. In that instance, it is likely that an insufficient number of dowel bars were loaded into the distribution carriage for that joint. The single missing dowel was located in the first position on the same side of the road where the distribution carriage is loaded. In that instance, it is possible that the missing dowel resulted from a misfeed or jam of the distribution system.

If every joint on the three inserter projects in this study was to be checked with a rebar locator, it is doubtless that additional occurrences of missing dowels would be identified, but the extent of the problem is currently not known. However, if the frequency of missing dowels noted at the test joints is an accurate indicator, then the incidence of missing dowels is probably relatively rare and isolated. This issue presents another good justification for having a magnetic rebar locator available on future inserter projects. The paving inspector cannot possibly observe the performance of the inserter on every joint. It would be good practice to make random checks with the rebar locator to verify dowel presence, and to make more extensive searches if a problem is suspected. If a significant number of bars was determined to be missing, payment penalties could be assessed by this type of survey. In conclusion, missing dowels do not appear to represent a widespread problem on the inserter projects in this study, but should still be monitored on future projects.

<u>Other Brands of Dowel Bar Inserters</u>: All of the data and conclusions in this study are valid only for the Guntert & Zimmerman dowel bar inserter used on the projects in this study. If a different brand of dowel bar inserter is used which differs greatly in design and operation from the Guntert & Zimmerman model, a thorough performance evaluation of the new machine will be essential. Performance with respect to any or all of the placement parameters discussed in this analysis could be widely different for a different machine.

SUMMARY AND CONCLUSIONS

With any new form of technology, there will always be some problems that need to be resolved during the initial learning period. The dowel bar inserter is no exception to this rule, and several problems have been identified with its performance on the projects in this study. However, none of these problems appear to be insurmountable. Through the continued cooperative efforts of WisDOT construction personnel and the contractors, it should be possible to improve construction procedures to obtain consistent satisfactory results with the dowel bar inserter.

The primary general recommendation of this study is to accept the dowel bar inserter as an equal alternate to dowel baskets for future WisDOT doweled PCC construction projects. The following list of specific conclusions and recommendations are based on the results of this study.

1. The dowel bar inserter is capable of consistent satisfactory placement of dowel bars with respect to vertical translation (average depth), vertical rotation (difference in depth between two ends of dowel), and horizontal rotation (difference in transverse position between two ends of dowel).

2. The initial set-up of the dowel bar inserter with respect to depth of dowel placement is critical at the start of each project, and dowel depths should be verified by probing through the fresh concrete.

3. The construction procedures currently used for marking and sawing joints need improvement both for inserter and basket projects, to consistently and accurately align the sawn joints with the midpoints of the dowel bars.

4. Having a magnetic rebar locator available on all doweled PCC construction projects would be useful in aligning sawn joints with the dowel bars and in identifying missing dowels.

5. Ride quality has improved on each successive inserter project, and on the latest project, the South Madison Beltline, a project PSI of 4.6 was achieved with minimal diamond grinding.

6. Improved concrete consolidation around the dowels is needed both on inserter and basket projects, and quality control coring is needed to assess future progress in solving the problem of voids around the dowel bars.

7. Problems with missing dowel bars on existing inserter projects appear to be infrequent and isolated, but this problem should be monitored on future projects.

8. All of the data and conclusions in this study are valid only for the Guntert & Zimmerman dowel bar inserter used on the projects in this study. If a different brand of dowel bar inserter is used which differs widely in design and operation from the Guntert & Zimmerman model, a thorough performance evaluation will be essential.

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1. Ray, Gordon K., "Dowel Inserters", Roads and Bridges, April 1988, page 19.

2. Okamoto, Paul A., "Field Evaluation of Dowel Placement Along a Section of I-90 Near Janesville, Wisconsin - Final Report", Construction Technology Laboratories, Inc., for Wisconsin Department of Transportation, August 1988.

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TABLE 1. DESCRIPTION OF PROJECTS INCLUDED IN DOWEL BAR PLACEMENT STUDY

PROJECT LOCATION DESCRIPTION	DISTRICT	STATE PROJECT NUMBER	CONSTRUCTION YEAR	PAVING CONTRACTOR	DOWEL PLACEMENT TECHNIQUE	PAVEMENT THICKNESS	BASE COURSE
I-90 in Rock County (Westbound Lanes Only) Madison - Illinois State Line Road (Manogue Road - USH 14 at Janesville)	1	1001-01-75	1987	James Cape & Sons, Inc.	Inserter	10"	8" Thick Dense Graded
USH 18/151 in Dane County (Eastbound Lanes Only) Dodgeville - Mt. Horeb Road (West County Line - CTH "PD")	1	1204-04-72	1988	James Cape & Sons, Inc.	Inserter	9"	Varied
USH 12/18 in Dane County South Madison Beltline (I-90 - South Towne Drive)	1	1206-02-79	1988	James Cape & Sons, Inc.	Inserter	10"	6" Thick Dense Graded
USH 12/14 in Dane County West Madison Beltline (Old Sauk Interchange)	1	5303-00-71	1987	Trierweiler Construction and Supply, Inc.	Baskets	10"	6" Thick Dense Graded
STH 29/32 in Brown, Shawano, & Outagamie Counties Shawano - Green Bay Road (STH 156 - CTH "U")	3	9202-02-76	1988	Streu Construction Co.	Baskets	10"	4" Thick Open Graded Over 4" Dense
STH 29/32 in Brown County Shawano - Green Bay Road (CTH "U" - USH 41)	3	9202-02-77	1988	Vinton Construction Co.	Baskets	10"	4" Thick Open Graded Over 4" Dense

TABLE 2. LOCATION OF TEST SECTIONS ON PROJECTS INCLUDED IN DOWEL BAR PLACEMENT STUDY

PROJECT LOCATION DESCRIPTION	DISTRICT	STATE PROJECT NUMBER	DIRECTION OF TESTING	TEST SECTION NUMBER	STATIONS
I-90 in Rock County (Westbound Lanes Only) Madison - Illinois State Line Road (Manogue Road - USH 14 at Janesville)	1	1001-01-75	Westbound	1 2 3	559+88 to 553+38 509+27 to 503+36 464+87 to 458+28
USH 18/151 in Dane County (Eastbound Lanes Only) Dodgeville - Mt. Horeb Road (West County Line - CTH "PD")	1	1204-04-72	Eastbound	1 2 3 4	961+37 to 968+01 1014+13 to 1020+70 1066+17 to 1072+73 1232+17 to 1238+73
USH 12/18 in Dane County South Madison Beltline (I-90 - South Towne Drive)	1	1206-02-79	Westbound	1 2 3	191+87 to 185+20 154+79 to 148+27 64+93 to 58+21
USH 12/14 in Dane County West Madison Beltline (Old Sauk Interchange)	1	5303-00-71	Eastbound	1 2 3	100+17 to 106+84 125+07 to 131+72 138+00 to 144+72
STH 29/32 in Brown, Shawano, & Outagamie Counties Shawano - Green Bay Road (STH 156 - CTH "U")	3	9202-02-76	Westbound	1 2 3	484+91 to 478+45 419+80 to 413+17 369+90 to 363+30
STH 29/32 in Brown County Shawano - Green Bay Road (CTH "U" - USH 41)	3	9202-02-77	Eastbound	1 2 3	675+19 to 682+64 725+20 to 731+76 760+20 to 766+72

TABLE 3. RANDOM SAMPLING SEQUENCE FOR LATERAL POSITION OF DOWELS FOR DOWEL BAR PLACEMENT STUDY

SEQUENCE FOR 12-FOOT LANE	SEQUENCE FOR 14-FOOT LANE
DOWEL BAR NUMBER	DOWEL BAR NUMBER
2	2
1	1
8	8
7	7
9	9
12	12
6	6
10	10
3	3
5	5
4	4
11	13
	14
	11

TABLE 4. SUMMARY OF CORING DATA FOR DOWEL BAR PLACEMENT STUDY

TEST PARAMETERS (Note: Dimensions for all measurements are in inches.)		PROJECT DESCRIPTIONS AND DATA					
		DOWEL BAR INSERTER			DOWEL BASKETS		
		1001-01-75 I-90 ROCK CO. PAVED 1987	1206-02-79 USH 12/18 DANE CO. PAVED 1988	1204-04-72 USH 18/151 DANE CO. PAVED 1988	5303-00-71 USH 12/14 DANE CO. PAVED 1987	9202-02-76 STH 29/32 BROWN CO. PAVED 1988	9202-02-7 STH 29/32 BROWN CO. PAVED 198
NUMBER OF DOWELS TESTED		84	90	120	90	90	9
VERTICAL TRANSLATION (Average depth of dowel)	MEAN	4.31	5.22	4.49	4.79	4.59	4.1
	STANDARD DEVIATION	0.46	0.38	0.20	0.42	0.57	0.3
	MINIMUM VALUE OBSERVED	3.34	3.97	3.97	3.50	2.81	2.8
	MAXIMUM VALUE OBSERVED	5.56	6.06	4.88	5.81	5.84	5.0
	RANGE	2.22	2.09	0.91	2.31	3.03	2.1
	DEPTH SPECIFICATIONS	3.875+-1in.	4.375+-1in.	3.938+-1in.	3.875+-1in.	4.375+-1in.	4.375+-1i
	PERCENT OF OBSERVATIONS EXCEEDING SPECIFICATION						
	TOO SHALLOW	0%	0%	2%	0%	3%	[illegible]
	TOO DEEP	10%	39%	0%	39%	7%	[illegible]
VERTICAL ROTATION (Difference in depth between opposite ends of dowel)	MEAN	0.25	0.19	0.15	0.16	0.17	0.1
	STANDARD DEVIATION	0.21	0.14	0.13	0.33	0.18	0.1
	MINIMUM VALUE OBSERVED	0.00	0.00	0.00	0.00	0.00	0.0
	MAXIMUM VALUE OBSERVED	1.13	0.75	0.63	3.00	1.00	0.6
	PERCENT OF OBSERVATIONS EXCEEDING SPECIFICATION (diff. depth > 0.5 in.)	10%	3%	5%	6%	4%	
HORIZONTAL ROTATION (Difference in transverse position between opposite ends of dowel)	MEAN	0.26	0.25	0.21	0.36	0.40	0.3
	STANDARD DEVIATION	0.23	0.31	0.21	0.31	0.32	0.3
	MINIMUM VALUE OBSERVED	0.00	0.00	0.00	0.00	0.00	0.0
	MAXIMUM VALUE OBSERVED	1.31	2.00	1.00	1.44	1.81	1.6
	PERCENT OF OBSERVATIONS EXCEEDING SPECIFICATION (diff. trans.> 0.5 in.)	10%	8%	8%	22%	26%	17
LONGITUDINAL TRANSLATION (Longitudinal offset between sawed joint and midpoint of dowel)	MEAN	1.62	0.87	1.86	2.12	1.66	0.88
	STANDARD DEVIATION	1.31	0.65	2.29	1.91	1.50	0.74
	MINIMUM VALUE OBSERVED	0.00	0.00	0.00	0.00	0.00	0.00
	MAXIMUM VALUE OBSERVED	6.88	3.31	17.31	9.00	7.75	3.81
	PERCENT OF OBSERVATIONS EXCEEDING SPECIFICATION (offset > 3.0 in.)	8%	1%	15%	22%	20%	1
PERCENT OF DOWELS WHICH HAD A SIGNIFICANT VOID OVER AT LEAST ONE END OF THE DOWEL		26%	34%	22%	0%	0%	40

Note: Depth was measured from pavement surface to top of dowel end.
Transverse position was measured from edge of lane to center of dowel end.
Longitudinal position was measured from center of sawed joint to dowel end.

FIGURE 1. CONFIGURATION AND NUMBERING OF DOWELS FOR A TYPICAL ROADWAY SECTION

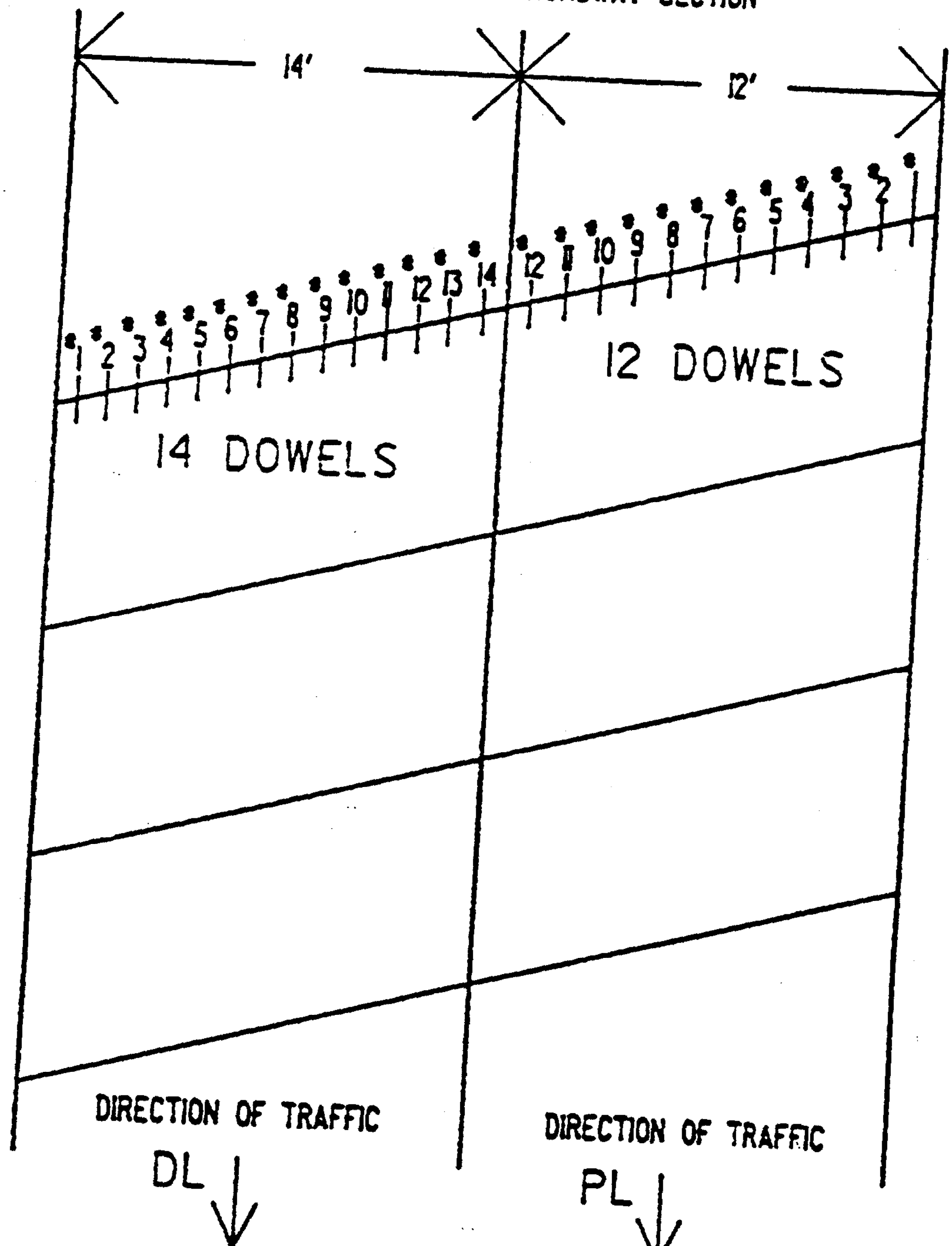
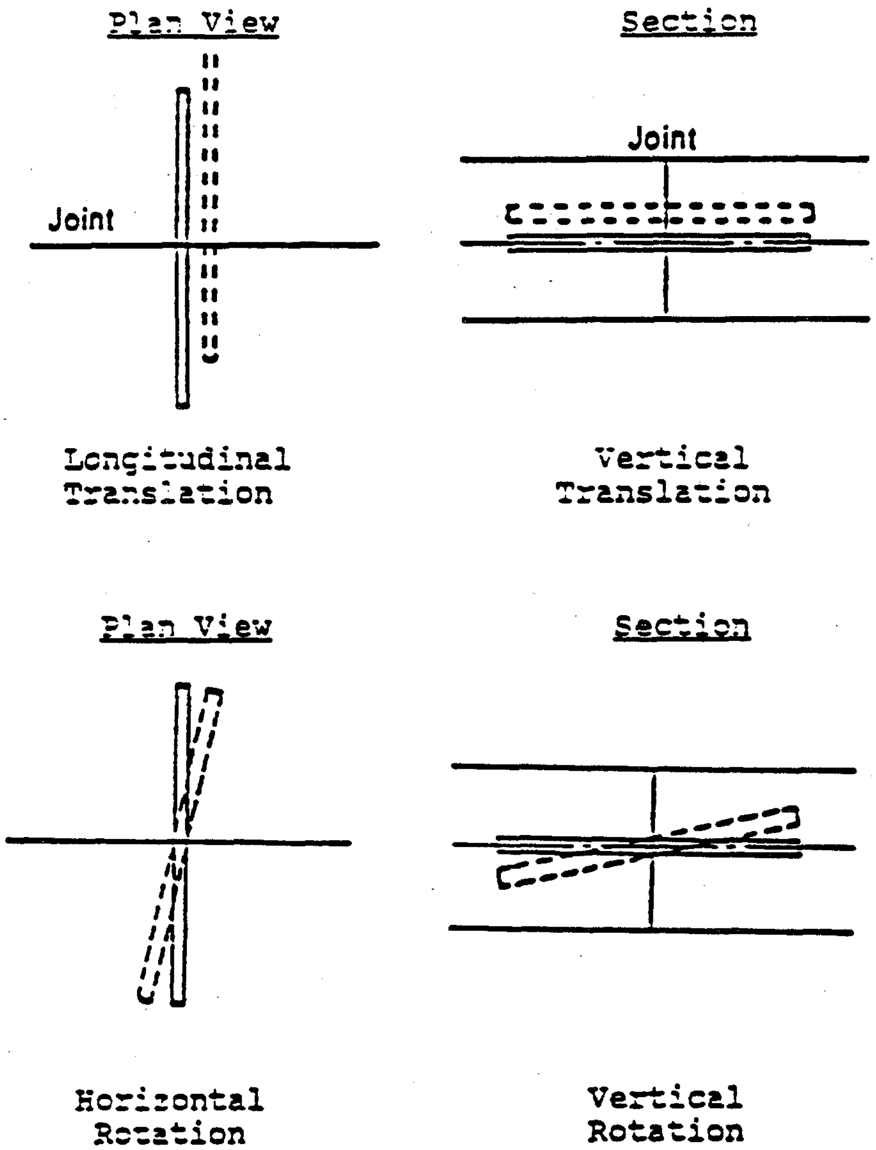


FIGURE 2: Illustration of Dowel Placement Parameters



APPENDIX A. DETAILED DATA FOR DOWEL PLACEMENT STUDY

Key for Appendix A

Variable Name		Description
BARNUMBR	=	Lateral position of dowel in lane
DIRECTON	=	Direction of traffic
LANE	=	Driving lane (1) or passing lane(2)
STATION	=	Station of test joint
VOID	=	Severity rating for voids at ends of dowel N = none S = small (less than 1/4") M = medium (1/4" to 1/2") L = large (greater than 1/2")
DEPTH1	=	Vertical position of upstream end of dowel (upstream with respect to traffic direction)
DEPTH2	=	Vertical position of downstream end of dowel
TRANS1	=	Transverse position of upstream end of dowel
TRANS2	=	Transverse position of downstream end of dowel
LONG1	=	Longitudinal position of upstream end of dowel
LONG2	=	Longitudinal position of downstream end of dowel
AVGDEPTH	=	Average depth of dowel
DIFDEPTH	=	Difference in depth between two ends of dowel
DIFTRANS	=	Difference in transverse position between two ends of dowel
LOFFSET	=	Longitudinal offset from midpoint of dowel to sawn joint

(Counter variables: yes = 1 and no = 0)

AVGDEPSM	=	Average depth of dowel too shallow for spec
AVGDEPLG	=	Average depth of dowel too deep for spec
DIFDEPLG	=	Difference in depth too large for spec
DIFTRNLG	=	Difference in transverse position too large for spec
OFFSETLG	=	Longitudinal offset too large for spec
VOIDNONE	=	Void severity rating is "none"
VOIDSMAL	=	Void severity rating is "small"
VOIDMEDM	=	Void severity rating is "medium"
VOIDLARG	=	Void severity rating is "large"

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

SUBJECT	FHWA TECHNICAL ADVISORY
CONTINUOUSLY REINFORCED CONCRETE PAVEMENT	T 5080.14 June 5, 1990

1. PURPOSE. To outline recommended practices for the design, construction, and repair of continuously reinforced concrete pavement (CRCP).

2. CANCELLATION. Technical Advisory T 5080.5, Continuously Reinforced Pavement, dated October 14, 1981, is cancelled.

3. BACKGROUND

a. Continuously Reinforced Concrete Pavement is a portland cement concrete (PCC) pavement that has continuous longitudinal steel reinforcement and no intermediate transverse expansion or contraction joints. The pavement is allowed to crack in a random transverse cracking pattern and the cracks are held tightly together by the continuous steel reinforcement.

b. During the 1970's and early 1980's, CRCP design thickness was approximately 80 percent of the thickness of conventional jointed concrete pavement. A substantial number of the thinner pavements developed distress sooner than anticipated.

c. Attention to design and construction quality control of CRCP is critical. A lack of attention to design and construction details has caused premature failures in some CRCPs. The causes of early distress have usually been traced to; (1) construction practices which resulted in pavements which did not meet design requirements; (2) designs which resulted in excessive deflections under heavy loads; (3) bases of inferior quality, or; (4) combinations of these or other undesirable factors.

DISTRIBUTION: Level 1: Headquarters(OA,PL,RD,HI,RE,SA, ED,AD)
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Level 3: Divisions(SH)

OPI: HHO-12

4. DESIGN RECOMMENDATIONS

a. Concrete Thickness. Generally the slab thickness is the same as the thickness of a jointed concrete pavement unless local performance has shown thinner pavements designed with an accepted design process to be satisfactory.

b. Reinforcing Steel

(1) Longitudinal Steel

(a) A minimum of 0.6 percent (based on the pavement cross sectional area) is recommended to aid transverse crack development in the range of 8 feet, maximum, and 3.5 feet, minimum, between cracks. Exceptions should be made only where experience has shown that a lower percentage of steel has performed satisfactorily. In areas where periods of extreme low temperature (average minimum monthly temperatures of 10° F or less) occur, the use of a minimum of 0.7 percent steel is recommended.

(b) Deformed steel bars that meet the requirements set out in AASHTO Specifications, Part I, AASHTO M31, M42, or M53 are recommended. The tensile requirements should conform to the American Society for Testing and Materials (ASTM) Grade 60. Recommended spacing of the longitudinal steel is not less than 4 inches or 2 1/2 times the maximum sized aggregate, whichever is greater, and not greater than 9 inches. A minimum ratio of 0.03 square inches of steel bond area per cubic inch of concrete is recommended. See Attachment 1 for an example problem for determining the minimum longitudinal steel spacing and the minimum bond ratio. Table 1 shows the minimum and maximum bar sizes for given pavement thicknesses and reinforcement percentages. These bar sizes meet the minimum bond ratio and the minimum bar spacing criteria stated above.

(c) The recommended position of the longitudinal steel is between 1/3 and 1/2 of the depth of the pavement as measured from the surface. The minimum concrete cover should be 2-1/2 inches with 3 inches preferable. For pavements thicker than 11 inches, several States have begun to experiment with the use of two layers of longitudinal steel. Pavements constructed with two layers of steel have not been in service long enough to evaluate performance; therefore, this technique should be considered experimental.

Table 1 - Recommended Longitudinal Reinforcement Sizes

	Minimum and Maximum Bar Size					
	Pavement Thickness					
% Steel	8"	9"	10"	11"	12"	13"
0.60	4,5	5,6	5,6	5,6	5,6	6
0.62	5,6	5,6	5,6	5,6	5,6	6
0.64	5,6	5,6	5,7	5,7	6,7	6,7
0.66	5,6	5,7	5,7	5,7	6,7	6,7
0.68	5,6	5,7	5,7	6,7	6,7	6,7

Note: Bars are uncoated deformed bars.

(d) The use of epoxy coated reinforcing steel is generally not necessary for CRCP. However, in areas where corrosion is a problem because of heavy applications of deicing salts or severe salt exposure, epoxy coating of the steel may be warranted. The bond area should be increased 15 percent to increase the bond strength between the concrete and reinforcement if epoxy-coated steel reinforcement is used.

(e) When splicing longitudinal steel, the recommended minimum lap is 25 bar diameters with the splice pattern being either staggered or skewed. If a staggered splice pattern is used, not more than one-third of the bars should terminate in the same transverse plane and the minimum distance between staggers should be 4 feet. If a skewed splice pattern is used, the skew should be at least 30 degrees from perpendicular to the centerline. When using epoxy-coated steel, the lap should be increased a minimum of 15 percent to ensure sufficient bond strength.

(f) Plan details or specifications are needed to insure sufficient reinforcing at points of discontinuity as described in paragraphs 4e(3) and 4f(1).

(2) Transverse Reinforcing and Tiebars

(a) If transverse reinforcement is included, it should be #4, #5, or #6 grade 60 deformed bars meeting the same specifications as mentioned for the longitudinal reinforcement.

(b) Although it can be omitted, transverse reinforcing reduces the risk of random longitudinal cracks opening up and thus reduces the potential of punch-outs. If transverse reinforcement is included, the following equation can be used to determine the amount of reinforcement required (see number 5 of Attachment 2):

$$P_t = \frac{W_s F}{2f_s} \times 100$$

Where:
- P_t = transverse steel, %
- W_s = total pavement width, (ft)
- F = subbase friction factor
- f_s = allowable working stress in steel, psi, (0.75 yield strength)

(c) The spacing between transverse reinforcing bars can be calculated using the following equation (see numbers 1 and 5 of Attachment 2):

$$Y = \frac{A_s}{P_t D} \times 100$$

Where:
- Y = transverse steel spacing (in)
- A_s = cross-sectional area of steel, (in^2) per bar (#4, #5, or #6 bar)
- P_t = percent transverse steel
- D = slab thickness (in)

Note: The transverse bar spacing should be no closer than 36 inches and no further than 60 inches.

(d) In cases where transverse steel is omitted, tiebars should be placed in longitudinal joints in accordance with the FHWA Technical Advisory, Concrete Pavement Joints.

c. Bases

(1) The base design should provide a stable foundation, which is critical for CRCP construction operations and should not trap free moisture beneath the pavement. Positive drainage

is recommended. Free moisture in a base or subgrade can lead to slab edge-pumping, which has been identified as one of the major contributors to causing or accelerating pavement distress. Bases that will resist erosion from high water pressures induced from pavement deflections under traffic loads, or that are free draining to prevent free moisture beneath the pavement will act to prevent pumping. Stabilized permeable bases should be considered for heavily traveled routes. Pavements constructed over stabilized or crushed stone bases have generally resulted in better performing pavements than those constructed on unstabilized gravel.

(2) The friction between the pavement and base plays a role in the development of crack spacing in CRCP. Most design methods for CRCP assume a moderate level of pavement/base friction. Polyethylene sheeting should not be used as a bond breaker unless the low pavement/base friction is considered in design. Also, States have reported rideability and construction problems when PCC was constructed on polyethylene sheeting.

d. Subgrades. Continuously Reinforced Concrete Pavement is not recommended in areas where subgrade distortion is expected because of known expansive soils, frost heave, or settlement areas. Emphasis should be placed on obtaining uniform and adequately compacted subgrades. Subgrade treatment may be warranted for poor soil conditions.

e. Joints

(1) Longitudinal Joints. Longitudinal joints are necessary to relieve stresses caused by concrete shrinkage and temperature differentials in a controlled manner and should be included when pavement widths are greater than 14 feet. Pavements greater than 14 feet wide are susceptible to longitudinal cracking. The joint should be constructed by sawing to a depth of one-third the pavement thickness. Adjacent slabs should be tied together by tiebars or transverse steel to prevent lane separation. Tiebar design is discussed in the FHWA Technical Advisory entitled "Concrete Pavement Joints".

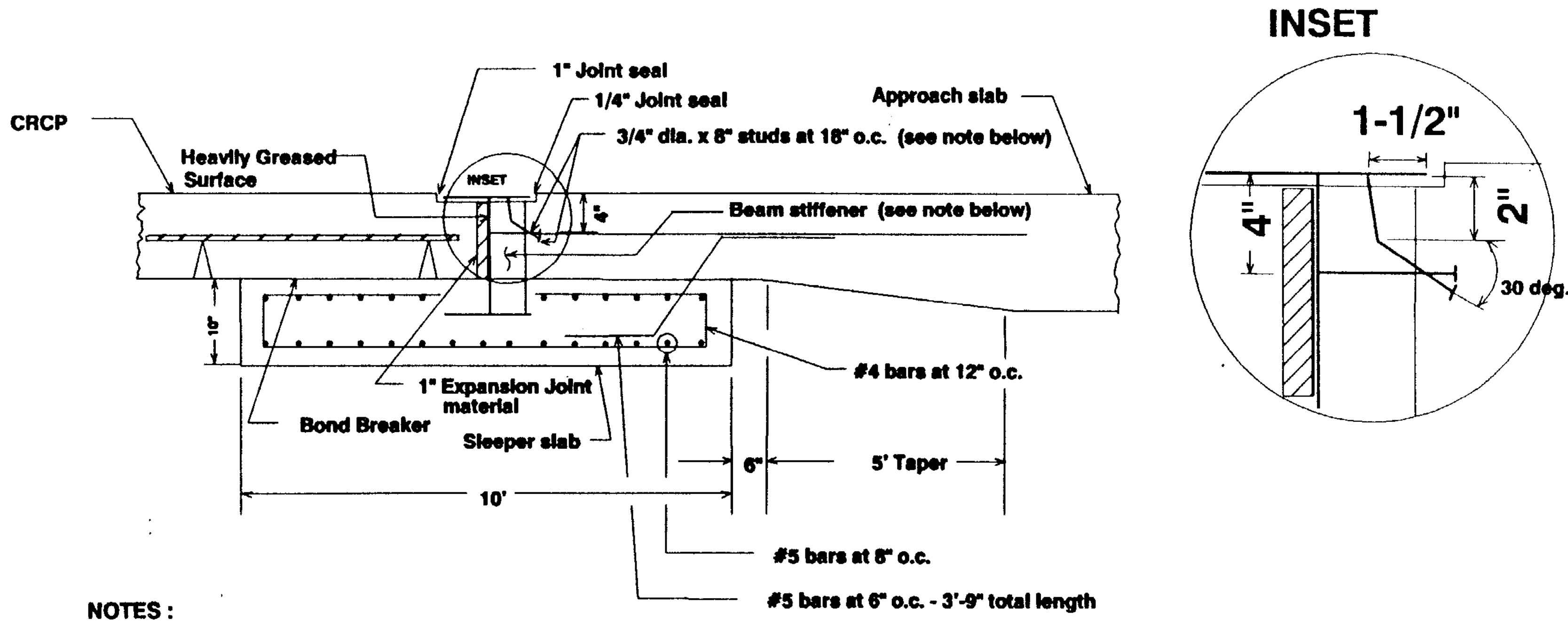
(2) Terminal Joints. The most commonly used terminal treatments are the wide-flange (WF) steel beam which accommodates movement, and the lug anchor which restricts movement.

(a) The WF beam joint consists of a WF beam partially set into a reinforced concrete sleeper slab approximately 10 feet long and 10 inches thick. The top flange of the beam is flush with the pavement surface. Expansion material, sized to accommodate end movements, is placed on one side of the beam along with a bond-breaker between the pavement and the sleeper slab. In highly corrosive areas the beam should be treated with a corrosion inhibitor. Several States have reported premature failures of WF beams where the top flange separated from the beam web. Stud connectors should be welded to the top flange, as shown in Figure 1, to prevent this type of failure. Table 2 and Figure 1 contain recommended design features.

TABLE 2 - Recommended WF Beam Dimensions

WF Beam (weight and dimensions)					
CRCP Thickness (in.)	Embedment in "Sleeper" slab - in.	WF Beam Size	Flange		Web Thickness (in.)
			Width (in.)	Thickness	
8 9	6 5	14 x 61	10	5/8	3/8
10 11	6 5	16 x 58	8-1/2	5/8	7/16

(b) The lug anchor terminal treatment generally consists of three to five heavily reinforced rectangularly shaped transverse concrete lugs placed in the subgrade to a depth below frost penetration prior to the placement of the pavement. They are tied to the pavement with reinforcing steel. Since lug anchors restrict approximately 50 percent of the end movement of the pavement an expansion joint is usually needed at a bridge approach. A slight undulation of the pavement surface is sometimes induced by the torsional forces at the lug. Since this treatment relies on the passive resistance of the soil, it is not effective where cohesionless soils are encountered. Figure 2 shows a typical lug anchor terminal treatment.



NOTES :

Weld beam stiffener to ends of beams
Weld shear connectors to flange and web of beam

FIGURE 1 - Recommended WF Steel Beam Terminal Joint Design

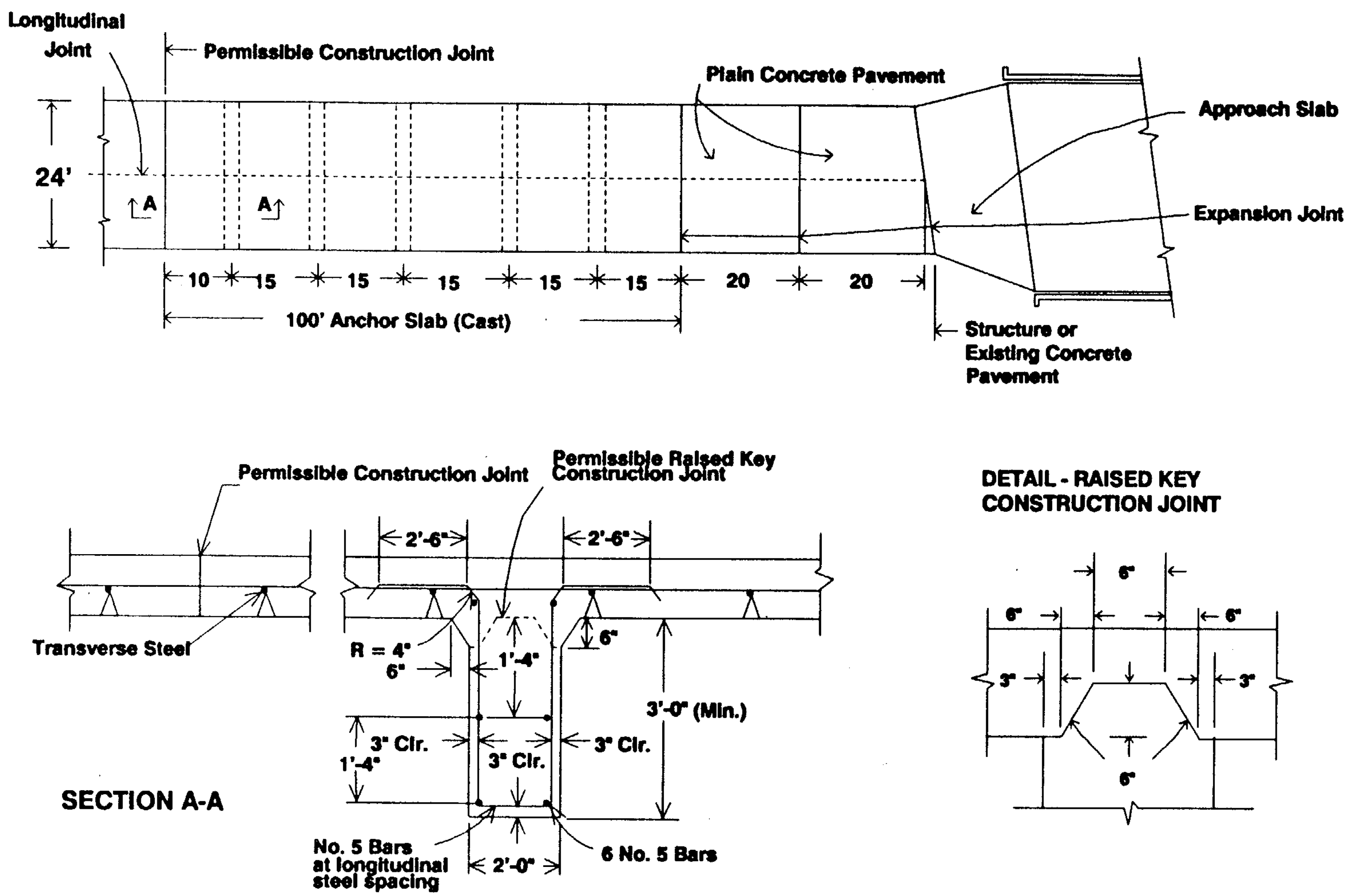


FIGURE 2 - Lug Anchor Treatment

(3) Transverse Construction Joints

(a) A construction joint is formed by placing a slotted headerboard across the pavement to allow the longitudinal steel to pass through the joint. The longitudinal steel through the construction joint is increased a minimum of one-third by placing 3-foot long shear bars of the same nominal size between every other pair of longitudinal bars. No longitudinal steel splice should fall within 3 feet of the stopping side nor closer than 8 feet from the starting side of a construction joint. Refer to paragraph 4b(1)(e) for recommended splicing patterns. If it becomes necessary to splice within the above limits, each splice should be reinforced with a 6-foot bar of equal size. Extra care is needed to ensure both concrete quality and consolidation at these joints. If more than 5 days elapse between concrete pours, the adjacent pavement temperature should be stabilized by placing insulation material on it for a distance of 200 feet from the free end at least 72 hours prior to placing new concrete. This procedure should reduce potentially high tensile stresses in the longitudinal steel.

(b) Special provisions for the protection of the headerboard and adjacent rebar during construction may be necessary.

f. <u>Leave-Outs</u> Temporary gaps in CRCPs should be avoided. The necessity for leave-outs is minimized by giving proper consideration to the paving schedule during project design. The following precautions can be specified to reduce distress in the leave-out portion of the slab in the event a leave-out does become necessary.

(1) Leave-outs require 50 percent more longitudinal deformed bars of the same nominal size as the regular reinforcement. The additional reinforcement should be spaced evenly between every other normal pavement reinforcing bar and should be bonded at least 3 feet into the pavement ends adjacent to the leave-outs. All regular longitudinal reinforcement should extend into the leave-out a minimum of 8 feet. Required slices should be made the same as those in normal construction.

(2) Leave-outs should be paved during stable weather conditions when the daily temperature cycle is small. Because of the closeness of the steel extreme care should be exercised in placing and consolidating the concrete to prevent honeycombing or voids under the reinforcement

(3) If it becomes necessary to pave a leave-out in hot weather, the temperature of the concrete in the free ends should be stabilized by placing an adequate layer of insulating material on the surface of the pavement as described in paragraph 4e(3)(a). The curing compound should be applied to the new concrete in a timely manner. The insulation material should remain on the adjacent pavement until the design modulus of rupture of the leave out concrete is attained.

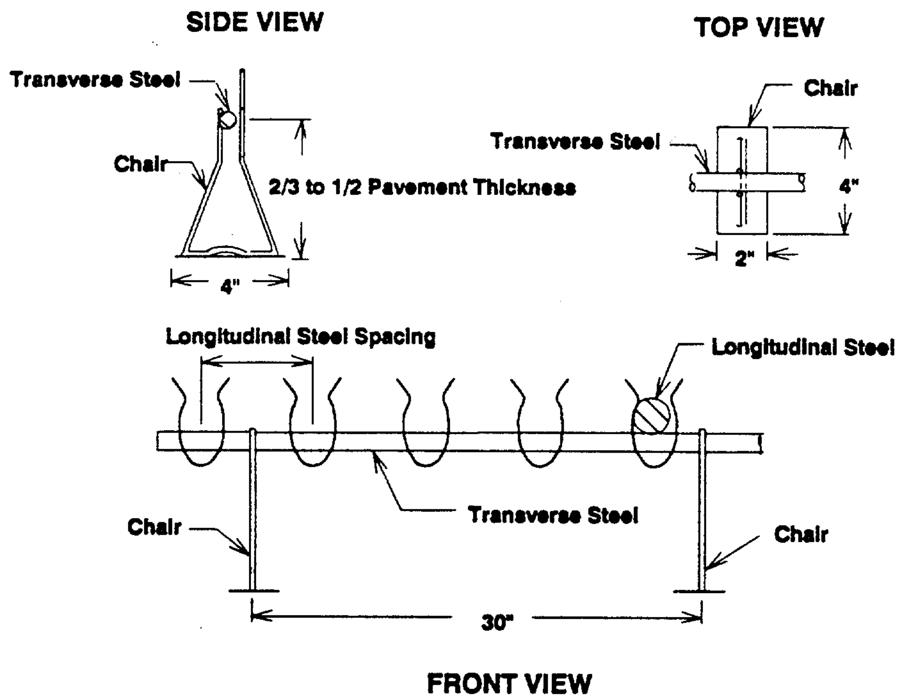
g. Ramps, Auxiliary Lanes and Shoulders. PCC pavement for ramps, auxiliary lanes, and shoulders adjacent to CRCP is recommended because of the possible reduction in pavement edge deflections and the tighter longitudinal joints adjacent to the mainline pavement. Ramps should be constructed using jointed concrete pavement. The use of jointed pavement in the ramps will accommodate movement and reduce the potential for distress in the CRCP at the ramp terminal. When PCC pavement is used for ramps, auxiliary lanes, or shoulders, the joint should be designed as any other longitudinal joint. Refer to the FHWA Technical Advisory T 5040.29, Paved Shoulders, for further information on proper joint design.

h. Widened Lanes. Widened right lane slabs should be considered to reduce or eliminate pavement edge loadings. This is discussed in the FHWA Technical Advisory T5040.29, "Paved Shoulders".

5. CONSTRUCTION CONSIDERATIONS

a. Many CRCP performance problems have been traced to construction practices which resulted in a pavement that did not meet the previously described design recommendations. Because CRCP is less forgiving and more difficult to rehabilitate than jointed pavements, greater care during construction is extremely important. Both the contractor and the inspectors should be made aware of this need and the supervision of CRCP construction should be more stringent.

b. Steel placement has a direct effect on the performance of CRCP. A number of States have found longitudinal steel placement deviations of ±3 inches in the vertical plane when tube feeders were used to position the steel. The use of chairs is recommended to hold the steel in its proper location. The chairs should be spaced such that the steel will not permanently deflect or displace to a depth of more than 1/2 the slab thickness. An example chair device is shown in Figure 3.



Note: Chairs should be securely fastened the base.

FIGURE 3 - Combination Chair and Transverse Steel Detail

c. Procedures should be implemented to ensure a uniform base and subgrade. Soft spots or gradeline variations should be repaired and corrected prior to concrete placement. Emphasis should be placed on batching, mixing, and placing concrete to obtain uniformity and quality. Strict inspection of batching and mixing procedures is extremely important and may require rejection of batches because of deviations that may have been considered minor under previously existing practices. When placing concrete, adequate vibration and consolidation must be achieved. This is especially critical in areas of pavement discontinuity such as construction or terminal joints. Automatic vibrators should be checked regularly to ensure operation at the specified frequency and amplitude and at the proper location in the plastic concrete. Hand-held vibrators should be used adjacent to transverse joints. Any concrete which exhibits signs of aggregate segregation should be replaced immediately.

d. Inspection procedures are needed to ensure that final reinforcing splice lengths and patterns, as well as bar placement, are consistent with the design requirements. Special precautions should be taken to prevent rebar bending and displacement at construction joints. When leave-outs are necessary, they should be constructed in absolute conformity to the design requirements. Longitudinal joints should be sawed as early as possible to prevent random cracking. This is especially true in multi-lane construction. Sawing should not begin until the concrete is strong enough to prevent raveling.

e. Asphalt concrete patches are not recommended as a temporary or a permanent repair technique because they break the continuity of the CRCP and provide no load transfer across the joint.

A R Kane

Anthony R. Kane
Associate Administrator for Engineering
and Program Development

Attachments

EXAMPLE PROBLEM

The design engineer should perform the following calculations to ensure that the bond between the reinforcing steel and the concrete and the longitudinal steel spacing meet the criteria in paragraph 4c. The equation to determine the ratio of bond area to cubic inches of concrete is as follows and the equation to determine the minimum longitudinal steel spacing follows it:

$$R_b = \frac{(n) \times P_s \times (L)}{(W) \times (t) \times (L)}$$

Where:
- P_s = Perimeter of Bar (in.)
- L = Length of slab = 1"
- W = Width of slab (in.)
- t = Slab thickness (in.)
- n = Number of Longitudinal Bars

Given : #6 reinforcing bars, therefore P_s = 2.356" and Bar Area = 0.44 in.^{2}
W = 12'
t = 10"

Assume: 0.6% steel

Determine: The required minimum area of steel and the required minimum number of bars

Area of Conc. = 10 x 144 = 1440 in.^{2}

Required steel = 0.006 x 1440 = 8.64 in.^{2}

Minimum number if bars required (n) = 8.64 / 0.44 = 19.6 bars, say 20 bars

Determine: The minimum ratio of bond area to cubic inches of concrete.

$$R_b = \frac{(20) \times (2.356) \times (1")}{(1440) \times (1")} = 0.0327,$$

the minimum ratio of bond area to cubic inches of concrete is met so the minimum spacing should be checked.

Determine: Longitudinal steel spacing should be checked as follows:

$$S_b = \frac{(W)}{(n)} = \frac{144}{20} = 7.2 \text{ in., say 7 in.,}$$

therefore the minimum bar spacing is also met.

REFERENCES (CRCP)

1. "AASHTO GUIDE FOR DESIGN OF PAVEMENT STRUCTURES", 1986.

2. "FHWA Pavement Rehabilitation Manual", FHWA-ED-88-025, September 1985 as supplemented.

3. Mooncheol Won, B. Frank McCullough, W. R. Hudson, Evaluation of Proposed Design Standards for CRCP, Research Report 472-1, April 1988.

4. "Techniques For Pavement Rehabilitation - A Training Course", FHWA, October 1987.

5. "Design of Continuously Reinforced Concrete for Highways", Associated Reinforcing Bar Producers - CRSI, 1981.

6. "CRCP - Design and Construction Practices of Various States", 1981, Associated Reinforcing Bar Producers - CRSI.

7. "Design, Performance, and Rehabilitation of Wide Flange Beam Terminal Joints," FHWA, Pavement Branch, February 1986.

8. Darter, Michael I., Barnett, Terry L., Morrill, David J., "Repair and Preventative Maintenance Procedures for Continuously Reinforced Concrete Pavement", FHWA/IL/UI-191, June 1981.

9. "Failure and Repair of CRCP", NCHRP, Synthesis 60, 1979.

10. Snyder, M.B., Reiter, M.J., Hall, K.T., Darter, M.I., "Rehabilitation of Concrete Pavements, Volume I - Repair Rehabilitation Techniques, Volume III - Concrete Pavement Evaluation and Rehabilitation System," FHWA-RD-88-071, July 1989.

Memorandum

U.S. Department of Transportation
Federal Highway Administration

Washington, D.C. 20590

Subject Headquarter's Pavement Rehabilitation and Design Team - Case Study - Continuously Reinforced Concrete Pavement

Date JUN 22 1987

From Chief, Pavement Division

Reply to Attn of HHO-13

To Regional Federal Highway Administrators
Regions 1-10

We have had several requests from the field for information regarding the performance of various pavements. We have elected to use a case study approach as one method of meeting this desire. The case studies will be based on the Headquarter's Pavement Rehabilitation and Design Team field trip reports. Attached is a copy of our initial effort. It describes a distress problem on a continuously reinforced concrete pavement located in the eastern part of the country. The report provides an insight to the types of details that are commonly examined. The process that was followed can be applied to any pavement rehabilitation project.

As you know, the Team at the request of State highway agencies and our field offices has conducted numerous reviews of pavement distress problems. Typically, the Team has been asked to provide assistance when a pavement has experienced premature distress. The Team's role is to determine the cause of the early distress, recommend alternative strategies for rehabilitation/reconstruction and provide suggestions on how to prevent the distress on this or similar pavement.

The Team has been headed by the Pavement Division and has included members from other Headquarter's offices depending on the technical expertise needed to examine a particular problem. From our viewpoint, there have been tremendous benefits gained by the States and FHWA using this review team concept. The Team has always been willing to provide technical assistance when requested and we reaffirm our commitment to continuing these efforts.

We expect to furnish additional case studies that will be based on upcoming Team field trip reports. If you have any comments on this case study approach or the specifics of the attached report, please do not hesitate to contact us.

Sufficient copies of the report are being provided to your office to permit direct distribution to your division offices.

Norman J. Van Ness

CASE STUDY - CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

BASED ON A

FIELD TRIP REPORT

OF THE

PAVEMENT REHABILITATION AND DESIGN TEAM

BY

PAUL TENG

JOHN HALLIN

DON VOELKER

I. Purpose of Trip

To meet with Region, Division, and State engineers to review a Continuously Reinforced Concrete Pavement (CRCP) distress problem on Route XX and discuss its rehabilitation alternatives.

II. Scope of Review

A field review was conducted on (date). During the review, the State provided the Team with available data of the original design. A closeout meeting with the State engineers was held in the afternoon of (date). Subsequently, the State provided additional information concerning traffic data, paving schedules, corrosion surveys, core logs, chlorides studies, delamination surveys, a Pachometer survey and detailed crack surveys. On (date), the Team met with the State engineers to inspect the cores and discuss our recommendations and conclusions. This report summarizes the Team's comments and recommendations.

III. Contacts

State

XXX

FHWA Division Office

XXX

FHWA Region Office

XXX

FHWA Washington Office

XXX

IV. Background Information

Route XX is a six-lane, divided Interstate facility. The project is 13 miles long and involves the overlay of two Jointed Reinforced Concrete Pavement (JRCP) 24-foot roadways with an unbonded 6-inch Continuously Reinforced Concrete Pavement (CRCP) and the construction of an additional 12-foot right lane of a 9-inch CRCP in each direction. The original dual lane was constructed in 1952 as U.S. Route XX. The overlay and outer lane construction projects were completed in 1974-76.

The original pavement consisted of a 9-inch jointed reinforced concrete pavement on a 6-inch Type II (crushed aggregate) subbase constructed on an A-5 silt soil. Transverse joints were sawed at 40-foot spacings. A 1-inch bituminous concrete layer was placed on the original pavement to serve as a leveling course, to correct superelevation, and to serve as a bondbreaker to the 6-inch CRCP overlay. The 6-inch CRCP overlay uses #5 deformed bars spaced at 8.5 inches (0.6 percent steel of the cross sectional area). Transverse steel (#4 bars) was spaced at 34 inches and tied underneath the longitudinal steel. The steel reinforcement was supported on metal chairs and the plans specified a cover tolerance of 2 1/2 - 2 11/16 inches.

The added 12-foot wide CRCP lane was placed on the outside of the overlay portion. The new lane is a 9-inch CRCP on a 4-inch crushed granular base. It uses #5 deformed bars spaced at 5.5 inches (0.6 percent of the cross sectional area). Transverse steel (#4 bars) was spaced at 30 inches and tied underneath the longitudinal steel. The steel reinforcement was supported by metal chairs and the plans specified a cover tolerance of 3 3/8 - 3 7/8 inches.

The plans provided the option of constructing the added right lane with the overlay or separately with a keyway. State highway personnel, however, report the keyway construction joint was likely used. The plans indicate there are tie bars in the keyway but we are unable to determine their length or spacing.

Traffic survey data gathered during the overlay design stage (1970) indicate ADT of 10,600-14,050; 12-13 percent trucks; projected 1991 ADT of 40,650-42,450; 12-13 percent trucks.

An extensive soils investigation was conducted during the overlay design stage. A brief review of the soil boring tabulation indicated the presence of silt, and rock fragments occurring within the top 5 feet of roadbed in the location of the widened outer lane. Little information is available for the roadbed beneath the original pavement. Also, there is little information available concerning the condition of the original pavement during design of the overlay.

Three cores of the existing CRCP pavement were recently taken and a petrographic examination was conducted. An analysis of the cores is found in Section VI.

A French drain was installed where the original roadway was in sag alignment and then daylighted to the reconstructed 6:1 side slopes. State personnel have stated these are not presently functioning.

The inside shoulder width is 4.0 feet. The outside shoulder is 10.0 feet. Each is a 3-inch thick bituminous concrete shoulder on a 7-inch dense-graded stabilized aggregate subbase. The shoulder joint was designed to be sealed with hot-poured, rubber-asphalt joint sealing compound.

Personnel who were present during the overlay construction projects reported that the project was shut down several times because of fines and clay balls in the aggregate. Also, there were difficulties in finishing the slab. The westbound overlay (middle and high speed) lanes were placed in May-July period. The eastbound overlay lanes were placed in October and

November when there were reported temperature variations as high as 40 degrees.

The pavement opened to traffic in late 1975. Pavement failures in the widened (right) lane of the eastbound roadway were noted in February 1976. The number of failures increased substantially during March and April. The State acted quickly in formulating a plan for investigating the causes of the failures.

In June 1976, a report was issued that included data from an extensive field study. However, the study only examined project data related to the widened lane and does not discuss cracks, subbase, soils, concrete quality, etc. of the overlay section of the project since, at that time, there was little distress in the overlay section. Briefly, the report concluded the failures were design associated and included the following: a) inadequate pavement support and the inability of the granular base to drain water away from under the pavement could have resulted in lower stability, b) adverse climatic conditions had reduced the concrete maturity at an early age and resulted in formation of closely spaced transverse cracks, c) the nature of the chairs and poor workability of the concrete could have contributed to the voids and weaknesses in the concrete cross section.

A second report was prepared in June 1982 and attempted to expand on what was learned from the 1976 study. At the time, the westbound lanes of Route XX had performed satisfactorily whereas the eastbound lanes had exhibited distress. The study concluded: a) pavement failures were primarily in these outside (widened) lanes, b) percent of steel reinforcement met the specifications, c) the pavement thicknesses were within the specification tolerance, d) numerous voids were reported in the lower half of the slabs but have not contributed significantly to the failures, e) the CRC overlays were performing satisfactorily and no visible signs of distress were noted, f) a significant portion of the CR-6 subbase material had a high content of fines that led to poor drainage characteristics, g) percent of air entrainment for eastbound and westbound lanes was within specification limits, h) unfavorable curing temperatures were present for the eastbound lanes, i) the crack spacing for the eastbound lanes was in the range of 2 feet apart while the range in the westbound lanes was generally 4-16 feet, and j) the eastbound shoulder lane exhibited high deflection characteristics.

By 1986, it was noted that a large number of areas of pavement distresses were beginning to occur in the middle lane and a lesser number were showing up in the high speed lane. The FHWA Division Office on (date), sent to the State a special report on the Route XX pavement distress and requested a detailed investigation of the pavement to determine the most cost-effective type of repair to be undertaken and determine what lessons could be learned and applied toward other proposed CRCP projects.

V. Details of Field Review

It was noted there were a significant number of punchouts in the middle lanes in both the eastbound and westbound roadways. Of particular interest was the radial cracking around the punchouts. Also, these areas were primarily concentrated in the wheel paths and appeared to be clustered. There were often long sections of pavement that appeared sound followed by sections of distressed pavement. The transverse distress consisted of fatigue cracks resulting in delaminated sections of pavement. These appeared to be located in the vicinity of the transverse bars.

The Team noted several areas of fine longitudinal cracks that appeared to be spaced at approximately the spacing of the longitudinal reinforcing steel; however, there was no evidence of staining of the pavement from possible corrosion of the reinforcing steel.

The full depth patching previously performed by State contractors appeared to be satisfactory. Discussions with State personnel indicate the patching details are in accordance with the state-of-the-practice procedures. Another patching project was initiated in late 1986 but shut down before repairs were completed. Operations are not expected to resume because of limited available funds to complete the necessary work; therefore, the Team was unable to observe the patching operations or the condition of the reinforcing steel.

There are several locations in all lanes where asphalt has been placed over distressed areas as a temporary measure. It is apparent that water is infiltrating the repaired areas.

The outside shoulders in both eastbound and westbound roadways were extremely distressed. In the westbound lanes, alligator cracking was noted to be particularly severe on the section between US XX to west of Route XX. There is a truck weighing station located on WB Route 99 near Route XX. Several trucks were parked on the shoulder, and the weigh station was open. State personnel informed us this was common practice. Also, the top of the shoulder had settled below the top of the mainline pavement.

It appeared the joint had been properly sealed; however, since the shoulder pavement was severely distressed, water is likely infiltrating the pavement structure from below the shoulder surface.

In the eastbound lane, there were large areas of full-depth patching that had been performed under previous maintenance contracts. The quality of the patches appeared to be good. The two outside lanes, but particularly the middle lane, appeared to be showing signs of severe distress at some locations. The previously mentioned punchouts, with their radical cracking patterns, were numerous. The punchouts were centered in the wheel paths. Where the high speed lane exhibited punchouts, it was noted that distressed areas of the center lane were nearly adjacent but staggered from these punchouts.

In the westbound roadway, the most distressed areas were found in the outer and middle lanes. The crack spacing pattern appeared to be acceptable. There were a few hundred feet of a longitudinal crack in the middle lane that was observed to be located about 2 feet inside the joint with the outer lane. There was some concern by project personnel that these cracks were related to the overall pavement distress; however, the team believes while these are undesirable, they are not affecting overall pavement performance.

VI. Discussion Items

We reviewed information on traffic data and a paving schedule for the roadway overlay and outer lane widening work.

Comparing the traffic data for 1970 and 1991 on the cover plan sheet for Project No. XX traffic counts, it appears the forecasted ADT's are within generally accepted margins of error. However, it appears the percent of truck traffic and number of equivalent 18-kip single axle loads being placed on the pavement has increased significantly over the projected loadings. Based on the 1970 traffic survey data, the percent of trucks was 12 percent and projected to remain at 12 percent in 1991. Recent loadometer data from a weigh-in-motion station on the eastern end of the project indicate the percent of trucks may be as high as 21 percent and have average 18-kip equivalent truck load factors as high as 3.76. The current lane truck distribution information indicates there is slightly higher than usual percentage of trucks in the middle lanes. This combination causes heavier than expected loadings on the 6-inch CRCP overlay.

There had been a concern that the weather conditions during placement of the overlay projects affected pavement performance. Two of the three overlay projects had substantial sections of concrete placed in the Fall when there were reported large temperature variations. The State supplied the Team with a paving schedule and reported daily temperatures from the projects' records. We compared this information with identified distress condition survey data. However, we were unable to correlate the two because of the inability to conclusively identify project station numbers with the mileposts shown in the condition survey data. Due to time constraints, we did not pursue this analysis.

The State performed an in-depth evaluation of three 200-foot sections of Route XX. The selected sections were believed to have been low, medium, and high distress areas. The evaluation consisted of a corrosion survey, core logs, longitudinal delamination survey, transverse delamination survey, chloride test results, and steel and chemical tests.

The distress information we received indicates there is extensive corrosion occurring in the three 200-foot test sections. The average chloride content amounts at the depth of the reinforcing steel were 3.1 lbs./cu. yd. for the low distress area, 3.3 lbs./cu. yd. for the medium distress area and 4.5 lbs./cu. yd. for the high distress area. According to accepted practice, chloride contents above 1.5-2.0 lbs./cu. yd. indicate a high potential for corrosion.

The corrosion surveys conducted on these sections show there is a high probability that steel corrosion is widespread throughout the three sections, and a high potential exists for cracking of the pavement due to corrosion. The current ASTM C876-80 specification indicates the following regarding the significance of the numerical value of the potentials measured:

1) Less than 0.2 volts, there is a greater than 90 percent probability that no reinforcing steel corrosion is occurring.
2) Between 0.2 and 0.35 volts, corrosion activity is uncertain.
3) Over 0.35 volts, there is a greater than 90 percent probability that corrosion is occurring.

Also, in laboratory tests where potentials were greater than 0.5 volts, approximately half of the specimens cracked due to corrosion activity.

The delamination survey data for the three sections indicates substantial areas of delaminated concrete in the medium and high distress areas.

The Pachometer survey data for these three sections indicates a range of concrete cover from 1.5 inches to 3.75 inches in the overlay areas. The plans specified a tolerance of 2 1/2 - 2 11/16 inches in the overlay area.

Based on our field observations of the cracks and distresses in the pavement and the above distress survey information, we believe there is a significant amount of corrosion of the reinforcing steel in each of the three test sections. There may be a somewhat lower level of corrosion activity in the low distress area as compared to the medium and high areas, nevertheless, extensive corrosion is likely occurring.

In reviewing the cores from the three test sections, it is apparent that the corrosion activity is predominately in the transverse bars. Nearly without exception, vertical and horizontal cracks were present where transverse bars were experiencing even minor corrosion. See Attachments 1 and 2 for illustrations. The vertical crack above the bars likely contributed to creating an environment that allowed corrosion to begin.

The Team also observed there had been a significant amount of full-depth patching within the project. We understand patching operations have been ongoing the last several years. As stated above, the outer lanes, particularly the EB outer lanes, experienced early distress, and extensive patching was already done in these lanes. Generally, the patches appeared to be performing satisfactorily. Data showing the rate of deterioration on this project was not available.

State personnel informed us they believe there has been a significant increase in the number of distress areas in the overlay areas (middle and fast lanes) in the last couple of years.

The results of the petrographic eximination are given in Attachment 3. Briefly, none of the cores contained reinforcing steel. They all appeared to contain sound concrete. One core, however, appeared to have a shale-type, limestone aggregate while the others had a marble-limestone aggregate. The Team did not have any data that showed whether one or more sources of aggregate were used on the project. State personnel who were present during construction indicated there was only one aggregate source. They did believe the project was shut down several times due to mudballs in the aggregate; however, there was no evidence of this problem in the cores we received.

VII. Recommendations and Conclusion

Information on this project was provided by the State. The information consisted of condition data only for the 6-inch CRCP overlay section (middle and high speed lanes). It must be recognized that we based our proposed alternatives on this limited information.

There are strong indications this pavement is rapidly deteriorating. The relatively thin (6-inch) CRCP overlay specification called for 2 1/2 - 2 11/16-inch cover over the reinforcing steel. The pachometer survey showed there were some areas with as little cover as 1.5 inches and many areas where the cover was in the range of 2 to 2.5 inches. The chloride studies, delamination surveys, and corrosion surveys show there is widespread corrosion in the test sections. The cores confirmed that there was active corrosion in the transverse bars.

If the corrosion occurring in the test sections is representative of the entire project, there is probably very little which can be done to prevent the disintegration of the pavement. Assuming this is the case, the ultimate solution to this problem is a reconstruction alternative.

A detailed distress and corrosion survey of the entire project needs to be made to quantify the amount of pavement that needs repair. An economic analysis should then be made to determine if it is more cost effective to continue heavy maintenance by patching the currently distressed sections or immediately reconstructing this section. Alternatives that need to be considered:

1. Continue heavy maintenance by patching identified distressed sections.

2. Remove the existing PCC and construct a new pavement section. The existing pavement may be suitable for recycling into a new PCC or AC pavement. If it is recycled, we believe the State will need to remove the existing pavement structure (including the original roadway), and construct an adequate drainage system. If a new PCC

pavement is constructed, we also recommend using full-depth, tied concrete shoulders.

3. Construct a minimum 4-inch asphalt overlay. The AC overlay must be thick enough to bridge the distressed areas. Also, we recommend the State perform repairs of currently distressed areas before placing an overlay. Depending on the thickness of the overlay, this may be only a short range improvement given the amount of corrosion activity occurring in the test sections. Experience in another State with overlays of 2 inches on corrosion-distressed CRCP has shown good performance for only 3-4 years. They are presently placing 4-to 6-inch overlays on CRCP and expect to get 10 years of service life.

There is also a need to improve the shoulders on the project, particularly the WB outside shoulder east of the truck weigh station. It is evident the trucks are stopping on the shoulder, presumably to avoid passing through open scales.

From our discussions with the State engineers, there was a recognition that the thin CRCP overlay, corrosion of the transverse bars and heavier than expected traffic loadings are major contributors to the distress. The State may wish to consider using epoxy-coated reinforcing steel on an experimental basis in future CRCP pavements. Another State has placed some CRCP having epoxy-coated reinforcing steel. We are not aware they are experiencing any problems, however, the pavements have not been in place long enough to judge long-term performance.

VII. Closing Remarks

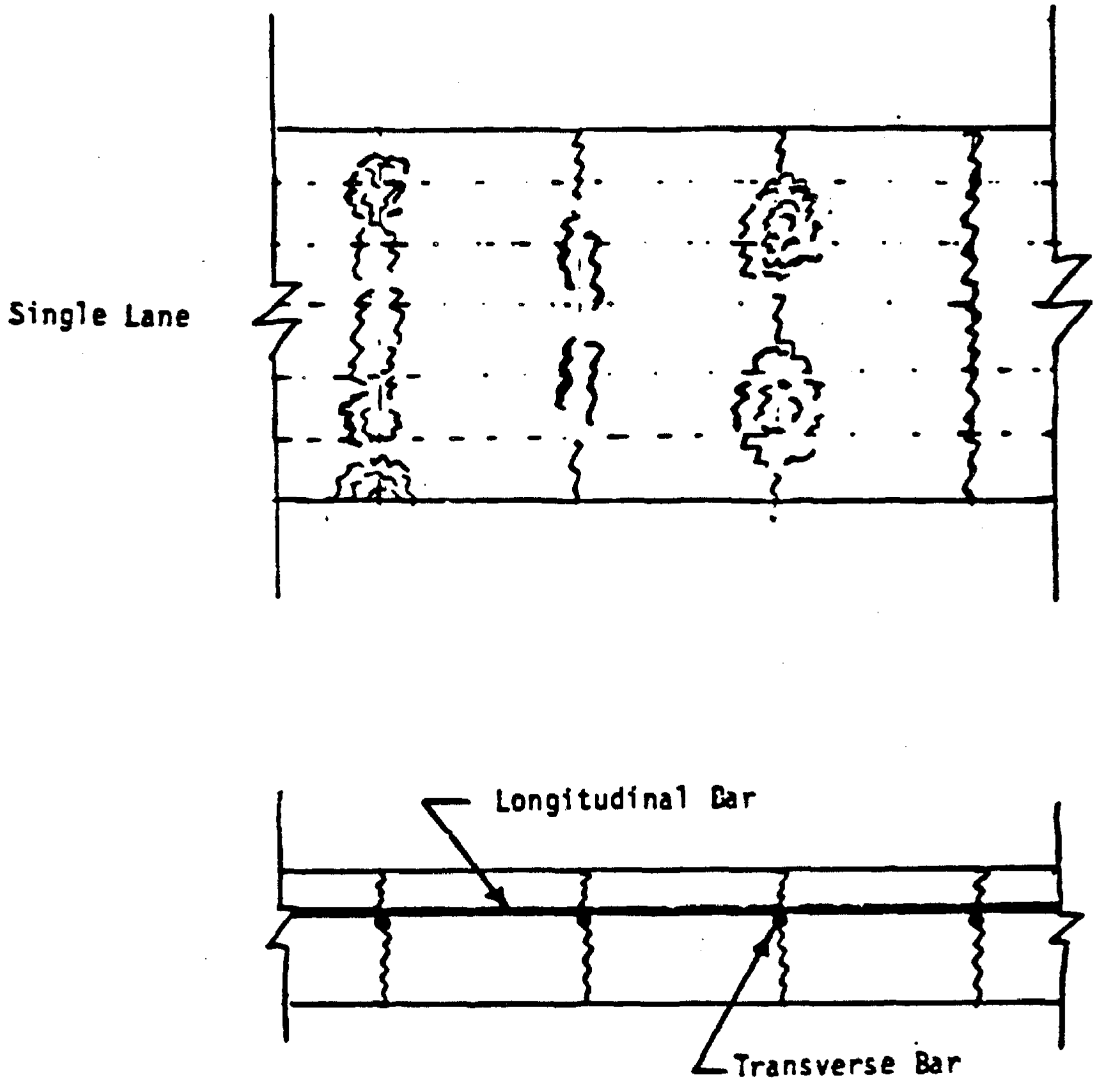
It is highly desirable to determine the cause of the distress prior to developing feasible rehabilitation alternatives to ensure that the selected strategy corrects the cause of distress. For any pavement rehabilitation project, the States are encouraged to follow the approach for an engineering and economic analysis as outlined in Mr. Barnhart's November 15, 1983, memorandum. Briefly, this includes the following steps:

(a) Establish existing condition of pavement.
(b) Identify distress.
(c) Determine cause of distress.
(d) Develop feasible alternatives.
(e) Conduct economic (life cycle cost) and engineering analysis of each alternative.
(f) Select most appropriate alternative.
(g) Design alternative.
(h) Provide feedback on performance.

We believe this a logical, practical approach to addressing pavement rehabilitation projects. Our observations recommendations and con lusion are based upon a limited review. We do not feel that we can briefly examine a pavement in a short time period and conclusively give the State the ultimate solution for a problem their engineers have been investigating for many months. We appreciate the opportunity to provide an outside opinion and to provide items for consideration. We hope our visit was as beneficial to the State as it was to us.

During the field trip, we observed a close working relationship among our Regional Office, Division Office and the State. We think this spirit of cooperation is excellent, and we look forward to continuing to work with the State and our field offices whenever we can provide assistance.

Attachment 1



CRCP DISTRESS PATTERN

Not to Scale

US Department of Transportation

Federal Highway Administration

Memorandum

Subject: Lateral Load Distribution and Use of PCC Extended Pavement Slabs for Reduced Fatigue

Date

From: Chief, Pavement Division
Washington, D.C. 20590-0001

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Attached are two copies of a report entitled "Lateral Load Distribution and Use of PCC Extended Pavement Slabs for Reduced Fatigue." The report was written by Mr. Mark Sehr, Highway Engineer Trainee, and the final editing was done by ERES Consultants.

The paper summarizes data and findings from several studies on Lateral Load Distribution and Load Stress at Pavement Edge. It includes discussion of the advantages of extended (or widened) lanes for PCC pavements and their effect on stress, strain, deflection, and PCC pavement deterioration.

We believe Mr. Sehr has prepared an excellent report and that it should be distributed to the division offices and shared with the States. We don't, however, have a sufficient number of copies to accomplish the desired distribution. Feel free to make copies or contact Mr. Donald Petersen at FTS 366-2226 to arrange for the printing of additional copies, or if you have any questions concerning the report.

Louis M. Papet

Attachment

LATERAL LOAD DISTRIBUTION

AND

THE USE OF PCC EXTENDED PAVEMENT SLABS

FOR REDUCED FATIGUE

by

Mark Sehr
Assistant Regional Pavement Engineer

June 16, 1989

FEDERAL HIGHWAY ADMINISTRATION
REGION 10

A paper prepared as a project during an Assistant Regional Pavement Engineer Training assignment.

NOTICE

PURPOSE

This paper summarizes data concerning lateral wheel distributions and presents conclusions based on that data. It also examines the advantages of extended (or widened) portland cement concrete (PCC) pavement slabs in terms of their effect on stress, strain, deflection, and PCC pavement deterioration.

BACKGROUND

Early road widths were only 15 ft (4.6 m), wide enough to handle the demands of horse-drawn vehicles. Following the discovery of the internal combustion engine and the development of motorized vehicles, traffic steadily increased. The width of roadways increased to 16 ft (4.9 m), and then to 18 ft (5.5 m). By the late 1920's, primary paved roadways were needed and the construction of 10 ft (3.0 m) lanes (20 ft [6.1 m] roadway) were standard practice. Today, conventional designs use 12 ft (3.7 m) lanes as standard practice.

PAST STUDIES AND COMMENTS ON THE DATA

The lateral location of traffic in the travel lane is the criteria to determine that a 12-ft (3.7 m) channelized lane is wide enough to withstand the repetitive loads of heavy truck traffic, and there have been several studies to determine the lateral truck wheel distribution in the pavement lane. These studies were generally initiated for design and safety concerns. There recently has been consideration of PCC pavement stresses and deflections and their connection with lateral wheel loads and shoulder encroachments. The studies attempt to determine the lateral wheel distribution and evaluate the damage done by differing transverse loads to help designers in building an adequate pavement structure. To summarize the information on lateral wheel distribution and the probability of pavement edge and shoulder encroachment, the results from a number of studies on lateral wheel path traffic distribution are highlighted in the following text.

The first study on lateral wheel distribution was completed by Taragin of the Federal Highway Administration in 1958.(1) This data, which is still used in both current PCC and asphalt concrete (AC) pavement design, showed that the highest frequency of travel and mean travel path distance occurred at little more than 2 ft (0.61 m) from the right pavement edge. The findings stated that an average of 2.5 percent of the mainline truck traffic encroached up to 12 in (305 mm) on the outside shoulder of the test section. The findings also stated that about 4 percent of the overall traffic drove closer to the edge than 12 in (305 mm). Taragin's study was completed on 12-ft (3.7 m) pavement lanes with unpaved shoulders.

The applicability of these results for current conditions can be questioned. There is some thought that the unpaved shoulders in the study may have been an artificial deterrent to the trucks encroaching on the shoulder and thus biased these results. Another consideration is that the size, speed, and number of trucks on the road in the 1950's is much less than today (1989). The 2.5 percent truck traffic encroaching on the shoulder is considered low based on current traffic characteristics shown in studies discussed later in the text.

The second study was done by Emery of the Georgia Department of Transportation in 1974.[2] This study found that 53 percent of the overall traffic traveled within 12 in (305 mm) of the pavement edge. It was also observed that at least 2.4 percent of the truck traffic encroached on the pavement edge. Nine percent of the traffic was driving in a 15-in (381 mm) wide wheel path that started 3 in (76 mm) inward from the right pavement edge and extended outward to include 12 in (305 mm) of the shoulder. This data was obtained from PCC pavements with an asphalt concrete shoulder; therefore it was concluded that a visible delineation existed between the pavement and the shoulder.

The data from this study showed that the motorist will drive near the edge of the pavement whenever possible in order to reduce or eliminate the uneasiness of close parallel travel to other vehicles in adjacent lanes.

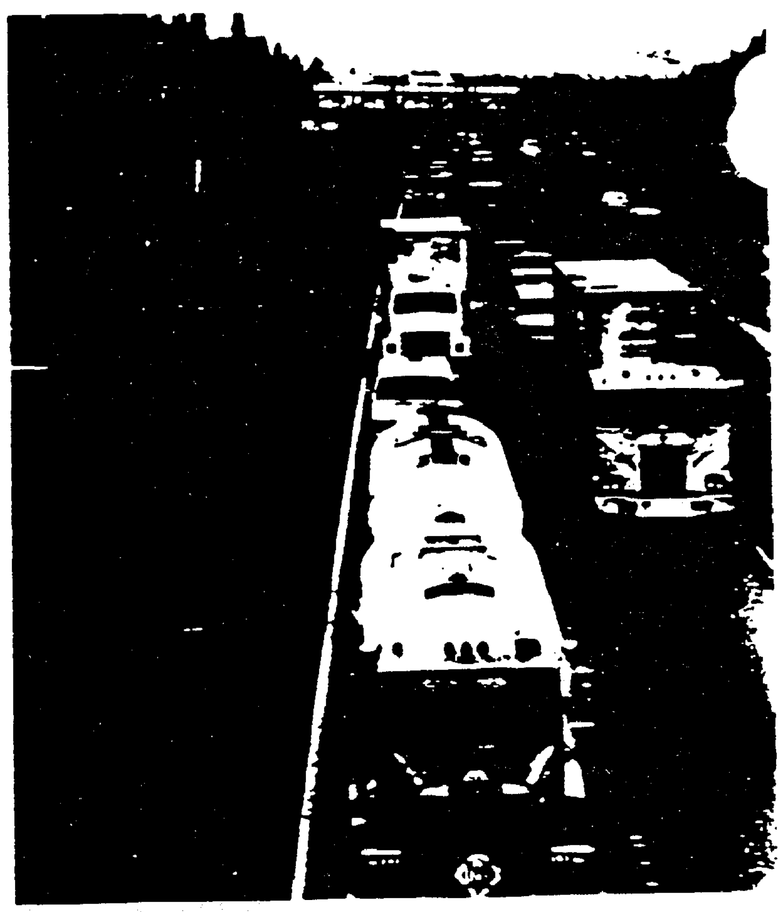


Photo 1 - Illustration of where vehicles tend to travel. Many vehicles are travelling within 18 inches of the edge of pavement.

A third study was performed by the National Cooperative Highway Research Program (NCHRP 14-3).[3] This project, conducted in Georgia during 1975, used a 10-mi (16.1 km) test section to follow and observe randomly selected trucks. Shoulder encroachment and the longitudinal length of the encroachment were recorded for each selected truck within the test section. A total of 205 trucks were followed in the 10 mi (16.1 km) section in Georgia. The results appear in tables 1, 2 and 3.[3] The length and location of the encroachments is shown in figure 1.[3]

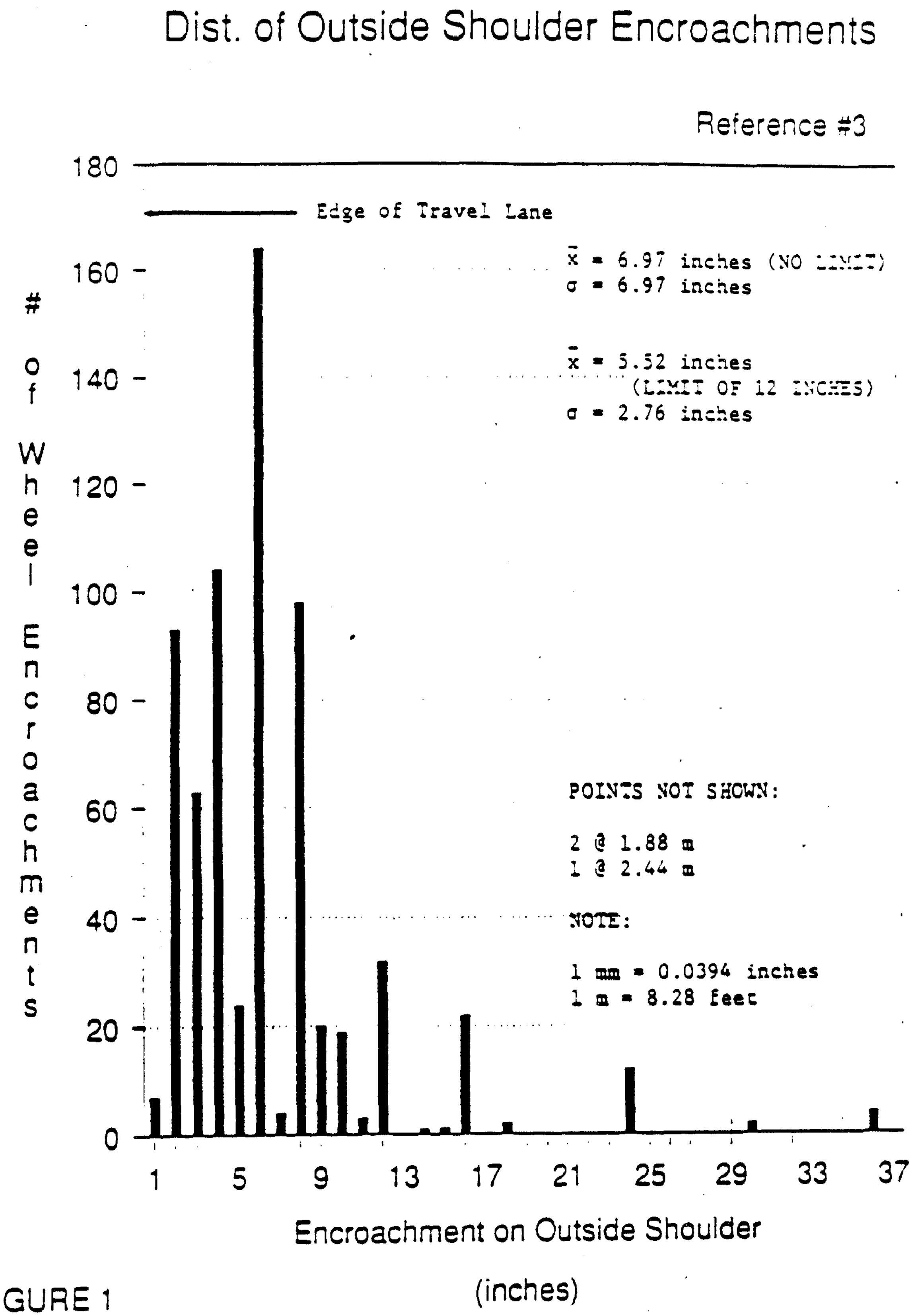
Table 1. Summary of outside shoulder encroachments by type of shoulder pavement (reference 3).

Item	Asphalt Concrete	Bituminous Surface Treatment	Total
Number of Samples	129.0	76.0	205.0
Number of Trucks Encroaching	83.0	50.0	133.0
Percent of Trucks Encroaching	65.3	65.8	64.9
Number of Encroachments	398.0	279.0	677.0
Avg. Encroachments Per Truck Encroaching	4.8	5.6	5.1
Avg. Encroachments Per Truck	3.1	3.7	3.3
Avg. Vehicle Speed, km/h	---	---	103.0

Note: 1 km/h = 0.621 mph

Table 1 is a summary of outside shoulder encroachments by type of shoulder material. Of the 205 trucks observed, 65 percent encroached on the shoulder at least once within the test section. Approximately the same percentage encroached on the different types of shoulders studied (AC and Bituminous Surface Treatment). This seems to indicate that the delineation between a PCC mainline pavement and an asphalt concrete shoulder or the rough surface of a bituminous surface treated shoulder does not necessarily deter trucks from encroachment. A total of 677 shoulder encroachments were recorded, which is an average of 3.3 encroachments per truck, per 10 miles of travel on rural interstate.

Table 2 provides a summary of the number of encroachments on the outside shoulder, by type of terrain. The percentage of trucks encroaching on the shoulder is approximately the same for both a flat and a rolling terrain. There is not enough data on a hilly terrain to make a conclusion, but it would be reasonable to assume that it would be approximately the same as the others.



Dist. of Outside Shoulder Encroachments
Reference #3
Edge of Travel Lane
x̄ = 6.97 inches (NO LIMIT)
σ = 6.97 inches
x̄ = 5.52 inches
(LIMIT OF 12 INCHES)
σ = 2.76 inches
POINTS NOT SHOWN:
2 @ 1.88 m
1 @ 2.44 m
NOTE:
1 mm = 0.0394 inches
1 m = 8.28 feet
# of Wheel Encroachments
0
20
40
60
80
100
120
140
160
180
1
5
9
13
17
21
25
29
33
37
Encroachment on Outside Shoulder
(inches)

FIGURE 1

Table 2. Encroachments on outside shoulder, by type of terrain (reference 3).

Type of Terrain	Total Trucks	Number of Trucks Encroaching	Encroaching Percent	Number of Encroachments	Encroachments Per Trucks Encroaching	Average Encroachment Per Truck
Flat	67	43	64.2	190	4.42	2.84
Rolling	134	87	64.9	480	5.52	3.58
Hilly	4	3	75.0	7	2.33	1.75
All Terrain	205	133	64.9	677	5.09	3.30

Table 3 shows that the outside shoulder encroachments occurred for an average of 4.5 seconds. The average encroachment was for a longitudinal distance of approximately 400 ft (122 m) and 0.60 ft (0.18 m) laterally onto the shoulder structure.

Table 3. Average time and distances of outside shoulder encroachments (reference 3).

Item	Outer Shoulder	Median Shoulder
Average encroachments per truck in 10 Miles	3.30	0.25
Average time on shoulder per encroachment, secs.	4.50	3.40
Average longitudinal distance on shoulder per encroachment, ft.	383.86	344.16
Average transverse distance on shoulder per encroachment, ft.	0.59	0.05

Figure 1 is a summary of the transverse encroachments onto the shoulder. As can be seen, the highest frequency of encroaching trucks was approximately 6 in (152 mm) onto the shoulder. A high percentage of times when trucks encroach, their dual tires are literally split between the pavement edge and shoulder.

The fact that shoulder encroachments occur regularly is well illustrated by the Georgia study. In the 10-mi (16.1 km) test section, the average number of encroachments per truck was 3.3. Of the 65 percent trucks encroaching, the average number of encroachments per 10 miles was 5.1 (see table 1[8]). Why do certain trucks go onto the shoulder more than others? The study was done with no cross-winds present, which eliminates an obvious external factor from biasing the results. Truck

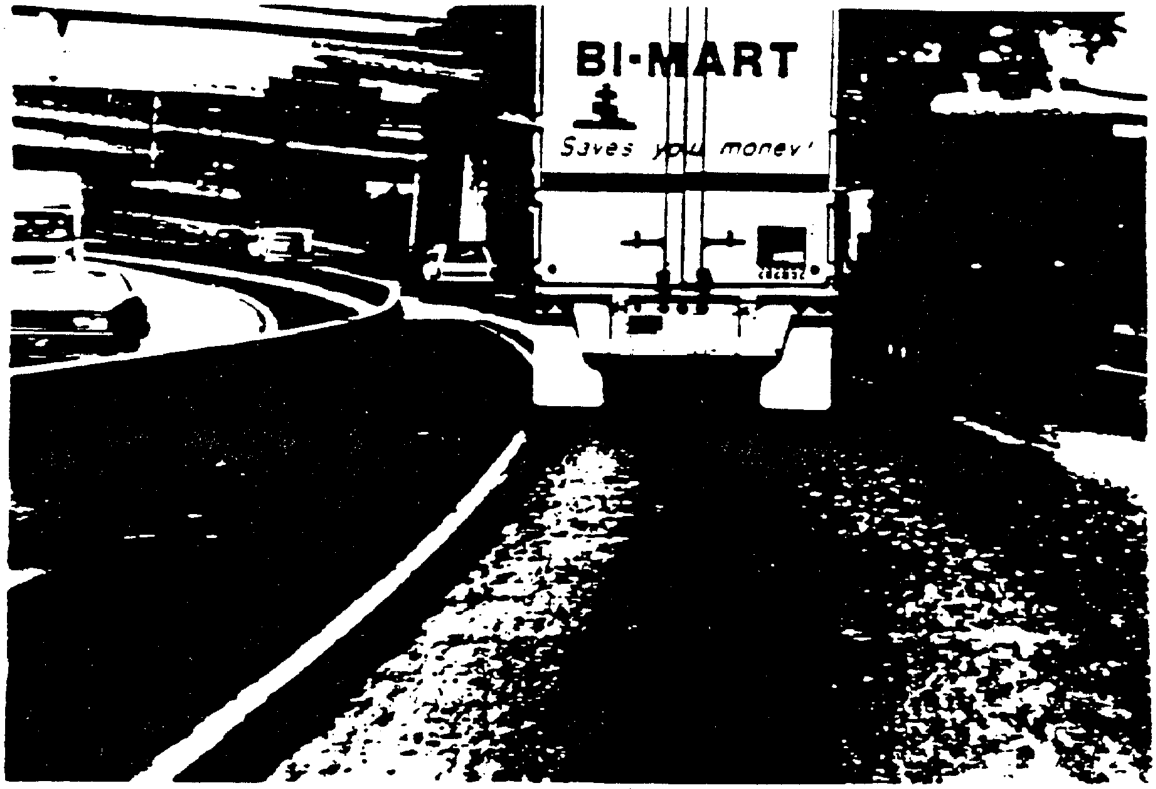


Photo 2 - Dual wheel split between shoulder and edge of PCC pavement. Note the pavement edge stripe location (12-foot PCC pavement slabs).

weight and the mental/physical condition of the driver may be factors, but their effect could not be determined from this study. The terrain seemed to have no effect on the percentage of trucks on the shoulder.

The main point to be made from this research is that, for whatever reason, trucks do encroach on the shoulder or beyond a 12-ft (3.7 m) channelized lane. The 3.3 encroachments per truck per 10 miles is not an alarming figure; however, when multiplied by the total truck miles accumulated, the total number of encroachments quickly adds up. As is discussed in the next section, the greatest damage to PCC pavements is done when loads are placed on the outside edge at the corner or at the midpoint of the slab. Obviously, encroaching truck traffic places loads at the pavement edge. Also, when an edge drop-off is present, the dual wheel splits the lane-shoulder joint and all of the load is transferred to only one wheel. This causes even higher load concentrations at critical points in the pavement structure. Because this edge loading can contribute to failure of the pavement, this is a subject that deserves more attention from the highway community. With the use of microcomputers and mechanistic design procedures, the pavement design engineer can now readily analyze critical load concentrations and calculate the necessary lateral support required to minimize damage and economize on thickness design.

The results presented by Emery were supported by a study done by Texas on the lateral placement of trucks on a highway during 1983.[8] This research concluded that approximately 0.5 percent of the trucks sampled completely encroached onto the shoulder. Complete encroachment is defined as occurring when the dual tire is off the pavement and entirely on the shoulder. Up to 12 percent of the trucks were partially on the shoulder, defined as occurring when the dual wheel is on the pavement and shoulder simultaneously. A main point of this study is that a higher percentage of trucks (12.0 compared to the 2.5 found by Tagarin and Emery) are running on the shoulder and are not being accounted for in current highway design practices.

ILLUSTRATION OF THE MEAN DISTANCE "D" FROM SLAB EDGE TO OUTSIDE OF DUAL TIRES

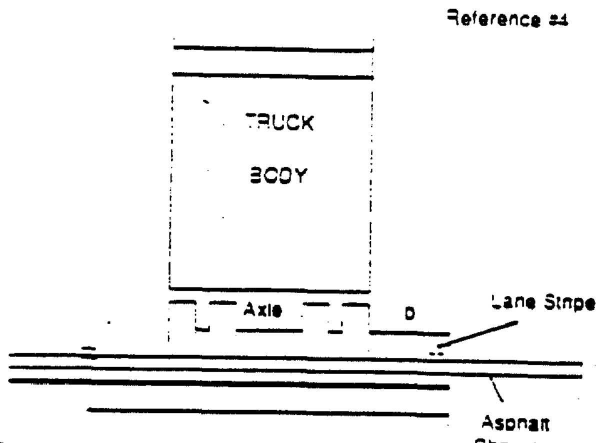


FIGURE 2 Typical control section cross section.

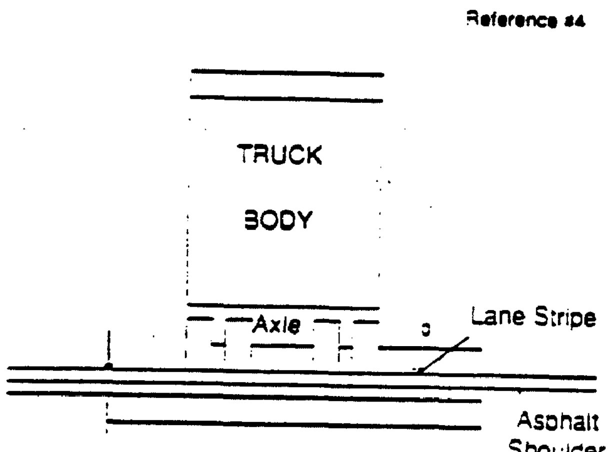


FIGURE 3 Cross section of the extended width concrete pavement.

A fifth study was recently completed by the Transportation Research Laboratory at the University of Illinois in March 1988.[4] This study compared the lateral distribution of truck wheel travel paths on a 12-ft (3.7 m) PCC freeway lane with AC shoulders with two extended PCC pavement slab widths (13.5 ft [4.1 m] and 13.7 ft [4.2 m]) that were also delineated into 12 ft (3.7 m) driving lanes. This is shown in figures 2 and 3.[4]

The extended PCC pavement slab test section was located on 4.1 mi (6.6 km) of rural freeway on I-57 in Illinois. The southbound outside lane was striped as a 12-ft (3.7 m) lane and the extended PCC pavement slab widened 18 in (457 mm) into the shoulder. The PCC pavement extended 18 in (457 mm) beyond the right lane stripe into the outer shoulder. The northbound outside slab extension was 20 in (508 mm), as shown in figure 3.[4] Both directions have asphalt concrete shoulders. Due to site limitations, the data collection for the section was limited to one location.

The type of truck monitored in this study was semi-tractor trailers. Passenger vehicles and smaller single unit trucks have considerably less impact on edge stresses and were not included in the sample. As with the Georgia study, no data was taken on days with a strong cross-wind. Also, the study section was on rural interstate and the terrain was flat, with no horizontal curvature within 2000 ft (610 m) of the test sections, in order to further reduce the effect of external factors. Film from 8mm movie cameras was reviewed by the University of Illinois staff to obtain the data.

A summary of the lateral encroachments on the edge of the pavement is given in table 4.[4] A comparison of the average lateral placement of the truck tires in the control section with the extended PCC pavement slab section leads to the conclusion that trucks will not significantly move outward in the designated traffic lane. Wheel path locations show that trucks tend to drive approximately 2 in (51 mm) closer to the edge line stripe on an extended PCC pavement slab. This would tend to indicate that with adequate slab width, the pavement stripe location controls the lateral wheel path in which the trucks travel rather than the overall width of the pavement itself.

The trucks stayed an average of 20 to 22 in (508 to 559 mm) away from the edge lane stripe, whether on a 12-ft (3.7 m) PCC slab or an extended PCC slab marked with a 12-ft (3.7 m) driving lane. At first, the 20 to 22 in (508 to 559 mm) distance may seem adequate, but after evaluating the data it can be seen that approximately 30 percent of the semi-tractor trailers travel 18 in (457 mm) or less

Table 4. Values of sample size, lateral distance D (figures 2 and 3[4]), standard deviation, and percent shoulder encroachments for the lateral distribution of truck wheel paths (reference 4).

	Sample Size (Vehicle)	D, in (Fig. 2&3)[4]	Distance From Edge Lane Stripe, in	Standard Dev., in	Percent Shoulder Encroachments (Beyond Lane Stripe)
Control Section	536	22.0	22.0	9.0	0.7
18-in Extended Section	691	38.1	20.1	9.1	1.7
20-in Extended Section	613	40.5	20.5	9.1	0.7

from the slab edge. The number of trucks within 12 in (305 mm) of the slab edge is approximately 10 percent (Figure 4).[4] This is a significant number of trucks travelling at the edge of the PCC pavements and, if these results can be considered representative of the rest of the country, indicates that a large number of edge loadings are not being considered in most of the State highway agencies design processes.

It should also be noted that none of the trucks sampled in the widened lane pavement test sections traveled on to the PCC pavement edge or AC shoulder. A small percentage did encroach on the edge lane stripe, but none went to the pavement edge itself.

Figures 4, 5, and 6 show the data collected for each test section and the control section.[4] These once again show that the average distance from the edge stripe is between 20 and 22 in (508 and 559 mm). A significant percent of the truck traffic does travel within 18 in (457 mm) of the PCC slab edge on the control section. The extended PCC pavement slab sections had very little traffic within 18 in (457 mm) of the edge of the PCC slab. This can be seen clearly in the graphs of the distribution of trucks, as indicated by the mean lateral distribution (x).

Photo 3 - A truck travelling near the edge of pavement. Note the pavement edge stripe location (12-ft PCC pavement slabs).

TRUCK LATERAL WHEEL DISTRIBUTION
(control section)

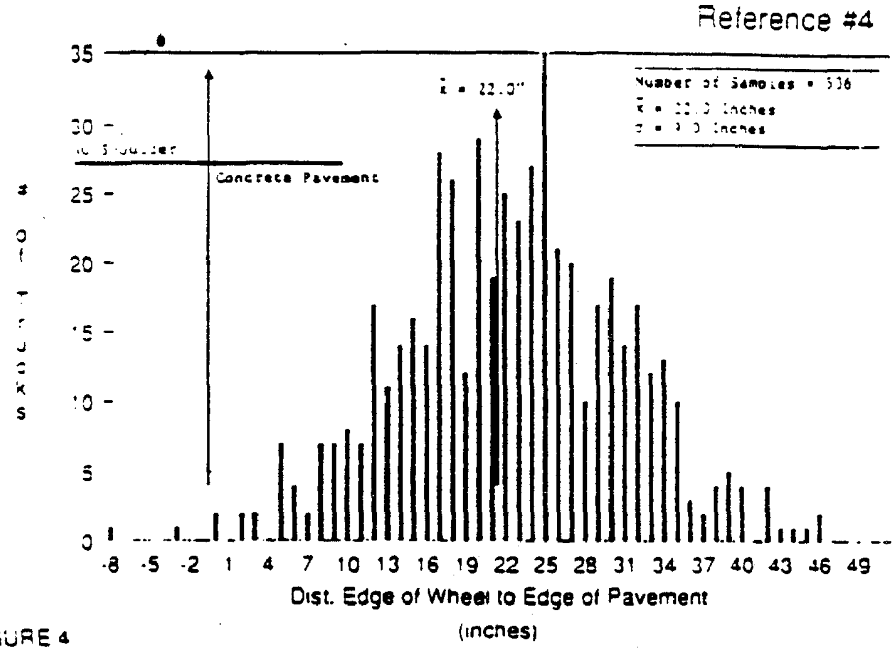


FIGURE 4

LATERAL WHEEL DISTRIBUTION
(Southbound - 18 inch slab extension)

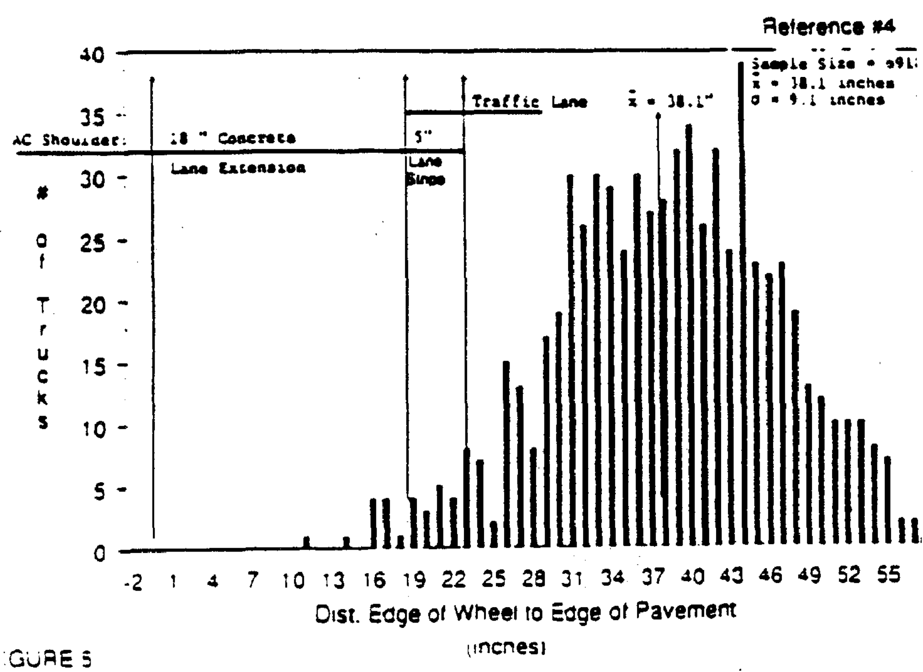


FIGURE 5

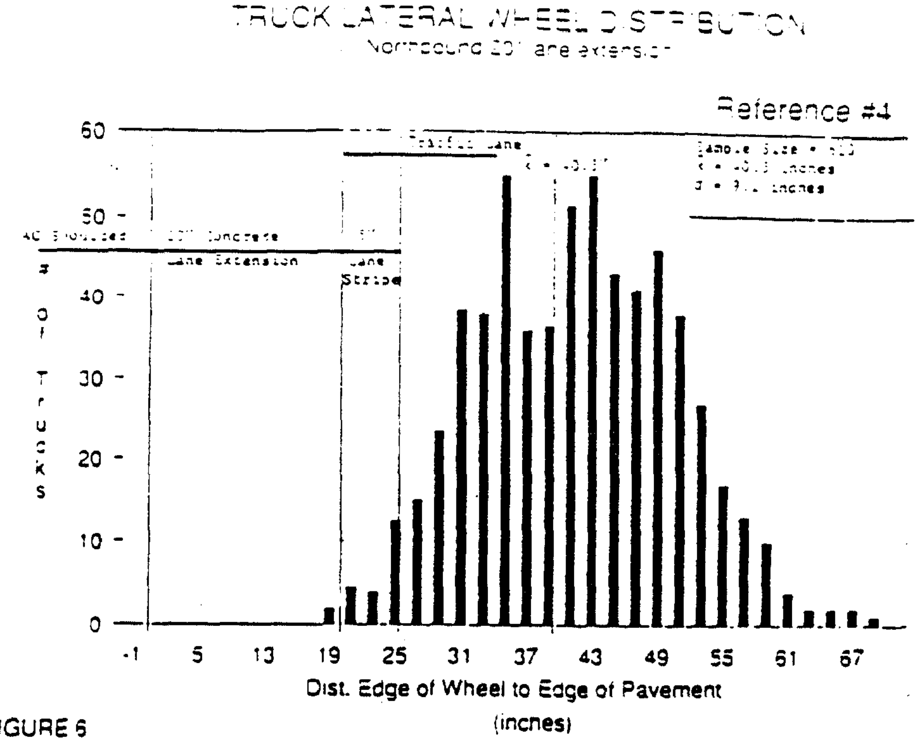


FIGURE 6

Since the extended PCC pavement slabs had little effect on the lateral wheel path of the trucks, it suggests that the pavement edge stripe has a major effect on the traffic distribution. However, opinions on this point differ among maintenance, traffic, design, and construction engineers. If the pavement edge stripe is located more than 12 ft (3.7 m) from the centerline and on an AC shoulder, where will the trucks' wheel paths be located? Will they be 20 in (508 mm) away from the edge of the PCC slab or 20 in (508 mm) away from the edge stripe itself? It is a question that deserves much more thought by highway agencies.

ANALYSIS OF DATA ON PCC PAVEMENT DETERIORATION

Extended PCC pavement slabs refers to PCC slabs which are built wider than the conventional 12-ft (3.7 m) striped traffic lane. The normal slab extension that has typically been constructed varies from 1 to 3 ft (0.3 to 0.9 m). The basic concept behind the construction of extended slabs is to keep the heavy wheel loads away from the outside edge of the pavement so that traffic loads used in design can be considered interior loads. "Truck wheel loads placed at the outside pavement edge create more severe conditions than any other load position. As the truck placement moves inward a few inches from the edge, the effects decrease substantially."[9] Extended PCC pavement slabs are not, however, a replacement for an adequate shoulder pavement structure. An extended PCC pavement slab should be used with an adequate shoulder structure to meet an agency's own design standards.

In considering the potential benefits from the use of extended pavement slabs, their effect on pavement deterioration should be recognized. In this section, slab stress, strain, deflection, and moisture infiltration are considered as they are related to extended PCC slabs.

Stress and accumulated pavement fatigue are two parameters used to calculate the damage done to a PCC pavement by applied loads. It is widely accepted that "the most critical pavement stresses occur when the truck wheels are placed at or near the pavement edge and midway between the joints."[5] Since the critical stress occurs at the mid-point of the panel, load transfer devices at transverse joints do not have a great influence on the load stresses at the mid-panel. The effect of trucks running at the pavement edge can be shown by the stress-fatigue analysis in figure 7.[5] The fatigue was calculated at various locations on the PCC slab, inward from the slab edge, for different truck wheel load placements. "This factor, when multiplied by edge load stress, gives the same degree of fatigue consumption that would result from a given truck placement distribution."[5] As the lateral truck wheel distribution moves away from the PCC slab edge and inward on the slab, the total number of load repetitions increases, but the damage due to stress decreases. As illustrated by figure 7, the fatigue stress decreases as the percent trucks at the edge decreases.[5]

Equivalent Edge Stress Factor Versus
Percent of Trucks at Edge

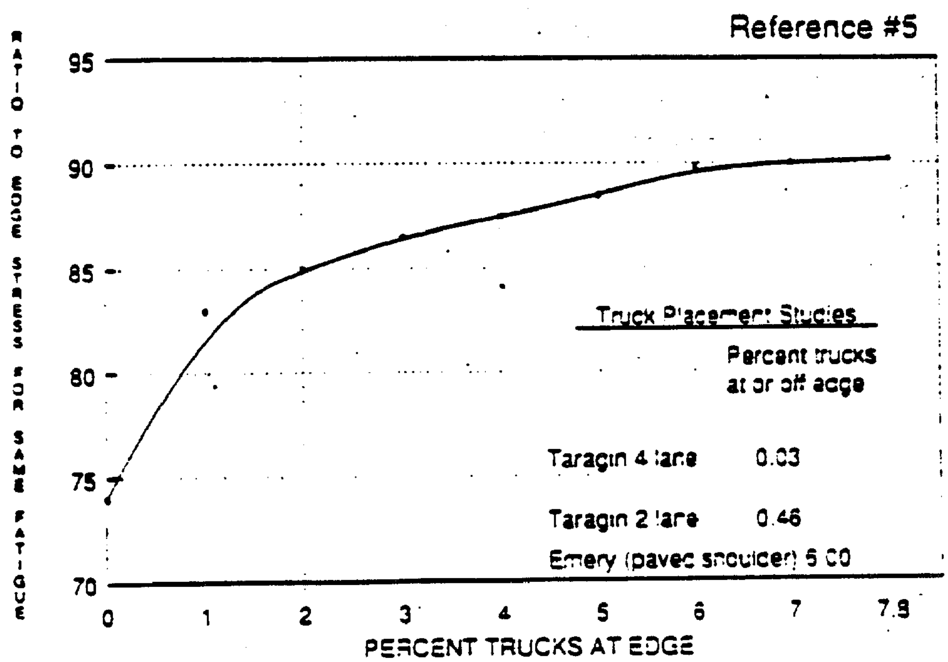


FIGURE 7

A theoretical evaluation of the effect of extended slabs on stress reduction was performed using the PC version of ILLISLAB. A 14 ft by 16 ft (4.3 by 4.9 m) slab of varying thicknesses was modeled on a subgrade with a load applied at the slab's midpoint on the outer edge. The induced stress was then calculated at that point (a 4-in [101 mm] offset, and at further offsets from the edge of 8 in, 16 in, and 24 in (203, 406, and 610 mm) by moving the load inward on the slab. As is shown in figure 8, there is a large reduction in stress that results from moving loads away from the slab edge. This reduction is greater for thinner slabs. Since stress is related to fatigue and, ultimately, deterioration of the slab, this reduction in stress is a desirable goal.

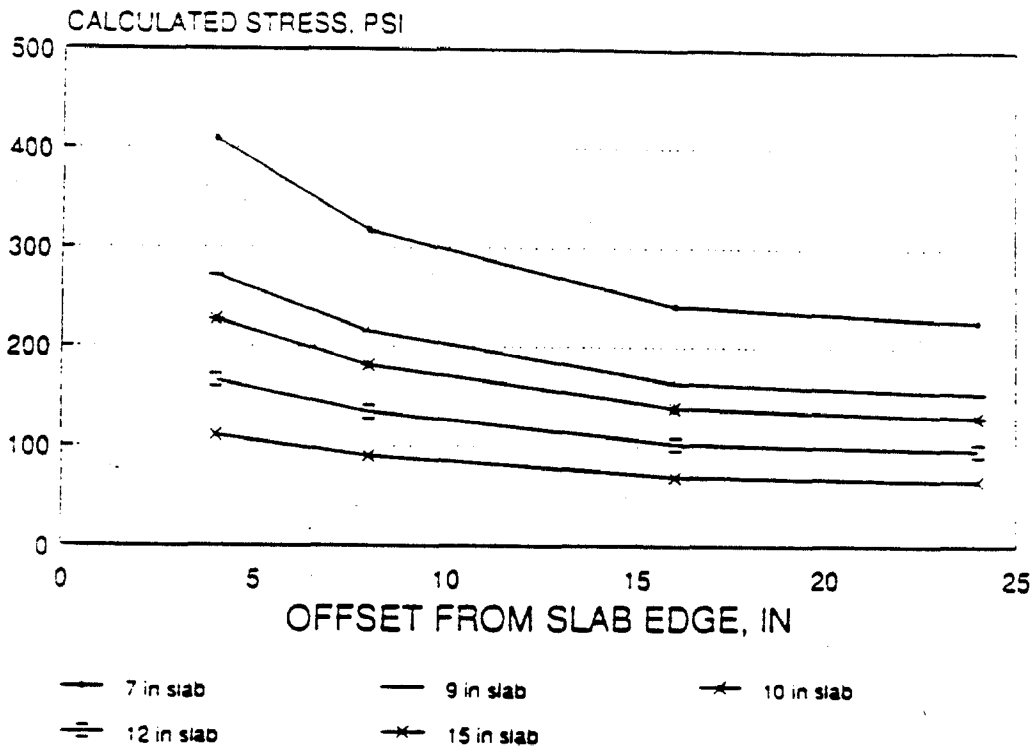


Figure 8

The effect of wheel load placement on pavement slab strains has also been documented. As measured in test sections by the Minnesota Department of Transportation (MnDOT), pavement strains are shown to be greatest at the free edge of the PCC slab (see figure 9).[6] The edge strain reduces quickly as the wheel load moves inward from the edge of pavement, as shown in figure 10 (from a MNDOT laboratory test slab) and tends to level off when the applied load is 18 to 24 in (457 to 610 mm) from the edge of pavement.[6] "In general, free-edge strains were 36 percent to 50 percent greater than interior strains."[6]

Maximum Strains at MNDOT Test Stations

Reference #6

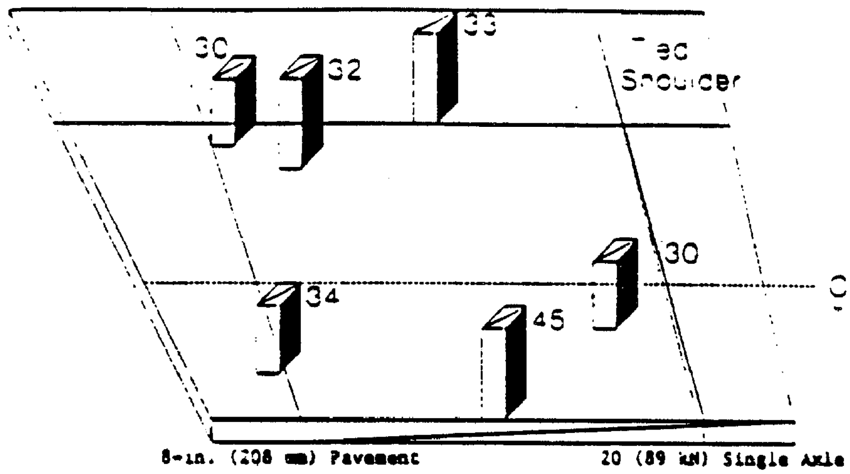


Figure 9

Edge Strain in Laboratory Test Slab

Reference #6

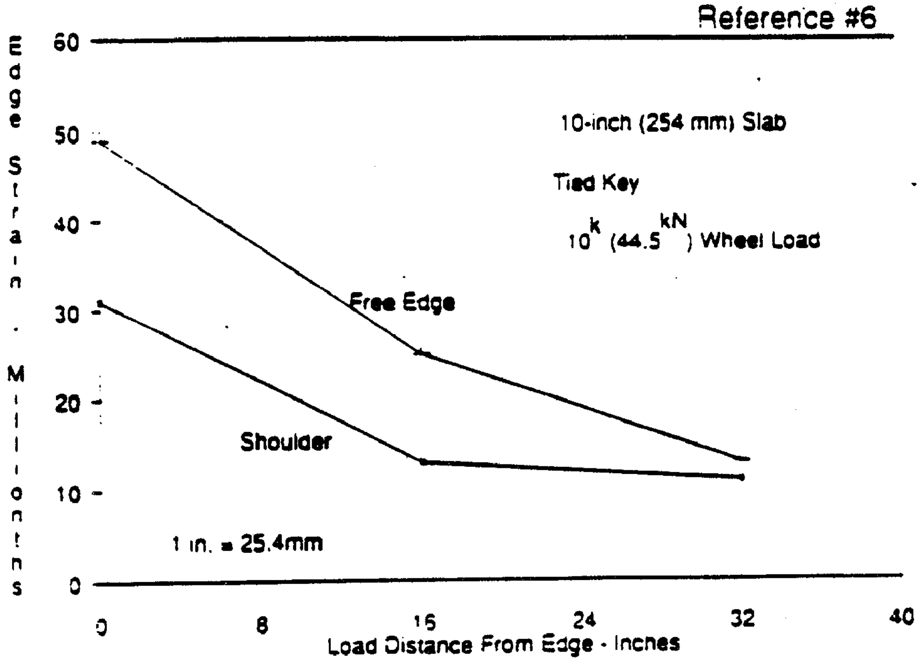


Figure 10

Measurements and established data on the warping and curling of PCC pavements and the effect of extended PCC pavement slabs with 12 ft (3.7 m) channelized traffic lanes is limited. "Warping leaves slabs unsupported for distances of as much as 4 to 5 ft (1.2 to 1.5 m) at slab corners and 2 to 3 ft (0.6 to 0.9 m) at slab edges."[10] The loss of support along the slab edges and the compressive forces of the concrete itself are two adverse effects of warping. Curling refers to the concrete slab behavior due to the differing temperatures in the slab depth. Slabs curl upward (corner support lost) during the night because the temperatures are cooler on the top surface than on the bottom of the slab. Conversely, slabs curl downward (corners downward) during the day due to the warmer temperatures on the top surface of the slab compared to the bottom. There is not enough information available to see any differences in warping and curling with the use of extended PCC pavement slabs.

Water infiltration underneath existing pavements is being emphasized as a major factor in the deterioration of some pavement structures. The use of extended pavement slabs would mean less traffic directly on the longitudinal shoulder joint (either PCC or AC shoulder). The required maintenance of the joint seal should be lower and the seal achieved during construction would function as an effective joint for a longer period of time because of the fewer applied loads. With a better performing joint, there should be less water infiltrating through the longitudinal edge joint to the underlying base material and less potential deflections due to the lateral wheel load location.

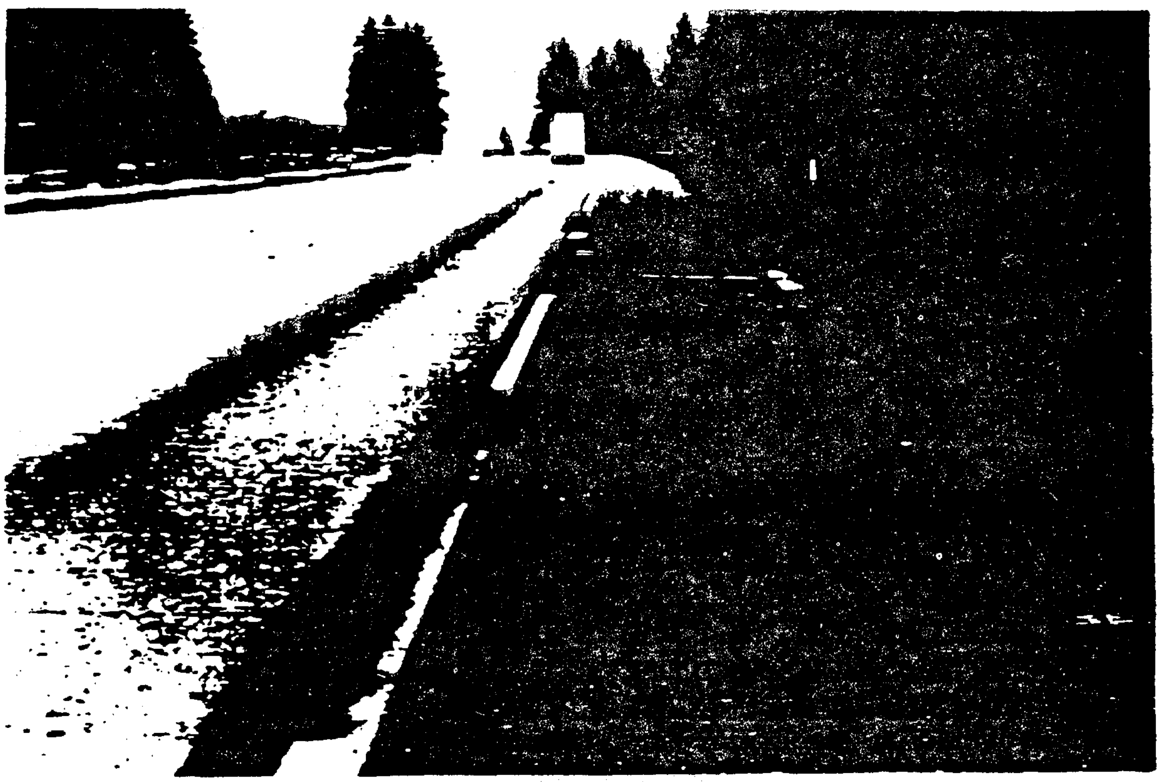


Photo 4 - Illustration of PCC pavement/shoulder joint deterioration. Note the pavement edge line location (12-foot lanes).

The use of extended pavement slabs will lead to improved pavement performance. The extended life will be due to lower edge strains, reduced overall stress, lower edge and corner deflections, and less water infiltration through the longitudinal pavement shoulder joint. It should be possible to design a thinner pavement section with extended pavement slabs and obtain the same performance as that of a thicker pavement without extended slabs. "Results of a study conducted by the Construction Technology Laboratories for the Federal Highway Administration indicate that lane widening is an efficient method of improving pavement response under traffic load."[11] Improved performance can realistically be expected from the use of extended pavement slabs without drastic changes of design. With the correct information available, highway designers will support the construction of extended pavement slabs to extend the life of the pavement, with no decrease in safety or increase in initial (unit price) costs.

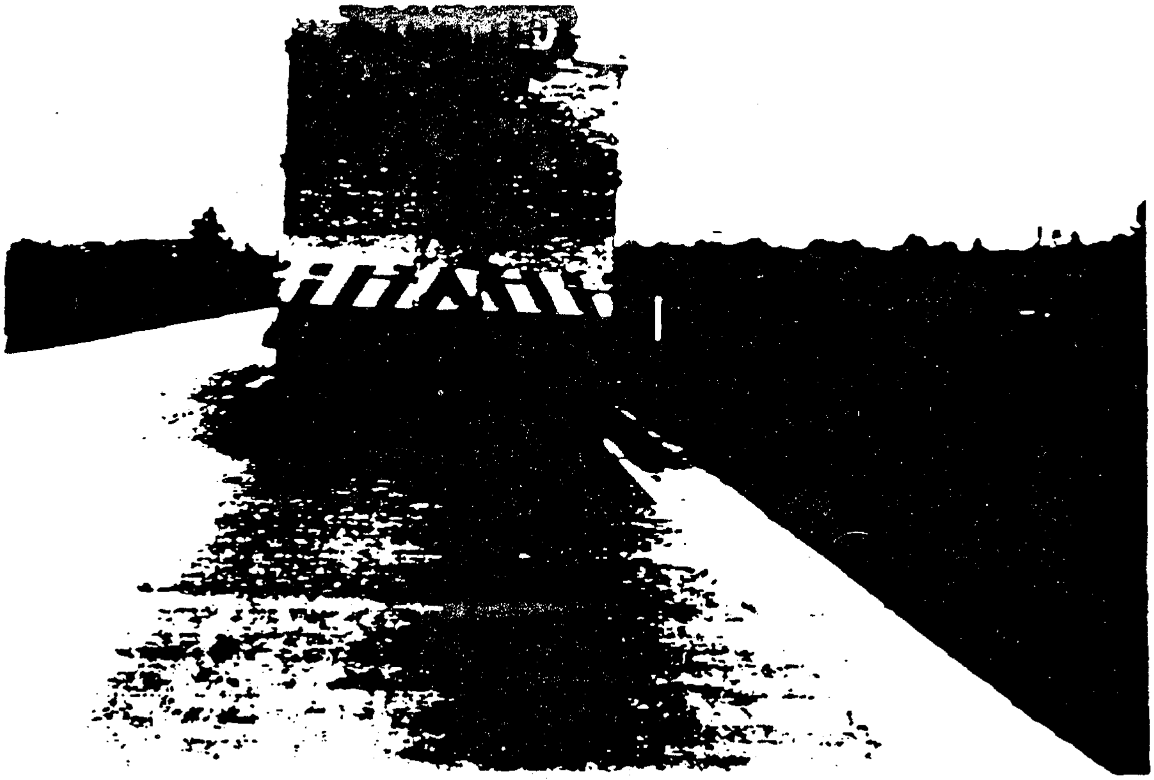
It should be noted that the benefits of extended slabs can only be realized if the slabs are properly striped. Since the stripe appears to control the lateral wheel distribution, placement of the lane-shoulder stripe must be done at the 12 ft (3.7 m) mark and not at the edge of the extended slab or even on the adjacent shoulder.

SAFETY CONCERNS

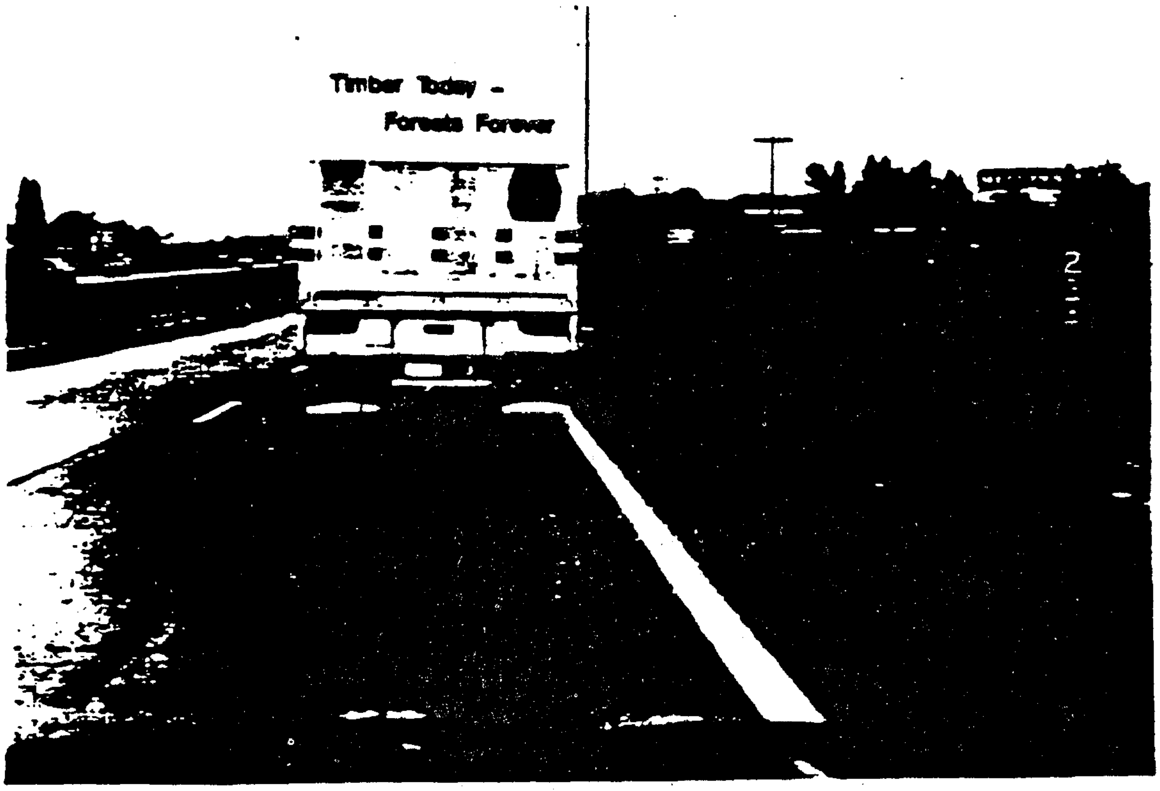
The issue of safety is also a subject that should be addressed when discussing the use of extended slabs with the traffic lane striped at 12 ft (3.7 m). Discussion with the Office of Highway Safety and Regional Safety Engineers indicates that there are no increased safety problems with the use of extended lanes when compared to standard 12 ft (3.7 m) lanes. In fact, widened lanes are equal or superior to the conventional lanes, from the standpoint of user safety. However, it should be noted that extended PCC slabs should not be considered a replacement for a shoulder structure.

In the area of user safety, lane edge stripe maintenance and location on extended pavement slabs has been a debated subject. Practices on standard 12 ft (3.7 m) pavements differ from State to State and even between different areas of a State. There are two basic theories about where to put the pavement lane edge stripe when shoulder structures differ from mainline pavement type. One practice places the edge line stripe on the PCC pavement (mainline) to keep the load off of the pavement/shoulder joint. The advantages of keeping the wheel loads 18 to 36 in (457 to 914 mm) away from the pavement have already been discussed and documented.

The second practice is to place the pavement edge line stripe beyond the pavement/shoulder joint and onto the shoulder (usually AC). One of the reasons for this practice is that the edge line will last a longer period of time, therefore reducing the associated maintenance costs (paint, trucks, crew). The other support for this practice lies in the color contrast between the white paint of the edge line and the



Photos 5 & 6 - Two examples of widened lane pavements performing as designed (inside lane 12 feet, outside lane 14 feet and striped at 12.5 feet).



black color of the asphalt shoulder. However, economic ramifications of accelerated pavement deterioration far exceed potential maintenance benefits of increasing the effective stripe life.

Most design engineers acknowledge the advantages of extended pavement slabs in keeping the traffic off of the shoulders. However, the control of the actual painting of the edge line stripes is in the hands of traffic and maintenance engineers. Until design concepts and concerns are thoroughly understood by this group, many extended slab designs will be wasted because of improper edge line placement. When comparing the costs of annual or bi-annual painting of edge lines and a 20-30 percent extended pavement life, the benefit-cost ratio supports correctly placed and maintained edge stripes. There is also little conclusive data that supports the concept of better edge line delineation on AC pavements than on PCC pavements.

CLOSING STATEMENTS

When gathering data for this paper, many research reports on differing subjects were reviewed that briefly mention extended pavement slabs and their benefits. Information regarding extended slabs and stresses, strains, deflections, and overall pavement deterioration is limited to portions of studies done on other subjects; there have been only a few studies performed recently that concentrate solely on lateral wheel load distribution and PCC pavement fatigue.

The greatest use of extended slabs is concentrated in the midwest (Iowa, Minnesota, South Dakota, and Wisconsin), with each State's design a little different. They are all building extended slabs for the same reason; to move the heavy truck wheel loads away from the edge of pavement. Extended pavement slabs have also been used in Georgia, Idaho, Louisiana, and Oregon. Delaware will be constructing its first extended pavement slab project during their 1989 construction season. As more pavements are built with extended slabs, much more information will be available on their performance.

There are points in the previous text that deserve repeating.

- Present wheel load distribution will be an average of 20 to 22 in (508 to 559 mm) away from the lane edge stripe on 12 ft (3.7 m) PCC slabs.
- The edge stripe, and not the overall width of the lane, controls lateral truck wheel distribution.
- Studies have shown that detrimental edge loads are reduced significantly at 16 to 20 in (406 to 508 mm) away from the PCC pavement edge.

- A PCC pavement that is widened 18 to 24 in (457 to 610 mm) and striped with a 12-ft (3.7 m) lane can expect a 20 to 30 percent increase in pavement fatigue life and reduced maintenance costs.

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Memorandum

U.S. Department of Transportation
Federal Highway Administration

Washington, D.C. 20590

Subject: Longitudinal Cracking at Transverse Joints of New Jointed Portland Cement Concrete (PCC) Pavement with PCC Shoulders

Date: NOV 30 1988

From: Chief, Pavement Design and Rehabilitation Branch

Reply to Attn. of: HHO-12

To: Regional Federal Highway Administrators
Attn: Pavement Specialists

Attached is a report outlining a longitudinal cracking problem that occurred at the transverse joints near the edges of a mainline PCC pavement. This cracking is believed to be the result of the intrusion of mortar into the mainline transverse joints during shoulder construction.

The report indicates that the mainline PCC pavement was placed in warmer summer weather while the PCC shoulders were subsequently placed during cooler weather after the contraction joints had opened. When the shoulders were placed, mortar intruded into the mainline contraction joints and hardened. During the placement of the shoulders the contractor provided additional vibration along the lane/shoulder joint. This may have increased the flow of mortar into the open joints. With warmer weather, the slabs, unable to expand, developed longitudinal cracks near both edges of the mainline pavement.

It is recommended that, in similar situations, States consider sealing the sides of the contraction joints prior to placing adjacent pavement. This would prevent the intrusion of mortar into the joint.

LKastner
for Paul Teng

Attachment

I-64 Longitudinal Cracking - 1988

PROBLEM

On June 3, 1988 project personnel on 35-miles of a yet-to-be opened to traffic section of I-64 noted some longitudinal cracking on both sides of the transverse contraction joints in the portland cement concrete (pcc) pavement. At first, the longitudinal cracking appeared to be limited to a couple of interchange ramps, however, it was soon found to be scattered throughout the approximately eleven miles of pcc pavement having pcc shoulders. The other 24 miles of yet-to-be opened section of I-64 consisting of pcc pavement having asphalt shoulders was not experiencing any cracking. The cracks were typically 12-18 inches in length and were located approximately 9-12 inches from the mainline/shoulder joint. When cracking occurred, the cracks were always present on both sides of the transverse contraction joint and near the inside and outside of the mainline/shoulder longitudinal joints.

BACKGROUND

The eleven miles of pcc pavement were placed in 1987 under 3 paving projects involving two contractors. The mainline pcc consists of 12-inch jointed, mesh-reinforced concrete pavement having 40-foot joint spacing. The typical section consists of two travel lanes with an additional truck climbing lane for several sections. The tied shoulders consist of 10-inch non-reinforced concrete pavement having 20-foot joint spacing. The shoulders are tied to the mainline pavement with hook bolts. Free draining base material consisting of aggregate (No. 57 stone) treated with two percent of paving asphalt by weight of the aggregate is in place beneath the mainline and shoulder pcc pavement. Longitudinal and transverse joints were sawed and sealed using a low modulus silicone sealant. This section of I-64 was scheduled to be opened to traffic on July 15 so it was imperative that the cause of the problem be determined and an acceptable solution found before the road was opened.

On June 6, Messrs. W.T. Kelley, D.M. Hart and D.J. Voelker observed WVDOH personnel conducting a coring operation on the eastbound off-ramp at the Beaver Interchange. The core bit had caused a spall at the pavement/shoulder interface adjacent to the transverse contraction joint. It appeared that hardened mortar was in the transverse contraction joint below the backer rod. See attached drawing.

Although there were two different paving contractors, we believe they used similar methods and sequence of paving operations as outlined below. From discussions with project personnel, we believe the contractor slip-formed 24-foot wide mainline pcc pavement and 16-foot wide pcc interchange ramps. As soon as possible thereafter but no later than 24 hours after placing the new pcc, an initial saw cut 5/16-inch wide and 3-1/4 inch deep was made at the contraction joints. The contractor then placed a 3/8-inch backer rod in the saw cut to keep out incompressibles. This was later removed when the final saw cut was made. Final sawing of the transverse contraction joints in the mainline pavement was done before the pcc shoulders were placed. Final reservoir shape was 5/8-inch width by 1 1/2-inch depth. The contractor placed backer rod and silicone in the joint, however, the specifications did not require him to seal the edges of the transverse pcc joints at the mainline/shoulder joint. As shown in Section B-B of the attached drawing, the

.trong possibility existed there was an opening below the backer rod at the transverse contraction joint/shoulder edge. Also, in many cases, a vertical crack below the contraction joint had formed and was not sealed before the adjacent shoulder was placed.

Since the State earlier had some concerns over proper consolidation of the pcc at the mainline/shoulder longitudinal joint, the contractors placed vibrators near this edge and may have increased the number of rpms on the vibrators. This could have permitted intrusion of mortar into the opening below the backer rod and in the vertical crack as the shoulder paving operation progressed past the mainline transverse contraction joint. Inspections of some joints indicate this mortar had typically entered the opening below the backer rod about 4-5 inches but in some cases (high side of super-elevated sections) this intrusion was as much as 7-8 inches. The mainline pavement was placed during June-July 1987. Most of the interchange ramps were slip-formed in August-September 1987. The pcc shoulders were placed on the mainline pavement during September-October and on the ramp sections during October-November, 1987. The longitudinal joints between the mainline and shoulders were saw cut to a depth of 1 1/2 inches and 5/8-inch width.

Project personnel stated that in some places, the contractor's saw blades during the final saw cut were hardly touching the walls of the transverse contraction joints. In other words, it is possible the slabs had substantially contracted during cooler weather causing the joints to open.

ll joint sealing operations were completed before the Winter of 1987-88. here was no indication of any cracking in the pcc pavement during the Spring of 1988.

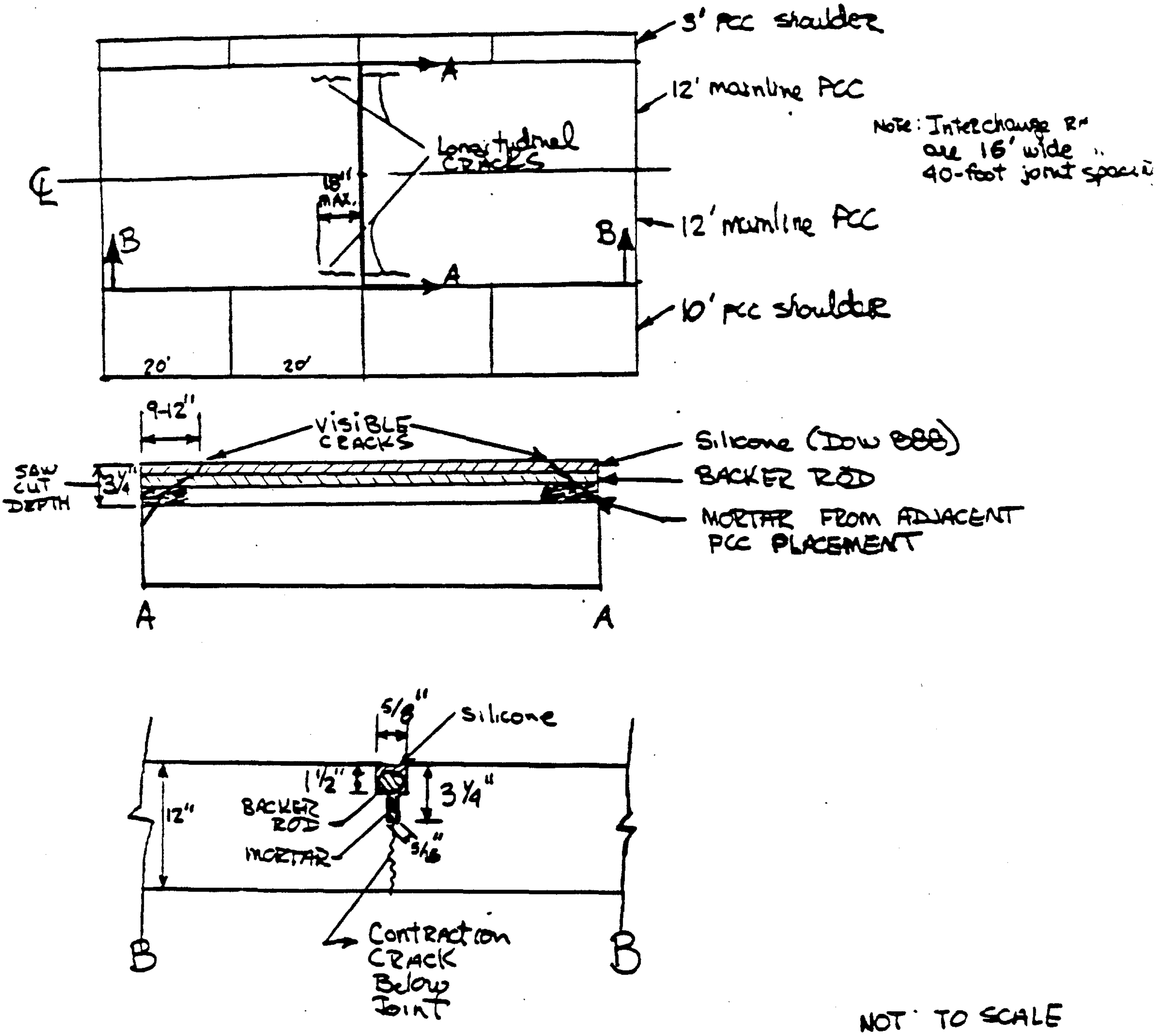
When the weather in May and June became very warm (highs in upper 80's and low 90's), cracks began to appear. It is believed the slabs were unable to properly expand due to the hardened mortar in the transverse contraction joints, causing the longitudinal cracking as shown in the attached drawing.

Solution:

It was agreed that all the transverse joints would be re-sawed at the mainline/truck lane and mainline/shoulder joints. It was felt whether a longitudinal crack was already present, it was imperative that each joint be sawed. A 5/8-inch wide by 6-8 inch deep saw cut was made. Care was taken to ensure the saw cut only extended about 6-7 inches into the mainline slab to prevent possible cutting of the dowel bars in the contraction joint. The new saw cuts were then cleaned using a wire brush mounted on a portable saw chassis and air blasted clean. If any incompressible appeared to remain in the joint, a second saw cut was made in an attempt to further breakdown the incompressible. The joints were then resealed with backer rod and low-modulus silicone.

Areas that already cracked were visually examined and sounded to determine if delaminations had occurred. Partial depth patches were made in delaminated areas. Cracks that had not caused delaminations were sealed with high .olecular weight methacrylate.

All operations were completed by July 1, allowing ribbon cutting ceremonies to proceed without delay on July 15. To prevent future problems, the State intends to modify their plans and specifications to require the contractor to prevent the intrusion of any incompressible materials into the transverse contraction joints when placing adjacent pavement. Also, consideration is being given to requiring final saw cutting within a reasonable time after initial saw cutting to prevent the potential of slabs significantly contracting or expanding due to weather.



I-64 Longitudinal Cracking

U.S. Department of Transportation

Federal Highway Administration

Technical Advisory

Subject

PORTLAND CEMENT CONCRETE MIX DESIGN AND FIELD CONTROL

Classification Code	Date
T 5080.17	July 14, 1994

1. PURPOSE. To set forth guidance and recommendations relating to portland cement concrete materials, covering the areas of material selection, mixture design, mixing, placement, and quality control.

2. BACKGROUND

a. Each year approximately 46 million cubic meters of concrete are used in all highway construction. The vast majority of States use a prescription type specification for portland cement concrete, often specifying minimum cement content, maximum water cement ratio, slump range, air content, and many times aggregate proportions. Admixtures such as fly ash are incorporated into mixes as a part of the prescription.

b. This system has worked fairly well in the past but may change as emphasis is placed on performance based specifications. States have begun to reduce or eliminate the amount of inspection at concrete plants as automation has increased productivity.

3. MATERIALS

a. Portland Cement. The proper type of portland cement should be specified for the conditions which exist.

OPI: HNG-23

(1) Types I, II, III, IP, and IS are typically used in highway construction. Type I is used when no special circumstances exist. Type II is used when sulfate exposure conditions are present. Type III is used when high early strengths are required. The use of Types IP and IS result in lower early strength gains and can be substituted for Type I cement when early strength is not a concern. In addition to the above mentioned types, Types IV and V are sometimes used in highway applications to meet special conditions. Further information about these cements can be found in the book <u>Design and Control of Concrete Mixtures</u> published by the Portland Cement Association (PCA).

(2) It is recommended that the acceptance of portland cement be based on certification by the supplier. The certification should contain the lot number of the cement. The supplier's test results should accompany the certification or be available to the State. Verification samples should be taken and used as part of the acceptance system.

(3) If alkali aggregate reactivity (AAR) is a concern, a maximum alkali content of 0.6 percent should be specified. Some State highway agencies consider this amount too high and recommend smaller amounts. If AAR is a problem in the State, a review of a States' Materials Manual is suggested. See Concrete Distress Conditions Section for other remedies.

b. Aggregates. Aggregates make up 60 to 70 percent of the volume of concrete mixes. A significant portion of poorly performing highway concrete can be traced to aggregate quality problems.

(1) The fine aggregate should meet the requirements of the American Association of State Highway and Transportation Officials (AASHTO) M 6.

(2) The range for the gradation of fine aggregate is quite broad. The fineness modulus (FM), calculated using AASHTO T 27, can be used as a tool for assessing the variability of the fine aggregate gradation. The specifications should limit the range of the FM between 2.3 and 3.1 according to AASHTO M6 and the variation of the FM should not be more than 0.20 from the value of the aggregate source.

(3) The FM is a means to control the influence that fine aggregate has on workability and the air content of the mix and is sometimes specified in the mix design. Further information regarding FM can be found in the Federal Highway Administration's manual FHWA-ED-89-006 (Portland Cement Concrete Materials Manual).

(4) It should also be noted that to provide good skid resistance, the PCA recommends that the siliceous particle content of the fine aggregate should be at least 25 percent. Consideration should be given, however, to the possibility of alkali-silica reactions when this is done.

(5) The coarse aggregate should meet the requirements stated in AASHTO M 80. For most parts of the country the severe exposure requirements should be used which means the use of class A aggregate for structural concrete and class B aggregate for pavements. The following table contains some of the more common information provided by Table 1 in AASHTO M 80.

	Class A Aggregate	Class B Aggregate
Clay lumps and friable particles	2%	3%
Chert	3%	3%
Sum of clay lumps, friable particles and chert	3%	5%
Material finer than No. 200	1%	1%
Coal and Lignite	0.5%	0.5%
Abrasion	50%	50%
Sodium Sulfate Soundness	12%	12%

c. Water

(1) The water serves as a key material in the hydration of the cement. In general, potable water is recommended although some non-potable water may also be acceptable for making concrete. Water of questionable quality should be examined since this can effect the strength and setting time. The following criteria is contained in Table 1 in AASHTO M 157 and is based on control tests made with distilled water:

Test	Limits
Compressive strength percent of control tests at 7 days	90
Time of set deviation from control	1 hour earlier to 1.5 hour later

(2) Wash water can be used to make concrete providing the resulting concrete mix water meets the following criteria in Table 2 in AASHTO M 157:

Chemical	Limits
Chloride as percent of weight of cement for the following uses:	
prestressed concrete	0.06
reinforced concrete in moist environment exposed to chlorides	0.10
reinforced concrete in moist environment not exposed to chlorides	0.15
sulfates	3000 ppm
alkalis	600 ppm
total solids	50,000 ppm

(3) If there is any question about the water, it should be tested using AASHTO T 26.

(4) It should be noted that the American Concrete Institute (ACI) provides more stringent tolerances for total chlorides in the mix. The chloride content for wash water in AASHTO M 157 is recommended for total chloride content in ACI 201.2R 22.

d. Admixtures. Admixtures are typically placed in mixes to improve the quality or performance. They can affect several properties and can have a adverse impact on the mix if not used properly. To avoid possible problems, it is suggested that trial batches be made to evaluate the mix.

(1) Air entraining admixtures should be specified when concrete will be exposed to freeze/thaw conditions, deicing salt applications, or sulfate attack. Recommendations for air content are contained in paragraph 4d.

(a) A vinsol resin type admixture should be added when fly ash having a variable loss on ignition (LOI) content (between 3 percent and 6 percent) is present. This is because of the effect that fly ash's fineness and carbon content has on the air entrainment system. Fly ashes not having a variable LOI do not have an adverse impact on entraining agents and therefore vinsol resin type admixtures may not be necessary.

(b) The specifications for air entraining admixtures are contained in AASHTO M 154.

(2) Chemical admixtures include water reducers, retarders, accelerators, high range water reducers (superplasticizers), corrosion inhibitors and combinations of the above. The specifications for chemical admixtures are contained in AASHTO M 194.

(a) Mixes containing admixtures are permitted an increase in shrinkage and a decrease in freeze thaw durability (as indicated in Table 1 AASHTO M 194) in comparison with mixes having no admixtures.

(b) Admixtures are usually accepted based on preapproval of the material and supplier certification. Verification tests should be performed on liquid admixtures to confirm that the material is the same as that which was approved. The identifying tests include chloride and solids content, pH, and infrared spectrometry.

(c) Water reducers and retarders may be used in bridge deck concrete to extend the time of set. This is especially important when the length of placement may result in flexural cracks created by dead load deflections during placement.

Often water reducers and retarders may increase the potential for shrinkage cracks and bleeding. Because of these concerns, increased attention needs to be placed on curing and protection.

(d) High range water reducers can be used to make high slump concretes at normal water cement (w/c) ratios or normal range slumps at low w/c ratios. The primary concern with the use of these admixtures is the loss of slump which occurs in 30 to 60 minutes. Redosing twice with additional admixture is allowed by ACI 212.4R; however, redosing typically reduces air entrainment. Type F and G high range water reducers may also be used. Type G has the added advantage of containing a retarding agent.

1 If transit mix trucks are used to mix high slump concrete, it is recommended that a 75mm slump concrete be used at a full mixing capacity to ensure uniform concrete properties. If transit mix trucks are used to mix low w/c ratio concrete, it is recommended that the load size be reduced to 1/2 to 2/3 the mixing capacity to ensure uniform concrete properties. Admixture companies are recommending additional mixing time with low w/c mixtures instead of decreasing the size of the load. This may have detrimental effects on some properties of the concrete such as the degradation of the aggregate resulting from over mixing.

2 High range water reducers may also affect the size and spacing of entrained air. If Freeze-Thaw

testing as described by ASTM C 666 indicates this to be a problem, it is recommended that the air content be increased by 1½ percent.

(e) Calcium chloride, the most commonly used accelerator, has been associated with corrosion of reinforcing steel and should not be used where reinforcing steel is present. In addition to the corrosion problem calcium chloride also reduces sulfate resistance, increases alkali-aggregate reaction, and increases shrinkage. Calcium chloride should not be used in hot weather conditions, prestressed concrete, or steam cured concrete. In applications using calcium chloride, the dosage rate should be limited to 2 percent by weight of cement.

(f) Non-Calcium Chloride accelerators are available and can be used where reinforcing steel is present. However, care must be taken in selecting these since some may be soluble salts which can also aggravate corrosion.

(g) Calcium Nitrate, which can be used as a corrosion inhibitor, also can function as an accelerator. There are no consensus standards available for the use of this material. Manufacturer specification sheets should be consulted for proper use.

(3) Mineral admixtures include fly ash, ground granulated blast furnace slag, natural pozzolans, lime, and microsilica (microsilica is also known as silica fume). Currently all of these materials are being used as additives or to reduce cement contents. Mineral admixtures are accepted based on approved sources with certifications and verification samples.

(a) According to the American Society of Testing and Materials (ASTM) C 618 and AASHTO M 295 there are two classes of fly ash, class C and class F. Since variability in fineness and carbon content can affect air content, the optional uniformity specifications in AASHTO M 295 should be specified when air entrained concrete is used. Fly ashes with LOI values less than 3 percent will typically not affect air content. Vinsol resin air entrainment admixtures should be specified when fly ash with LOI higher than 3 percent is used.

1 Fly ash may be used as a supplement or a replacement and is typically limited to 15 to 25 percent. If it is used as a replacement, it replaces cement on a 1.0 to 1.2:1 basis by weight.

2 Fly ash can be used to increase workability, reduce permeability, and mitigate alkali silica reaction (ASR); some Class C can make it worse. Class F fly ash with a calcium oxide content less than 10 percent can be used to mitigate ASR and sulfate attack. Fly ash with a calcium oxide content greater than 10 percent should be used in concrete which will be subjected to sulfate attack only with verification testing. This percentage and fly ash classification should only be used as a guide; further qualification should be based on ASTM C 452.

3 The cementing action with fly ash is pozzolanic in nature. The pozzolanic reaction with fly ash stops at approximately 4° Celsius.

Precautions need to be taken when using fly ash in concrete at lower temperatures. It should also be noted that fly ash can reduce early strength development and, therefore, should be monitored closely.

(b) Ground granulated blast furnace slag specifications are contained in AASHTO M 302.

1 Ground granulated blast furnace slag (GGBFS) is a cementitious material and can be substituted for cement on a 1:1 basis by weight for up to 50 percent of the cement in the mix.

2 For fresh concrete using GGBFS, the air entrainment agent dosage may need to be increased. The workability and finishability typically are improved but in mixes having high cementitious material content, mixes can be sticky and difficult to finish. Bleeding may be reduced and setting time may be longer.

3 Ground granulated blast furnace slag can reduce sulfate attack, alkali-aggregate reactions, and permeability. The rate of strength gain is usually decreased and sensitive to low temperature.

(c) Microsilica specifications are contained in AASHTO M 307. Microsilica can be used as an admixture or as a replacement for an equivalent amount of cement to produce high strength concrete. Microsilica will reduce permeability and help reduce alkali-aggregate reactions.

1 Microsilica has been used as an addition to concrete up to 15 percent by weight of cement, although the normal proportion is 10 percent. With an addition of 15 percent, the potential exists for very strong, brittle concrete. It increases the water demand in a concrete mix; however, dosage rates of less than 5 percent will not typically require a water reducer. High replacement rates will require the use of a high range water reducer.

2 Microsilica greatly increases the cohesion of a mix, virtually eliminating the potential for segregation. However, the cohesion may cause mixes to be sticky and difficult to finish. It may be necessary to specify a higher slump than normal to offset the increased cohesion and maintain workability. In addition, microsilica in the mix greatly reduces bleeding; therefore, mixes which contain microsilica tend to have a greater potential for plastic shrinkage cracking. It is imperative to use the proper curing methods to prevent the surface water from evaporating too quickly.

4. PROPORTIONING. Most of the concrete placed in highway facilities in the United States are under severe exposure conditions. State highway agencies specify a recipe for concrete mixes which includes minimum cement content, maximum water-cement ratio, air content range, and minimum strength. These requirements are necessary to achieve durability, as well as strength.

a. The maximum aggregate size should be as large as possible. This reduces total aggregate surface area and results in lower cement demand. The maximum aggregate size should be limited to 20 percent of the narrowest dimension of a concrete member, 75 percent of the clear spacing between reinforcing steel, or 33 percent of the depth of a slab for unreinforced concrete.

b. The minimum cement content refers to all cementitious and pozzolanic material in the concrete, including cement and any mineral admixtures that are being added to or substituted for cement. Replacement rates should be based on those contained in paragraph 3d(3).

(1) The PCA recommends a minimum cement content of 335 kg/m^3 for concrete placed in severe exposure conditions and ACI 316R recommends a minimum cement content of 335 kg/m^3 for concrete pavements in all locations unless local experience indicates satisfactory performance with lower cement contents. Even if strength requirements can be met with a lower cement content, a minimum cement content of 335 kg/m^3 should be used unless it can be demonstrated that the concrete will be durable.

(2) In cases where local experience allows a reduction in cement content below 335 kg/m^3 the cement content should not be reduced below the following minimum cement contents recommended by ACI 302.1R Table 5.2.4 for concrete slab and floor construction. The minimum cement contents listed below are based on the nominal maximum size of the aggregate. The cement content decreases as the nominal maximum aggregate size increases due to the decrease in aggregate surface area.

Nominal maximum size aggregate, mm	Cement content kg/m^3
37.5mm	280kg/m^3
25mm	310kg/m^3
19mm	320kg/m^3
12.5mm	350kg/m^3
9.5mm	365kg/m^3

(3) Low strength concrete in the field should not be addressed by arbitrarily increasing the cement content since an increase in cement content will increase the water demand leading to higher shrinkage and permeability. All changes in mix proportions should be evaluated with a trial batch.

c. The water-cement ratio in all cases should be as low as possible while maintaining workability. For freeze thaw resistance the following maximum water cement ratios are recommended in ACI 201.2R.

Thin sections (bridge decks, pavements and curbs) and sections with less than 25 mm cover and concrete exposed to deicing salts	0.45
all other structures	0.50

The water-cement ratio should include the weight of all cement, pozzolan, and other cementitious material.

d. The air content in the mortar fraction of the mix should contain approximately 9 percent air for concrete mixes exposed to severe conditions.

(1) The following recommendations are from ACI 201.2R Table 1.4.3.

Nominal maximum size aggregate, mm	Air content Percent
37.5mm	5-1/2
25mm	6
19mm	6
12.5mm	7
9.5mm	7-1/2

(2) The specified tolerance for air content should be ± 1½ percent.

5. <u>PROPERTIES OF CONCRETE</u>. Trial batches should be performed on all mixes at the expected placement temperatures. This is especially true for mixes containing multiple admixtures.

a. Workability. A concrete mix must be workable to ensure proper consolidation and finishing. The workability of a mix is a function of the gradation of the aggregate, amount and type of admixtures, water content, concrete temperature, and time. Once a workable mix is established during the trial batch process, slump can be used to monitor the consistency and uniformity of the mix. Slump, by itself, is not a measure of workability.

b. Durability

(1) Freeze-thaw durability depends on durable aggregates, proper air entrainment, low permeability, and a low water-cement ratio.

(2) D-cracking is strictly a pavement durability problem and is associated with aggregates. It should be addressed with the source approval of the aggregates.

(3) Alkali aggregate reactions are mostly the result of the alkali content of the cement in the concrete. The most common alkali aggregate reaction is associated with silicious aggregates although reactions have occurred with carbonate materials. If a reactive aggregate is encountered, several options are available: not using the source of aggregate, using a low alkali cement, using fly ash, or using microsilica. If alkali reactive aggregates are used, testing should be performed with the mix prior to its use to ensure a durable concrete.

(4) Resistance to or susceptibility to sulfate attack depends on the chemical composition of the cementitious portion of the concrete. Sulfate attack can occur from ground water, deicing salts, or sea water. Type II or Type V cement or some fly ashes, may be used to mitigate the problem.

c. Strength. The strength requirement is the compressive strength, f'_c, at 28 days. This must be equal to or exceed the average of any set of three consecutive strength tests. No individual test (average of two cylinders) can be more than 3.5 MPa below the strength requirements in the specification.

6. <u>MIXING, AGITATION, AND TRANSPORTATION</u>

a. In order to ensure proper operation, a concrete plant must be calibrated and inspected. Plant approval should include all the items covered in the Checklist for Portland Cement Concrete Plant Inspection (Attachment 1). This same checklist also discusses the inspection of truck mixers. The plant certification program operated by the National Ready Mix Concrete Association covers the same information contained in the attachment.

b. The mixing time for central mixers and approval of truck mixers should be determined by the uniformity test discussed in AASHTO M 157, Ready Mixed Concrete. The test is based on the comparison of tests on samples taken at the first and last 15 percent of the load. The following are maximum permissible differences to consider the mix properly mixed.

Test	Maximum Difference
Unit weight (air free basis)	15 kg/m^3,
Air content	1 percent
Slump	
less than 100mm	25mm
100 to 150mm	37.5mm
Coarse aggregate content	6.0 percent
Unit weight of air free mortar	1.6 percent
Compressive strength (7 day)	7.5 percent

c. Water added at the job site must be measured accurately. A water meter is the most accurate method for determining the amount of water added to the mix.

d. The recommendations for testing appear in paragraph 11, Quality Control and Testing, of this document.

e. The haul time should be limited to 90 minutes for truck mixers that agitate the mix and 30 minutes for trucks that do not agitate the mix. The maximum number of revolutions for truck mixers should be limited to 300.

f. No admixtures or water should be permitted to be added to the mix after the mixer has started unloading.

7. PLACEMENT AND CONSOLIDATION

a. Prior to placement of the concrete an inspection should occur covering the items in either the checklist for the placement of structural concrete (Attachment 2) or the checklist for the placement of concrete paving (Attachment 3).

b. Acceptance testing for pumped concrete should occur at the discharge end of the pump.

c. Aluminum pipe and chutes should not be used in concrete pumping operations.

d. Concrete can be conveyed to the location of placement by several commonly used methods including pumps, belt conveyors, buckets, chutes, and dropchutes. Care should be taken to ensure that there is no debris or blockages that will hinder or influence the properties or flow of the material. Concrete should not be allowed to free fall from distances greater than 1.2 meters to avoid segregation.

e. All concrete should be accompanied to the project with a delivery ticket. A sample delivery ticket appears as Attachment 4.

f. The proper consolidation of concrete is a significant factor in the ultimate performance of the concrete and it is achieved through vibration.

(1) The following are recommended frequencies for vibrators from ACI 309.

Diameter of head, mm	Frequency vibrations per minute
20 to 40 mm	10,000 - 15,000
30 to 65 mm	9,000 - 13,500
50 to 90 mm	8,000 - 12,000

8. CURING AND PROTECTION

a. Curing

(1) Curing is performed to maintain the presence of water in concrete and to provide a favorable temperature for cement hydration. Methods of curing include ponding, spraying, and fogging with water, wet covers such as burlap, plastic sheets, membranes, and the use of steam, electric forms, or insulation.

(2) The application rate of a particular curing compound should be based on the rate established during the approval process of the curing compound. The AASHTO M 148 indicates that a rate of application of $5m^2$/liter should be used for testing the material if no other rate is specified.

b. Protection

(1) Cold weather protection should be required when it is expected that the daily mean temperature for three consecutive days will fall below 4° Celsius. The following recommendations are for the minimum temperatures for delivered concrete as they appear in AASHTO M 157.

Air Temperature	Minimum Concrete Temperature Thin	Thick
-1 to 7°C	16°C	10°C
-18° to -1°C	18°C	13°C
Below -18°C	21°C	16°C

Thin sections are defined as those less than 300 mm.

(2) Concrete should never be placed on a frozen subgrade. Care should be taken to assure that the subgrade is free from frost.

(3) Hot weather conditions can be defined as a condition of high temperature, low humidity, and high winds. The existence of these conditions can be determined by finding the evaporation rate described in ACI 305 and included in Attachment 5. An evaporation rate exceeding 1 $kg/m^2/hr$ has the potential of causing plastic shrinkage cracks. The evaporation rate is a function of concrete temperature, ambient temperature, relative humidity, and wind velocity. This chart has been incorporated into several State specifications. It may not completely apply in all cases, especially in mixes containing admixtures which reduce the amount of bleeding.

(4) In addition to the plastic shrinkage cracking problem, ultimate strength will decrease with higher temperatures. The ACI has not recommended a maximum concrete temperature since strength loss can be compensated for by other means.

However, significant strength loss occurs above 32°C. Due to the strength loss and increase in potential for plastic shrinkage cracking, many States have set a maximum ambient placement temperature of 32°C. In all cases, trial batches should be performed at the highest expected temperature to ensure that the concrete will have the desired properties.

9. <u>CONCRETE DISTRESS CONDITIONS</u>

a. Alkali aggregate reactivity can be one of two types, alkali-silica and alkali-carbonate. The most prominent problem is cracking of the concrete due to the alkali-silica reaction (ASR).

(1) A widely used test to determine ASR is ASTM C 227. The current test criteria allow a maximum expansion of 0.05 percent at 3 months and 0.1 percent at 6 months. Research by PCA indicates that the critical criteria is 0.1 percent ultimate expansion. Since some reactions take longer than others, testing should continue as long as expansion is occurring. Some aggregates may take several years to show expansion.

(a) Recently the Strategic Highway Research Program developed a test which can be used for rapid determination of ASR. It is called the Gel Fluorescence Test and can be performed easily and inexpensively by field personnel. With this test, a 5 percent solution of uranyl acetate is applied on the concrete surface. Ultraviolet light is then used to illuminate the surface and if ASR exists, a yellow-green fluorescent glow will appear. Some safety concerns may be associated with this test so proper precautions are recommended. It should also be noted that the test is limited to preexisting concrete and not to fresh concrete.

(b) Alkali-silica reaction can be mitigated by limiting the alkali content of portland cement to 0.6 percent, by using class F fly ash or microsilica admixtures, or by reducing the water to cement ratio. The success of this approach may be limited; therefore, laboratory testing should be conducted. Protecting the final structure from moisture also reduces ASR.

(c) Although PCA recommends 25 percent of the fine aggregate be siliceous material to improve skid resistance, the use of some siliceous material can promote the ASR reaction and requires care to ensure this will not occur.

(2) Alkali-carbonate reaction (ACR) may occur with dolomitic limestones which contain large amounts of calcite, clay, or silts. ASTM C 586 is used to screen dolomitic materials for alkali-carbonate reactions.

b. D-cracking occurs when freeze-thaw conditions combine with saturated concrete made from susceptible coarse aggregates. The problem is only associated with pavements. Some dolomites and limestones are susceptible due to their pore structure.

(1) The most common test for predicting D-cracking susceptible aggregates is AASHTO T 161. There are two methods contained in the procedure. In method A the specimens are immersed in water for freezing and thawing. In method B the specimens are frozen in air and thawed in water. The number of freeze thaw cycles varies between 300 to 350. The minimum durability factor specified by the States range between 80 and 95. Some States have also specified a maximum expansion criteria range between 0.025 percent and 0.06 percent. It should be noted that the test method allows a significant range of time for freezing and thawing cycles. This can account for the variation in the criteria used by the States. Care needs to be taken when establishing criteria so that it will correspond to the test equipment and the history of performance of the aggregates.

(2) The hydraulic fracture test developed under SHRP may be able to provide a determination of the D-cracking susceptibility of aggregates in only about 1 week compared with the 8 weeks for T 161. In this test, dry aggregates are submerged in a pressure chamber and the pressure is increased to force water into the pores. After releasing the pressure, D-cracking susceptible aggregate will fracture as the water is forced out of the pores.

10. MANUFACTURED CONCRETE PRODUCTS Concrete products consist of structural elements constructed at a plant and trucked to the jobsite. These precast products typically consist of beams, pipes, barriers, poles and other special elements. The criteria outlined within this document apply to these products as well. Additional information about prestressed products are contained in the Checklist for Prestressed Concrete Products in Attachment 6.

11. QUALITY CONTROL AND TESTING

a. All testing should be performed by certified technicians. The ACI and the National Institute for Certification in Engineering Technologies (NICET) administer a concrete technician certification program. Guidance for establishing a certification program for testing personnel appears in a FHWA paper titled "Laboratory Accreditation and Certification of Testing Personnel."

b. Process control testing should be performed on aggregate moisture content, aggregate gradation, air content, unit weight, and slump at the plant.

(1) The specifications should require that the contractor provide a process control plan. The State should also provide guidance on the minimum requirements for a process control plan. As a minimum, the process control plan should include the information contained in Attachment 7.

(2) All process control tests should be plotted on control charts. Control charts are a good visual tool for discovering trends quickly before major problems occur.

c. The acceptance procedures should include monitoring of the process control activities including aggregate gradation testing. In addition, acceptance testing at placement would include slump, strength, and air content. Close monitoring of the water-cement ratio is also required since this will ultimately affect the durability and strength of the concrete.

Additional information on acceptance procedures is provided in the Technical Advisory on Acceptance of Materials T 5080.11.

d. It is recommended that compressive strength be accepted using statistical criteria (based on average strength and standard deviation) to ensure that the strength, f'_c, at 28 days, is equal or exceeded by the average of any set of three consecutive strength tests. No individual test (average of two cylinders) can be more than 3.5 MPa below the specified strength. There are two strengths to be considered. One is the minimum specified strength (f'_c) which is a function of the structural requirements. The second is the average strength for mix design (f'_{cr}). The f'_{cr} must be higher than f'_c to ensure that the concrete will exceed the minimum specified strength. The following recommendations for f'_{cr} are from ACI 318.

(1) Unknown Standard Deviation

Specified compressive strength, MPa	Required average compressive. strength, MPa
f'_c	f'_{cr}
Less than 20MPa	$f'_c + 6.9$
20MPa to 35MPa	$f'_c + 8.3$
Over 35MPa	$f'_c + 9.6$

(2) Known Standard Deviation

For greater than 30 test results (one test result is the average of two cylinder breaks) f'_{cr} is the greater of the two values from the following equations.

MPa

$$f'_{cr} = f'_c + 1.4s$$
$$f'_{cr} = f'_c + 2.4s - 3.5$$

s = Standard deviation

(3) For 15 to 30 test results the standard deviation in the above formulas can be modified by the following factors.

No. of Tests	Modification factor for standard deviation
Less than 15	use table for unknown s
15	1.16
20	1.08
25	1.03
30	1.00

e. Air content and slump should be accepted based on an attribute system, i.e., pass/fail. The following is a recommended criteria.

Acceptance criteria	Air content deviation, %	Slump deviation, mm
Acceptable	< 1.5	< 25mm
Acceptable for trucks on the road	1.5 to 2	25 to 31.5mm
Reject	> 2	≥ 31.5mm

f. Testing procedures for resistance to freeze-thaw damage, deicing salt attack, and abrasion resistance are long and involved and do not lend themselves to testing on a routine basis. These tests are usually conducted to determine the durability of the concrete. It should also be noted that high strength concrete does not always insure durable concrete.

A R Kane

Anthony R. Kane
Associate Administrator
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Attachments

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CHECKLIST FOR PORTLAND CEMENT CONCRETE PLANT INSPECTION

1. Materials

   A. Cements and Mineral Admixtures (cement, fly ash, etc.)

      (1) Is evidence of cement or fly ash acceptability present (certification, test results)?

      (2) Are bins or silos tight and provide for free movement to discharge opening?

      (3) Are bins or silos periodically emptied to check for caking?

      (4) Plants should provide separate storage for each type of cement or mineral admixture being used. Are the materials being isolated to prevent intermingling or contamination?

   B. Aggregates

      (1) Does the plant display evidence of source approval?

      (2) Are aggregates stockpiled to prevent segregation and degradation? The preferred method of stockpiling is in layers. Cone shaped stockpiles will segregate.

      (3) Are stockpiles adequately separated to prevent intermingling?

      (4) Does the plant maintain separate storage bins or compartments for each size or type of aggregate? Are the aggregates tested for gradation and moisture content?

      (5) What is the surface underneath stockpiles? Soil or paved? Are the stockpiles covered?

C. Water

(1) Does the plant have an adequate water supply with pressure sufficient to prevent interference with accuracy of measurement?

(2) Is there any evidence or history of contaminants in supply?

D. Liquid Admixtures

(1) Is there evidence of source approval?

(2) Is the admixture and dispensing equipment protected from freezing, contamination, or dilution?

(3) How often are the admixture metering and dispensing equipment periodically cleaned?

2. <u>Batching Equipment</u>

A. Scales

(1) Scales should indicate weight by means of a beam with balance indicator, full range dial, or digital display.

(2) For all types of batching systems the weighing devices must be readable by the batchman and the inspector from their normal stations.

(3) Scales should be certified or should be calibrated with a certified scale.

(4) Ten 25 kilogram test weights should be available at the plant at all times.

(5) Scale accuracy should generally be within plus or minus .4 percent of the scale capacity.

(6) Water meters will need to be calibrated to 1 percent of total added amount.

B. Batchers

(1) Cementitious material should be weighed on a scale that is separate and distinct from other materials.

(2) Bins with adequate separation should be provided for fine aggregate and each size coarse aggregate.

(3) Weigh hoppers should not allow the accumulation of tare materials and should fully discharge into the mixer.

(4) Batchers should be capable of completely stopping the flow of material and water batchers should be capable of leak free cut off.

(5) Separate dispensers will be provided for each admixture.

(6) Each volumetric admixture dispenser should be an accurately calibrated container that is visible to the batchman from his normal position.

(7) Aggregate should be measured to plus or minus 2 percent of the desired weight, cement to 1 percent, water to 1 percent and admixtures to 3 percent.

(8) Semi-automatic and automatic control mechanisms should be appropriately interlocked.

3. <u>Mixing</u>

A. Stationary Mixers

(1) Mixers should be equipped with a metal plate that indicates mixing speed and capacity.

(2) Mixers should be equipped with an acceptable timing device that will not permit discharge until the specified mixing time has elapsed.

(3) Mixers are to be examined periodically to detect changes in condition due to accumulation of hardened concrete or blade wear. A copy of the manufacturer's design, showing dimensions and arrangements of blades, should be available at the plant at all times.

B. Truck Mixers

(1) Mixers should be equipped with a metal plate that indicates mixing speed, capacity, mixing revolutions, agitating speed and agitating capacity.

(2) Mixers should be equipped with a revolution counter.

(3) Mixers are to be examined to determine satisfactory interior condition, that is, no appreciable accumulation of hardened concrete and no excessive blade wear. A copy of the manufacturer's design, showing dimensions and arrangements of blades, should be available at the plant at all times.

(4) Charging and discharge openings and chutes should be in good condition.

4. Weather

A. Hot Weather

(1) When concreting during hot weather, is plant equipped to cool ingredients? Is equipment available to produce acceptable ice?

(2) How are aggregates cooled? If by sprinkling, is provision made to account for excessive water?

B. Cold Weather

(1) When concreting during cold weather, is plant equipped to heat ingredients to produce concrete of applicable minimum temperature.

CHECKLIST FOR STRUCTURAL CONCRETE

1. TREATMENT OF FOUNDATION MATERIAL

Has special care been taken not to disturb the bottom of any foundation excavation?

2. CURING

Is the concrete being cured for 7 days, by one of the following methods?

(a) Waterproof paper method

(b) Polyethylene sheeting method

(c) Wetted burlap method

(d) Membrane curing method

3. REINFORCEMENT BAR STORAGE

Are all delivered rebars being stored above the ground upon skids, platform, or other supports? A light coating of rust will not be considered objectionable.

Are epoxy coated bars being stored on padded supports and handled to prevent damage to the bar coating?

4. FORMS

Are the forms clean, braced, tight, and sufficiently rigid to prevent distortion?

When wooden forms are used, are they dressed lumber or plywood and oiled prior to rebar placement?

Are all sharp corners in forms being filleted with 20 millimeters molding, unless otherwise specified?

5. REINFORCEMENT BAR PLACEMENT

Are all reinforcement bars tied securely in place? Are epoxy coated bars being tied with plastic or epoxy coated tie wire?

When epoxy coated bars are cut in the field, are they being sawed, sheared, or cut with a torch? Cutting with a torch is not acceptable. If cut in the field, the bars should be repainted at the cut ends with a similar type of epoxy paint.

Are at least 50 percent of the bar intersections being tied?

Are all rebar laps of the specified length?

Are all portions of metal bar supports in contact with any concrete surface galvanized or plastic coated? Are epoxy coated bars being supported with plastic, plastic coated, or epoxy wire chairs?

Are the reinforcement bar support in sufficient quantity and adequately spaced to rigidly support the reinforcement bars?

After epoxy coated bars are in place, are the bars inspected for damage to the coating and is the contractor repairing all scars and minor defects using the specified repair materials?

Is the finishing machine being used to detect high bars by making a "dry run" over the length of the deck prior to concrete placement? Is the proper coverage being maintained between the bars and any form work or surface, top, side, and bottom?

6. <u>PRE-POUR INSPECTION</u>

Prior to the placement of the concrete have the reinforcement bars, construction joints, and forms been cleaned of mortar, dirt, and debris?

Are the strike-off screeds set to crown, and other equipment on the job-site (such as vibrators) in good working condition?

7. <u>USE OF RETARDING ADMIXTURE (BRIDGE DECK)</u>

If the specified temperature is reached, is a retarding admixture being used in the bridge deck concrete?

8. TEMPERATURE CONTROL

Are proper precautions being taken for hot and cold weather concrete?

If outside temperatures warrant it, are temperature checks of the plastic concrete being taken?

9. TIME OF HAUL

Is all concrete that is being hauled in truck mixers being deposited within 90 minutes from the time stamped on the tickets?

If central-mixed concrete is hauled in nonagitor trucks, is the concrete being deposited within 30 minutes?

10. REVOLUTIONS

Have 70 to 100 mixing revolutions at mixing speed been put on the truck at the required speed (6-18 RPM)?

Have 30 mixing revolutions been placed on the truck at the required speed (6-18 RPM) after water has been added at the site?

Is the agitating speed between 2-6 RPM?

Are total number of revolutions being limited to 300?

11. CONCRETE DELIVERY TICKET

Are all truck tickets being properly completed, collected, and retained?

12. WATER CONTROL

Is all water that is being added to the mix accounted for and checked to ensure the w/c ratio is not exceeded?

13. AIR CONTENT DETERMINATION

Are air content tests being performed according to the required frequency?

14. SLUMP TEST

Are slump tests bring performed according to the required frequency?

15. STRENGTH TEST

Are concrete test specimens being cast at the site of work as per the required frequency?

16. PLACING CONCRETE

Is the concrete being deposited as near its final position as possible? (Moving concrete horizontally with vibrators is not permitted.)

Is the concrete being bucketed, belt conveyed, pumped, or otherwise placed in such a manner as to avoid segregation and is not being allowed to drop more than 1.2 meters?

17. CONSOLIDATION

Is all the concrete being consolidated with hand operated spud vibrators while it is being placed?

18. FINISHING (DECKS)

Is a finishing machine (having at least one reciprocating, nonvibratory screed operating on rails or other supports) being used to strike off and screed the bridge deck?

19. STRAIGHTEDGE TESTING AND SURFACE CORRECTION (DECK)

Is the plastic concrete being tested for trueness with a 3 meter straightedge held in contact with the slab in successive positions parallel to the centerline?

Are all depressions being immediately filled and all high areas being cut down and refinished?

20. SURFACE TEXTURING

Is the deck surface being textured with either a burlap drag or an artificial turf drag followed by tining with a flexible metal comb?

CHECKLIST FOR PORTLAND CEMENT CONCRETE PAVING

1. SUBBASE TRIMMING

   Has the subbase been trimmed prior to paving?

2. PAVING FORMS (IF USED)

   Are the forms: metal, not less than 3 meters in length, equipped with both pin locks and joint locks, within 2 millimeters along the length of its upper edge, within 7.5 millimeters along the length of its front face, and in sufficient supply.

   Is the height of form face at least the edge thickness of proposed pavement, the base width equal to or greater than the height, and are three steel pins being used to secure each section?

   Are the forms being set on a hard and true grade, built up in 12.5 millimeters maximum lifts of granular material in low areas (without using wooden shims) and oiled prior to the placing of concrete?

   When wooden forms are allowed, are they full depth, smooth, free of warp, not less than 50 millimeters thick when used on tangent, and securely fastened to line and grade?

   Are curved form of metal or wood being used on curves of 30 meters radius or less?

3. FORM ALIGNMENT

   Is the contractor checking the forms for line and grade and making necessary adjustments prior to concrete placement?

4. TEMPLATE

   Is the surface of the subbase being tested for crown and elevation by means of a template?

5. SUBBASE THICKNESS TEST

After trimming, is the thickness of the subbase being checked?

6. DRAINAGE

Is the subgrade being kept drained during all operations? Are all berms of earth deposited adjacent to the grade being kept drained by cutting lateral ditches through the berms?

7. LUG SYSTEMS (CONTINUOUSLY REINFORCED)

If concrete lug end anchorages are specified, are they staked and checked for dimensions and re-bar placement as shown in the plans?

Are they constructed of Structural Concrete at least 24 hours prior to pavement construction?

8. LONGITUDINAL JOINT KEYWAY AND BARS

Are the beginning and ending stations marked where adjacent curb, median, or pavement will necessitate the placement of keyway and/or bars in the edge of the proposed pavement?

9. SUPERELEVATION STAKING

Are the plan curb data examined for all curves to determine where to stake the beginning and ending stations for all superelevation transitions?

10. TEMPERATURE LIMITATIONS

Does the outside air temperature in the shade meet State specifications?

Does the temperature of the concrete meet State specifications at the time of placement?

11. REINFORCEMENT LAPPING

Are the locations and lengths of lap for bar or fabric reinforcement in conformance with the specifications.

Are all bar and fabric laps being tied?

12. TRUCK REQUIREMENTS

Is all concrete in a stationary mixer being deposited within 30 minutes when hauled in non-agitating trucks and within 90 minutes when hauled in agitator trucks?

Is transit mixed concrete being delivered and deposited within 90 minutes from the time stamped on the ticket?

If the contractor plans to use previously placed pavement as a haul road, are the truck weights checked to assure compliance with maximum weights permitted by State Law?

13. REINFORCEMENT PLACEMENT

Is the reinforcement being placed in accordance with one of the following methods?

Method A - After the full depth concrete is struck off the reinforcement should be placed into the concrete to the required depth by mechanical means.

Method B - The reinforcement should be supported on the prepared subbase by approved chairs having sand plates.

Method C - When the concrete is being placed in two layers the reinforcement should be laid full length on the struck-off bottom layer of concrete in its final position without further manipulation. (Cover within 30 minutes.) The depth of the first lift is 2/3 the depth of the pavement.

Method D - The reinforcement may be placed in the pavement using a method which does not require transverse steel or support chairs for support of the longitudinal steel. Tie bars at longitudinal joints are still required.

14. SEQUENCES OF FORM TYPE PAVING

Is all of the required concrete finishing equipment on the job and in acceptable working condition? Are the following sequences for form type paving being properly followed:

(a) Placing concrete. As little rehandling as possible. If equipment used can cause segregation, is the concrete being unloaded into an approved spreading device?

(b) Strike-off. Is the concrete being struck full width to the approximate cross section of the pavement?

(c) Consolidation. Is one pass of an approved surface vibrator or internal vibrator being made?

(d) Screeding. Are at least two passes with a machine having two oscillating screeds, and a finisher float being made?

(e) Straightedging - Are at least two 3 meter long shoulder operated or surface operated surface trueness testers (straightedges) being used?

(f) Surfacing Texturing - Are State specifications for texturing and tining being followed?

15. <u>SEQUENCES OF SLIPFORM PAVING</u>

When the contractor uses this optional method for the construction of the pavement are the following sequences being properly followed:

(a) Is the formless paver capable of spreading, consolidating internally, screeding and float finishing the newly placed concrete in one pass to the required line and grade?

(b) Is the pavement being straightedged, edged, and textured as required in the previous question 14?

(c) Does the contractor have available at all times metal or wooden sideforms and burlap or curing paper for the protection of the pavement in case of rain?

(d) Is the contractor immediately repairing all slumping edges in excess of 12.5 millimeters?

16. THICKNESS TEST

Is the thickness of the pavement being checked?

17. AIR CONTENT

Is the air content being tested as required by the frequency chart?

18. SLUMP

Is the slump being checked as required by the frequency chart?

19. REINFORCEMENT, DOWEL, AND TIE BAR DEPTH CHECKS

Is the concrete being probed to check the vertical and horizontal positioning of the pavement reinforcement, dowels, and tie bars?

20. STRENGTH

Are test specimens being cast at the site of work at the required frequency:

(a) at least one set per day
(b) one set for every 150 meters of two lane pavement (300 meters of one lane pavement)

21. LONGITUDINAL JOINT

(a) Are tie bars placed properly?

(b) Are the joints sawed at the same time as the transverse joints with pavement widths greater than 7.3 meters? Are they cleaned and immediately filled with sealer?

22. **TRANSVERSE JOINTS**

(a) Are the smooth dowel bars positioned parallel to the grade at a depth of ½ t.

Are the dowel bars coated with a thin bond breaker?

Are the capped ends of the bar coated with a debonding agent? (Expansion joints)

(b) Is a 1/3T deep groove being sawed over each assembly as soon as possible after concrete placement? Cleaned immediately?

(c) Are all joints being sealed after the curing period and before opening to traffic?

23. <u>TRANSVERSE CONSTRUCTION JOINTS (CONTINUOUSLY REINFORCED CONCRETE)</u>

(a) Are construction joints being placed at the end of each day's operation or after an interruption in the concreting operation of 30 minutes or more?

(b) Are construction joints being placed at least 1 meter from nearest bar lap?

(c) Are construction joints strengthened by supplementary 1.8 meter long bars of the same nominal diameter as the longitudinal steel so that the area of steel through the joint is increased by at least 1/3?

(d) Are construction joints formed by means of a clean (not oiled) split header board conforming to the cross section of the pavement?

(e) Is the concrete at construction joints being given supplemental internal vibration along the length of the joint both at the end of the day's operation and once again at the resumption on the next day? This is critical.

24. <u>TRANSVERSE CONSTRUCTION JOINTS (JOINTED PAVEMENT)</u>

(a) Are construction joints being placed at the end of each day's operation or after an interruption in the concreting operation of 30 minutes or more?

(b) Are construction joints being placed at least 3 meters from any transverse joint?

(c) Are construction joints being strengthened by epoxy coated dowel bars of the same size and positioning as specified for contraction joints?

Is a thin coating of bonding breaking agent applied to the dowels?

(d) Are construction joints being formed by means of a suitable header board conforming to the cross-section of the pavement?

25. SURPLUS - DEFICIENCY DETERMINATION

Is a daily check being made on the yield of produced concrete?

26. CURING

Are the pavement surface and edges being cured by one of the following methods:

(a) Waterproof Paper Method. Are the surfaces being covered as soon as possible with blankets or tear-free reinforced kraft paper, with 300 millimeter laps, properly weighted? Has the pavement been wetted with a fine spray first?

(b) Polyethylene Sheeting Method. Are surfaces covered as soon as possible with 30 meter long sheets of white polyethylene, with 300 millimeter laps, properly weighted? Has the pavement been wetted with a fine spray first?

(c) Wetted Burlap Method. Are surfaces covered as soon as possible with two layers of wet burlap, with 150 millimeter laps? Kept saturated by means of a mechanically operated sprinkling system or an impermeable covering? (Alternate: one burlap and one burlene blanket)

HAUL TICKET FOR TRUCK MIX CONCRETE

PROJECT NO. ____________________ DATE: ____________________
BATCHED FROM (PLANT) __________ TRUCK NO. ________________
NO. CUBIC METERS ______________ CLASS OF _________________
CONCRETE ______________________

BATCH WEIGHTS

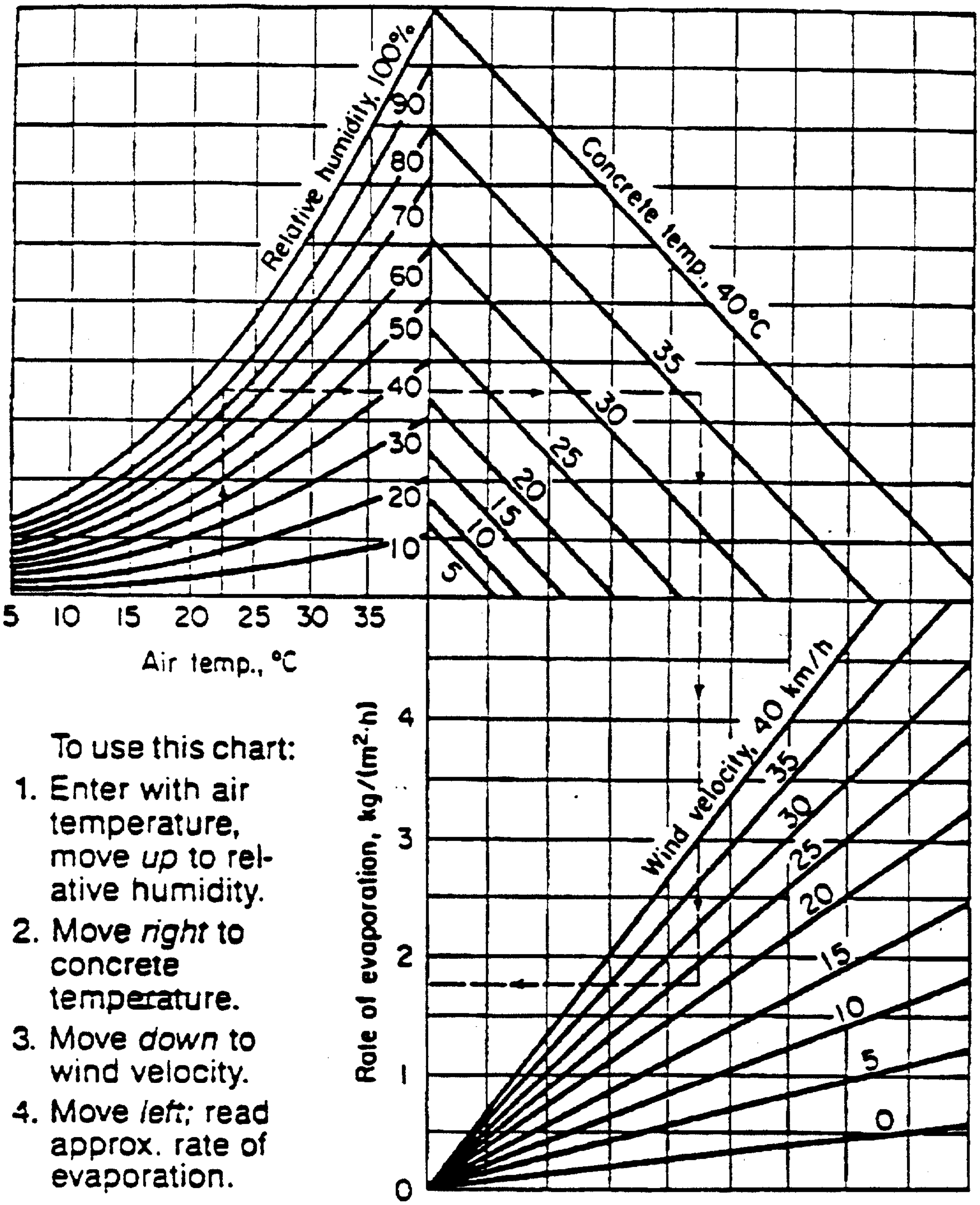
CEMENT BRAND ____________
kg ____________
FINE AGGR. SOURCE ________
kg ____________
COARSE AGGR. SOURCE ______
kg ____________
FLY ASH SOURCE ________
kg ____________

AIR ENTRAINMENT BRAND ________
grams ____________
RETARDER BRAND ____________
grams ____________
WATER REDUCER BRAND ________
ml ____________

WATER
MAXIMUM WATER ALLOWED, Liter ____________________
FREE MOISTURE
CA Liters ____________________
FA Liters ____________________
WATER ADDED AT PLANT Liters ____________________
MAXIMUM WATER THAT CAN BE ADDED AT THE SITE Liters ____________________

PLANT	SITE
TIME WATER ADDED TO MIX ________ AM/PM	TIME DISCHARED COMPLETED ________ AM/PM
	WATER ADDED AT JOBSITE Liters ______
	TOTAL WATER IN BATCH Liters ______
NUMBER OF MIXING	MIXING REVOLUTIONS AT SITE ______
	TOTAL NO. OF REVOLUTIONS
	SLUMP ______ AIR ______
	UNIT WEIGHT ____________
	CONC. TEMP ____________
	AIR TEMP ____________
Signature ____________	Signature ____________

NOMOGRAPH USED TO
DETERMINE EVAPORATION RATE



CHECKLIST FOR QUALIFICATION OF FACILITIES FOR PRESTRESSED CONCRETE PRODUCTION

1. Items which require written approval: (check applicable blanks)

   (a) Plans and computations of facilities ________

   (b) Concrete mix design (should include curves for 28-day strength) vs W/C Ratio: ________

   (c) Curing method ________

   (d) Epoxy-sand mortar, if used ________

   (e) Coal tar epoxy, if used ________

   (f) Water reducer-retarder ________

   (g) Design Engineer should be approved by State DOT ________

   (h) Gauge calibration should be certified ________

   (i) Computations regarding beam tests (2 weeks prior to testing) ________

2. What is length and capacity of stressing bed(s)

   Bed No. ______ Length ______ Capacity ______
   Bed No. ______ Length ______ Capacity ______
   Bed No. ______ Length ______ Capacity ______

3. Procedure of prestressing (pretensioning) and stress release:

   (a) Jacks, carriages, and struts are adequate to attain and maintain design stress.
   Yes ________ No ________
   Comments:
   ____
   ____

(b) Stressing of straight strands: (check applicable blanks)
Single strand method ________
Multiple strand method ________
Comments:
____
____

(c) Stressing of draped strands (check applicable blanks)
Single strand method _____
Multiple strand method _____
Final draped position _____ both ends _____
Partial draped position _____ one end _____
Comments:
____
____

(d) Single strand jack available.
Yes ________ No________

(e) Is an accurate dynamometer available for use in applying initial tension to the strands?
Yes ________ No ________

(f) What is proposed initial load to be applied ________ lbs.

(g) Is there a permanent, accurate linear gauge with which to measure elongation?
Yes ________ No ________

4. Forms: (Make comments in spaces provided)

(a) Metal
____
____

(b) True to shape and dimensions
____
____

(c) Adequate in number
____
____

(d) Condition and composition of bulkheads

____

____

(e) Type of hold-down device to be used

____

____

(f) Is provision being made to maintain 25 millimeter concrete cover over hold-down device?

____

____

(g) Are bulkheads and hold-down devices adequate to maintain dimensions of strand centers as shown on the plans?

____

____

5. Are facilities adequate for proper storage and handling of bridge members?
Yes ________ No ________

(a) Approximate available storage area ________________

(b) Condition of storage area ________________

6. Are facilities available for properly testing a member of the design type to be fabricated?
Yes ________ No________ (if No explain)

____

____

7. Are adequate lighting facilities available in the event that placing of concrete at night is necessary?
Yes ________ No ________

8. Vibrating equipment:

(a) Condition ______________________________

(b) Number to be used in placing ______________

(c) Two spaces available ______________

9. Source of Materials:

(a) Steel Wire and Strand (manufacturer)

____

(b) Cement (type and brand name)

____

(c) Coarse Aggregate (producer and location)

____

(d) Sand (producer and location)

____

(e) Retarder (brand name)

____

(f) Form Oil (type and name)

____

(g) Reinforcing Steel (producer)

____

10. Type of concrete mixing facilities: mixed at plant ______________
Ready Mix concrete ______________

(a) Are concrete batching facilities adequate to ensure good quality and sufficient quantity to avoid delays under all working conditions?
Yes ________ No ________

11. Testing equipment available: (check applicable blanks)

(a) Plastic cylinder molds ______________
No. Available ______________

(b) Slump Cone ______________

(c) Air content device ______________
(pressure ________ volumetric ________)

(d) Facilities for testing cylinders available at (proposed location)

____

12. Requirements for steam cure method:

(a) Three (3) recording thermometers available

____

(b) Temperature record charts

____

(c) Adequate temperature control valves

____

(1) What are the increments of spacing of control valves?

____

13. Are facilities available for proper protection and handling of component materials in storage? (Rate "S" if satisfactory, "U" if unsatisfactory, and "NA" if not applicable)

(a) Wire and/or strand ________
(b) Reinforcing steel ________
(c) Structural steel ________
(d) Cement ________
(e) Coarse Aggregate ________
(f) Sand ________

14. Is there a suitable shelter (at least 14 square meters floor space, facilities for lights, heat, desk(s), etc.) available for the inspector's use?

____

15. Personnel present during inspection of plants:

Producers/Contractors	Highway Department
________	________
________	________
________	________
________	________
________	________

GUIDE FOR QUALITY CONTROL PLAN FOR PORTLAND CEMENT CONCRETE

REQUIREMENTS

1. General Requirements:

The contractor should provide and maintain a quality control system that will provide reasonable assurance that all materials and products submitted to the State for acceptance will conform to the contract requirements whether manufactured or processed by the contractor or procured from suppliers or subcontractors or vendors. The contractor should perform or have performed the inspections and tests required to substantiate product conformance to contract document requirements and should also perform or have performed all inspections and tests otherwise required by the contract. The quality control inspections and tests should be documented and should be available for review by the engineer throughout the life of the contract.

2. Quality Control Plan:

The contractor should prepare a Quality Control Plan detailing the type and frequency of inspection, sampling and testing deemed necessary to measure, and control the various properties of materials and construction governed by the Specifications. As a minimum, the sampling and testing plan should detail sampling location and techniques, and test frequency to be utilized. The Quality Control Plan should be submitted in writing to the engineer at the preconstruction conference.

The Plan should identify the personnel responsible for the contractor's quality control. This should include the company official who will act as liaison with State personnel, as well as the Certified Portland Cement Concrete Technician who will direct the inspection program.

The class or classes of concrete involved will be listed separately. If existing mix designs are to be utilized, the Mix Design Numbers should be listed.

Quality control sampling, testing, and inspection should be an integral part of the contractor's quality control system. In addition to the above requirements, the contractor's quality control system should document the quality control requirements shown in Table 1. The quality control activities shown in Table 1 are considered to be normal activities necessary to control the production and placing of a given product or material at an acceptable quality level. To facilitate the States' activities, all completed gradation samples should be retained by the contractor until further disposition is designated by the State.

It is intended that sampling and testing be in accordance with standard methods and procedures, and that measuring and testing equipment be properly calibrated. If alternative sampling methods, procedures and inspection equipment are to be used, they should be detailed in the Quality Control Plan.

3. Documentation:

The contractor should maintain adequate records of all inspections and tests. The records should indicate the nature and number of observations made, the number and type of deficiencies found, the quantities approved and rejected, and the nature of corrective action taken as appropriate. The contractor's documentation procedures will be subject to the review and approval of the State prior to the start of the work and to compliance checks during the progress of the work.

4. Charts and Forms:

All conforming and non-conforming inspections and tests results should be kept complete and should be available at all times to the State during the performance of the work. Batch tickets and gradation data will be submitted to the State as the work progresses. All test data will be plotted on control charts. It is normally expected that testing and charting will be completed within 48 hours after sampling.

All charts and records documenting the contractor's quality control inspections and tests should become property of the State upon completion of the work.

5. Corrective Action:

The contractor should take prompt action to correct conditions which have resulted, or could result, in the submission to the State of materials and products which do not conform to the requirements of the Contract documents.

6. Non-Conforming Materials:

The contractor should establish and maintain an effective and positive system for controlling non-conforming material, including procedures for its identification, isolation, and disposition. Reclaiming or reworking of non-conforming materials should be in accordance with procedures acceptable to the State.

All non-conforming materials and products should be positively identified to prevent use, shipment, and intermingling with conforming materials and products. Holding areas, mutually agreeable to the State and the contractor, should be provided by the contractor.

7. Acceptance:

The State will monitor the performance of the contractor's quality control plan and will perform verification testing to ensure that proper sampling and testing procedures are used by the contractor. The State may shut down the contractors operations for failing to follow the approved process control plan. All acceptance testing will be performed by State personnel.

TABLE 1

CONTRACTOR'S QUALITY CONTROL REQUIREMENTS

Minimum Quality Control Requirement	Frequency
A. PLANT AND TRUCKS	
1. Mixer Blades	Prior to Start of Job and weekly
2. Scales	Prior to Start of Job and weekly
a. Tared	Daily
b. Calibrate	Prior to Start of Job
c. Check Calibration	Weekly
3. Gauges and Meters - Plant and Truck	
a. Calibrate	Yearly
b. Check Calibration	Weekly
4. Admixture Dispenser	
a. Calibrate	Prior to Start of Job
b. Check Operation and Calibration	Daily
B. AGGREGATES	
1. Fine Aggregate	
a. Gradation	21 Days
b. Deleterious Substances	Daily
c. Moisture	Daily
2. Coarse Aggregates	
a. Gradation	21 Days
b. Percent Passing No. 200 Sieve	Daily
c. Moisture	Daily
C. PLASTIC CONCRETE	
1. Entrained Air Content	One Per 1/2 Day Operation
2. Consistency	One Per 1/2 Day of Operation
3. Temperature	One Per 1/2 Day of Operation
4. Yield	One Per 1/2 Day of Operation

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Washington, D.C. 20590

Subject: Summary of State Highway Practices on Rigid Pavement Joints and Their Performance

Date: MAY 19 1987

From: Chief, Pavement Division

Reply to Attn of: HHO-13

To: Regional Federal Highway Administrators
Regions 1-10

The American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Construction is presently preparing a new edition of the AASHTO "Guide Specifications for Highway Construction." The AASHTO decided to survey the States' current practices on rigid pavement joints to help rewrite Section 514 titled "Joints." We agreed to assist them by preparing and then summarizing that survey (spacing, skew, dowel cages, epoxy coated bars, filler material, etc.). Attached for your information is a summary of the survey results. Please note there were five State highway agencies that did not respond to the survey.

The survey is intended to cover the States' current practices and recent performances with rigid pavement joints. It may not necessarily reflect each States's current standard specifications. However, we believe the summary contains worthwhile information that can be used as a reference tool for highway engineers.

We will appreciate your forwarding copies to the division offices. Copies of the survey have already been sent to the State highway agencies by the AASHTO Subcommittee on Construction. Any questions or comments may be directed to Mr. Don Voelker of my staff at FTS 366-1333.

Norman J. Van Ness

Norman J. Van Ness

Attachment

Quotations

(1). Inserts are no longer allowed as of 10/86.
(2). For CR pavements, there are four expansion joints @30 ft.; sealant is AASHTO M-33.
(3). Depends on a Bridge Movement Rating.
(4). Every second transverse joint is sawed within 4-12 hours.
(5). Sheet steel is used to form the keyway.
(6). Plastic coatings (17 mils) and powdered epoxy resins (7 mils) are also allowed.
(7). Only plain pavement joints are skewed at 2/12.
(8). Ravelling during sawing is not allowed but sawing must be done to preclude random cracking.
(9). Preformed bituminous, cork, or rubber plus compression seal.
(10). Only plain pavement is skewed at 2/12.
(11). Plastic coatings (11 mils) and red lead paint (no thickness specified) are also allowed.
(12). Yield strength of 40 ksi and ultimate strength of 70 ksi.
(13). Type A is low bond strength Doubl Coat by Republic Steel.
Type B is high bond strength, ie. Scotchkotz 202, Flintflex S31-6080, etc. but must have bond breaker HC-70, MS-2a or RC-250.
(14). At PC and PT of curves 2 deg. 30 min. and greater and at every eighth joint constructed between 9/15 and 4/15.
(15). Faulting occurs on plastic soils where dowels are not present.
(16). One coat of paint conforming to Federal Spec. TT-P-866 Type II or TT-P-645 or TT-P-310 or steel str. painting council spec. SSPC Paint II.
(17). Inside 4 ft.-13.3 ft.; centerline 3.3 ft.; outside 10 ft.-5.3 ft..
(18). Initial sawing is contractors option. Sawing for joint reservoir is a minimum of 72 hours.
(19). Sawing for preliminary crack control is done on approx. 50 ft. intervals with a 1/8-in. blade and a depth of D/4. Final sawing is done within 24-36 hours after concrete pour.
(20). New York's minimal problems related to slab cracking and joint spalling result from sawing too late. Faulting problems are present only in older pavements where a two-piece malleable iron load transfer device was used.
(21). Control joints (92-ft. intervals) are sawed as soon as possible with only minor ravelling allowed; remaining joints are sawed between 24-48 hours.
(22). Reinforced dowelled pavement is not sawed on skew. All others are at 2/12.
(23). Required but type not specified.
(24). Any grade of steel conforming ASTM A615 is permitted.
(25). (Concrete to Concrete) Low modulus silicone (cold) is preferred.
(Concrete to Asphalt) Hot rubberized asphalt ASTM D-3406 and ASTM D-3405.
(26). Rubberized asphalt over filler and/or polychloroprene compression seals.
(27). Either epoxy (7 mil thickness) or plastic (25 mil thickness) coatings are allowed.
(28). Initial sawing is 2 inches for plain pavement and 1 3/8 inches for plain dowelled pavement.
(29). Plain pavement initial saw depth is d/4. Plain Dowelled initial saw depth is d/3.

Quotations (con't)

(30). Alignment tolerances are plus or minus one-half inch of specified depth.
(31). Longitudinal sawed joints shall be cut before any equipment or vehicles are allowed on the pavement.

Other areas to be considered.

(1). California believes positive drainage mitigates transverse joint faulting.
(2). Georgia and Indiana believe a minor amount of ravelling during sawing is acceptable. If no ravelling is occurring, sawing has been delayed too long.
(3). Indiana reports formed groove-type contraction joints shall be used if early sawing causes erratic cracking.
(4). Iowa has a specification on maximum loading from the weight of saws. See attached.
(5). Kansas limits the use of inserts to the period May-Sept. to prevent longitudinal cracks.
(6). Louisiana believes transverse joint problems are attributable to moistu and incompressibles not the metnod of construction.
(7). Mississippi is experiencing transverse cracking on continuous reinforced pavements.
(8). New Jersey believes TRB Synthesis of Highway Practice 19 contains useful information.
(9). Ohio's keyed longitudinal joints have a proven poor performance.
(10). Delaware recommends that edges of construction joints shall be tooled to a 1/8 inch radius; sawed joints are chamfered similarly. Also, joints shall be thoroughly cleaned by brushing, air blasting, sand blasting, or other means to completely remove all foreign materials.
(11). Puerto Rico reports pumping problems due to no joint sealing caused by lack of proper maintenance.
(12). Colorado requires longitudinal sawed joints to be cut before any equipment or vehicles are allowed on the pavement. Also, every second transverse joint shall be sawed within 4 to 12 hours after pavement placement. The intermediate joints shall be sawed within 48 hours after pavement placement.

US Department of Transportation
Federal Highway Administration

Memorandum

Washington, D.C. 20590

Subject: Bondbreakers for Portland Cement Concrete Pavement with Lean Concrete Bases

Date: JUN 13 1988

From: Chief, Pavement Division

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators

During the past 2 years, we have reviewed several projects with Portland Cement Concrete (PCC) pavements constructed over lean concrete bases, which have experienced premature cracking. We have suspected that the principal cause of the distress was the partial bonding of the PCC slab to the lean concrete bases, during the period of joint and crack formation in jointed and continuous PCC pavements. Generally, this bond was believed to be weak, and would be lost within 6 to 12 months, because of stresses caused by loading and/or temperature variations. This weak bond would also be broken during coring or following the development of pavement distress. However, recently on two projects, cores were retrieved with the slab bonded to the lean concrete base. These projects which lend support to our theory are described below:

1. A Continuously Reinforced Concrete Pavement (CRCP) began experiencing premature punchouts. The pavement section consisted of 9 inches of CRCP over 6 inches of lean concrete base on a cement treated subgrade. During coring operations 5-plus years after construction, approximately 30 percent of the cores indicated the slab was bonded to the base. Failure of this pavement is believed to have resulted because the amount and location of steel was designed based on the unbonded condition. When bonding occurred, the slab was significantly under reinforced, and the reinforcement was located well above the neutral axis of the composite section. As a result, the steel was overstressed causing excessive crack widths, steel ruptures, and ultimately punchouts.

2. An 8-inch Jointed Plain Concrete Pavement (JPCP) over an 8-inch lean concrete base experienced random cracking within 6 months after construction. Coring revealed that the cracks were forming from the top of the slab downward, and were not reflective cracks. Also, cores of numerous sawed joints revealed that cracking had not occurred at the joints. Project records and discussions with project personnel indicated that sawing was done in a timely manner. There was no correlation between cracking, and temperature extremes at the time of construction. A number of the cores taken during the investigation of the cracking were retrieved with the slab bonded to the lean concrete base. We now believe that partial bonding during the joint formation period resulted in the saw cuts being an inadequate depth to force cracking at the joints. The depth of the saw cuts was based on the thickness of the slab in the unbonded condition.

We recognize that some States have been working in strengthening their asphalt concrete mix design and field control practices. These efforts are appropriate and continued involvement of all the field offices in encouraging conformance with the attached TA will be expected.

Other factors such as truck weights, high tire pressures, etc., also contribute to the rutting and stripping problems and we are working on these issues. We are convinced though that significant gains in solving rutting and stripping problems can be achieved by using quality materials and strengthening specifications and construction practices. We expect those States where rutting and stripping is a problem to include a priority effort to improve the design and construction of asphalt concrete pavements. The Pavement Division and the Construction and Maintenance Division are available upon request to provide technical support and guidance, which may be necessary in achieving these actions.

R. D. Morgan
R. D. Morgan
Executive Director

Attachment

The use of polyethylene sheeting is not recommended for use as a bondbreaker, because of construction problems which have occurred on projects where it was specified.

We are also currently evaluating the magnitude of slab curling on pavements, constructed over lean concrete bases. Actual field measurements of curling and deflection are being made on pavements in four States in Region 4. We believe the stiffness of the lean concrete base tends to cause higher curling stresses. In longer slabs, the combined curling and load stresses can exceed the slab strength resulting in transverse slab cracking. We suggest that to be on the safe side, when JPCP pavements are constructed over lean concrete bases, the joint spacing be limited to a maximum of 15 feet.

We intend to closely monitor the performance of PCC pavements over lean concrete bases, and would appreciate receiving feedback on the performance of this type of pavement in your region. Please contact Mr. John Hallin at FTS 366-1323, if you have any questions or comments on the use or performance of lean concrete bases.

Norman J. Van Ness