EARLY OPENING OF PCC PAVEMENTS TO TRAFFIC

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The objective of the study was to produce information from pavement load tests that will permit early loading of portland cement concrete (PCC) highway pavements by construction and/or public vehicular traffic. This ongoing study was undertaken to provide additional information and further develop guidelines proposed in a recent FHWA sponsored investigation. Full-scale load test data and calculated pavement responses were collected and analyzed. An in-depth dowel bar bearing stress analysis at early concrete ages was also made. Results were used to further develop early loading guidelines, refine the analysis of dowel bearing pressures, and evaluate the effects of early loading fatigue damage.

Deflection and strain data were collected from full-scale load tests slabs constructed and loaded at ages ranging from five to 26 days. Each slab section consisted of three pairs of two-lane slabs. Sections were constructed with a both an aggregate interlock and doweled joint. Slabs were either constructed over a granular subbase or lean concrete base. Slab thickness was 254 mm (10 in.). Strains and deflections due to a 90 kN (20 kip) single-axle load with dual wheels positioned at slab edges, longitudinal joints, transverse joints, and interior positions were measured at early ages. Results were compared to stresses and deflections computed using the ILLI-SLAB finite element computer program.

A detailed investigation of dowel bar bearing pressures was also conducted to evaluate maximum bearing pressures and pressure distributions. Modulus of concrete-dowel interaction computed using a three dimensional finite element computer program was input into the traditional Friberg analysis. Assumptions used in the Friberg analysis at early ages were modified based on the JSLAB finite element computer analysis.

INTRODUCTION

Newly-placed PCC pavements are often subjected to traffic shortly after initial curing but long before they have attained their design strength. Construction traffic may use pavements as a working platform to facilitate subsequent construction activities. Limits to user delays may require earlier opening times to commercial traffic in cases of intersection, joint (patching), or ramp reconstruction. Lighter construction equipment such as water trucks, concrete haulers, and joint sawing equipment also load concrete pavements at very early ages when concrete strengths are relatively low.

Issues regarding the early loading of concrete pavements that need to be addressed include:

- The required strength the pavement may be loaded by construction traffic without causing significant damage to the pavement.
- Damage done to a pavement if it is subjected to only a few repetitions of a heavy construction load.
- Evaluating damage caused by early loading by lighter construction equipment.
- Damage done by opening to public traffic.
- Guidelines in evaluating early opening times.

A method to evaluate fatigue damage at ages less than 28 days was developed in a recent FHWA study.(1) The evaluation of early loading damage incorporated factors commonly used in mechanistic pavement design procedures including concrete strength (modulus of rupture), elastic modulus, subbase/subgrade support, and thickness. Material properties at early ages (less than 28 days) were investigated. The early loading analysis and guidelines were refined using supplemental data developed in a separate study on early loading conducted for the Portland Cement Association (PCA).(2)

Based on the early loading fatigue analysis, guidelines were developed to determine opening times. A dowel bearing stress analyses was also conducted to evaluate bearing stresses at early ages. Guidelines were also established to minimize loss in load transfer capability due to early age loading of dowelled contraction joints.

FATIGUE ANALYSIS

A concrete pavement analysis was conducted to evaluate early opening fatigue damage. The fatigue analysis compared the actual or expected number of traffic load applications to the allowable number of loads that the pavement may sustain before cracking. The allowable number of traffic loads depends on the flexural stress and the flexural strength (modulus of rupture) of the slab.

Free Edge Flexural Stress

The early loading fatigue procedure is based on a stress analysis due to edge loads with the finite element computer program ILLI-SLAB.(3) The program allows considerations of slabs with finite dimensions, variable axle-load placement, and modeling of load transfer between pavement and concrete shoulder . Critical free edge flexural stresses were computed for 90 kN (20 kip) single- and 180 kN (40 kip) tandem-axle loads positioned at mid-span of a 4.6 m (15 ft) long by 3.7 m (12 ft) wide slab. Dual tires 127 mm (5 in.)wide spaced at 127 mm (5 in.) were assumed in the analysis. Tire contact area was 290 cm^2 (45 $in.^2$). Tandem axle spacing was set at 1.3 m (50 in.). Only the dual wheel assembly at the free edge were used in the analysis. Concrete modulus of elasticity was varied between 6895 and 34470 MPa (1 and 5 million psi) at 6895 MPa (1 million psi) increments. Slab thicknesses of 152, 203, 254, and 305 mm (6, 8, 10, and 12 in.) were used in the analysis. Subgrade support was characterized using a Winkler model with k-values (modulus of subgrade reaction) of 27, 81, and 136 MPa/m (100, 300, and 500 $lb/in.^3$) Stresses for single- and tandem-axle loads are computed as follows:

$$20 \text{ kip edge } SAL = 6 / t^2 \, [\, 4705.9 \log (L) - 3068.9 \,] \quad R^2 = 0.991 \; \ldots\ldots\; (1)$$

$$40 \text{ kip edge } TAL = 6 / t^2 \, [\, 731.1 \, \mathrm{sqrt}(L) - 759.9 \,] \quad R^2 = 0.996 \; \ldots\ldots\; (2)$$

$$L = \{ \, E t^3 / [\, 12 \, (1 - \mu^2) \, k \,] \, \}^{0.25} \; \ldots\ldots\; (3)$$

where

SAL = 20 kip single-axle load free edge flexural stress, psi
TAL = 40 kip tandem-axle load free edge flexural stress, psi
L = radius of relative stiffness, in.
E = modulus of elasticity, psi
t = slab thickness, in.
μ = Poisson's ratio (assumed 0.15)
k = modulus of subgrade reaction, $lb/in.^3$

For equal conditions of axle load, thickness, elastic modulus, and subbase support, single-axle load stresses are larger than tandem-axle load stresses. Tandem-axle load stresses are more sensitive to modulus of elasticity (square root function of L) than single-axle load stresses (log function of L).

A similar interior load analysis was made for axle loads placed 610 mm (2 ft) inward from the free edge at slab midspan. Interior stresses for single- and tandem-axle loads are computed as follows:

$$20 \text{ kip interior } SAL = 6 / t^2 [\, 468.3 \, \mathrm{sqrt}(L) - 410.9 \,] \quad R^2 = 0.997 \; \ldots\ldots\; (4)$$

$$40 \text{ kip interior } TAL = 6 / t^2 [\, 487.8 \, \mathrm{sqrt}(L) - 788.4 \,] \quad R^2 = 0.993 \; \ldots\ldots\; (5)$$

where

SAL = 20 kip single-axle load interior flexural stress, psi
TAL = 40 kip tandem-axle load interior flexural stress, psi

Effect of Tied Concrete Shoulders

Tied concrete shoulders can significantly reduce flexural load induced edge stresses. Shoulders at ages of less than 28 days not yet subjected to traffic should maintain a relatively high degree of joint stress transfer. Joint stress transfer is defined as:

$$LT_\sigma \% = (\, \sigma_u / \sigma_l \,) \times 100 \; \ldots\ldots\ldots\ldots\; (6)$$

where

LT_σ % = stress load transfer efficiency, percent
σ_u = unloaded slab (shoulder) stress
σ_l = loaded slab (slab) stress

A review of measured load transfer efficiencies on several projects at ages of less than 28 days indicted that stress transfer ranges from 70 to 74 and averaged 72 percent. For older pavements subjected to traffic and laboratory test specimens tested under dynamic loading the stress transfer ranged from 5 to 70 and averaged 24 percent. Stress transfer was estimated from load deflection efficiency data. The PCA thickness design method assumes a shoulder stress transfer efficiency of approximately 30 percent. The review of stress transfer efficiencies is summarized in Table 1.

A stress transfer of 70 percent was assumed in the analysis when tied concrete shoulders are used. Longitudinal flexural edge stresses at early ages incorporating the benefits of tied concrete shoulders are computed as follows:

$$\sigma = \sigma_{FE} / (1 + LT_\sigma \% / 100) \; \ldots\ldots\ldots\ldots\; (7)$$

where

σ = flexural stress
σ_{FE} = free edge stress (eqs. 1 and 2)
LT_σ % = stress load transfer efficiency (assumed 70 percent at early ages), percent

Subgrade Support Beyond Pavement Edge

Conventional theories on layered systems assume that subgrade-subbase layers do not exist beyond the pavement edge. The effect of support beyond the slab edge was evaluated with the MATS computer program.(10) For mature concrete with different k-values and slab thicknesses, the slab stresses ranged from 95 to 99 and averaged 96.7 percent of those determined without outside support.(11) The support factor is incorporated into the analysis only for pavements without tied concrete shoulders. With a tied concrete shoulder the support outside the shoulder edge is too far away from truck loads to have any effect.

Fatigue Consumption

The fatigue consumption approach is commonly based on studies of concrete fatigue and uses Miner's hypothesis that fatigue resistance not consumed by repetitions of one load is available for repetitions of additional loads. Theoretically the total fatigue life consumed should not exceed 100 percent (fracture). Fatigue damage is summed for each load or load category and is defined as:

Table 1 - Shoulder Stress Transfer Efficiencies

Source	Thickness, mm	Subbase Type	Joint Type	Age or Cycles	Stress Transfer, percent (1)	Ref. No.
I-15 (UT) Load Tests	254	lean concrete (102 mm)	tied	4 to 8 days (2)	70	1
		lean concrete (102 mm)	tied	1 year (2)	44	
I-90 MN Load Tests	228.6	granular	tied, keyed	initial	74 (3)	5
		granular	tied, keyed	6 years	21 (4)	6
I-90 MN Load Tests	203.2	granular	tied, keyed	initial	73 (3)	5
		granular	tied, keyed	6 years	29 (4)	6
I-74 IL Load Tests	177.8 (152 mm sh.)	granular	tied, keyed	10 years	70 (5)	7
		none	tied, keyed	10 years	20 (5)	
I-80 IL Load Tests	203.2	finer granular	tied	9 years	10 (5)	7
		granular	tied	9 years	5 (5)	
		none	tied	9 years	10 (5)	
Full Scale Tests	254	granular	tied	28 days	73	2
	254	LCB (127 mm)	tied	6 to 19 days	71	
Laboratory Specimen Tests	152.4	granular	tied	2.1 million	20 (5)	8
	152.4	granular	tied, keyed	1.6 million	20 (5)	
	152.4	granular	tied, smooth jt.	1.5 million	20 (5)	
	152.4	granular	tied, round key	3.3 million	25 (5)	
	152.4	cem. treated (76 mm)	tied, smooth jt.	2.6 million	25 (5)	
	203.2	granular	tied	3.9 million	25 (5)	
	203.2	granular	tied, keyed	2.2 million	20 (5)	
	203.2	granular	tied, smooth jt.	1.6 million	15 (5)	
	203.2	granular	tied, round key	3.0 million	20 (5)	
	152.4	cem. treated (152 mm)	tied, smooth jt.	1.7 million	15 (5)	
	152.4	cem. treated (152 mm)	tied	4.4 million	40 (5)	
PCA Design	102 to 305	****	agg. interlock	****	30 (6)	4

NOTES:
1. Stress tranfer efficiency = unloaded slab stress / loaded slab stress * 100%.
2. Not yet opened to public traffic.
3. Average 89 kN SAL (two sites) stress transfer.
4. Average for 89 kN SAL, 151 kN TAL, 187 kN TAL, and 187 kN tridem axle load (two sites) stress transfer.
5. Estimated stress transfer from deflection transfer using ref. 9 procedure.
6. Average for 80 kN SAL and 160 kN TAL, k= 27 to 136 MPa/m.

$$\text{Damage} = \sum (n / N) \quad (8)$$

where

Damage = proportion of life consumed
n = actual or expected number of traffic loadings
N= maximum allowable number of traffic l loadings before fracture

One commonly used fatigue model to determine the allowable number of loads is that used in the 1984 Portland Cement Association (PCA) pavement thickness design procedure.(4) The allowable number of load repetition for a given axle load is determined based on the stress ratio (flexural stress divided by third-point loading modulus of rupture). The number of allowable load repetitions is determined as follows:

Stress Ratio, SR	Allowable Repetition, N . (9)
> 0.55	$\log_{10} N = (0.9718 - SR) / 0.0828$
0.45 through 0.55	$N = [4.2577 / (SR-0.4325)]^{3.268}$
< 0.45	N = unlimited

Variations in Concrete Strength

Expected ranges of variations in concrete modulus of rupture have a significantly greater effect on early age fatigue life than the usual variations in other pavement material properties including subgrade support, subbase thickness, subbase strength, and layer thicknesses. Recognition of concrete material variability was incorporated into the early loading fatigue evaluation by reducing the modulus of rupture by one coefficient of variation.

A recent laboratory study of the modulus of rupture coefficient of variation at ages of 1, 3, 7, 14, and 28 days for four different mixes ranged from 0 to 23.9 and averaged 7.0 percent.(1) The study evaluated early age strengths for paving mixes using two coarse aggregate types and two cement contents. Two beam specimens were cured at 10, 22, and 38 °C (50, 72, and 100 °F) at 50 percent relative humidity. Specimens were also moist cured at 22 °C (72 °F). The standard deviation at each age for each curing condition and age was estimated from the range in strength.(12) In a separate early loading study conducted for PCA, the coefficient of variation of field cured beams ranged from 0.9 to 7.1 percent for two different mixes. Average coefficient of variation at ages ranging from 2 through 28 days was 3.9 percent.

The overall (two studies) average coefficient of variation was 6.6 percent for the 90 pairs of beams tested. The early loading evaluation assumed a 7 percent coefficient of variation in modulus of rupture to account for material variability.

EARLY LOADING FATIGUE GUIDELINES

Flexural stresses continually increase with increases in modulus of elasticity (increase in radius of relative stiffness). The modulus of rupture also increases with time. Since both stresses and strength increase with time at different rates, the stress ratio (stress divided by modulus of rupture) is also dynamic. Opening strength requirements may be different for commercial as opposed to construction traffic loads. Construction traffic opening strengths are of interest if the contractor uses pavements as a working platform to facilitate subsequent activities. Mitigation of user delays may require earlier opening times to commercial traffic in cases of fast-track paving, intersection reconstruction, patching, or ramp reconstruction.

Commercial and Public Traffic Opening Strengths

Free edge stresses were computed as shown in eq. 1 for an 90 kN (20 kip) SAL. Stresses were multiplied by the 0.967 factor to account for outside edge support if no tied shoulders are used or divided by 1.7 as shown in eq. 7 if tied concrete shoulders are concurrently cast with mainline pavement. To account for material variability stresses are then divided by 0.93 (7 percent coefficient of variation). An 90 kN (20 kip) SAL was used as the critical design load since single-axle load stresses are significantly greater than tandem-axle load stresses. Design load stresses as a function of elastic modulus are shown in Fig. 1 for a 254 mm (10 in.) thick slab. As shown in eq. 1, stress increases with the logarithm of modulus of elasticity raised to the 0.25 power. Percentage stress increases are relatively low compared to increases in elastic modulus. For the 203 mm (8 in.) slab with no shoulder, k of 54 MPa/m (200 lb/in.^{3}), and elastic modulus of 20685 MPa (3 million psi) the 90 kN (20 kip) SAL stress is 2.5 MPa (368 psi). Stresses at elastic moduli of 13790 and 34470 MPa (2 and 5 million psi) are 5 percent less

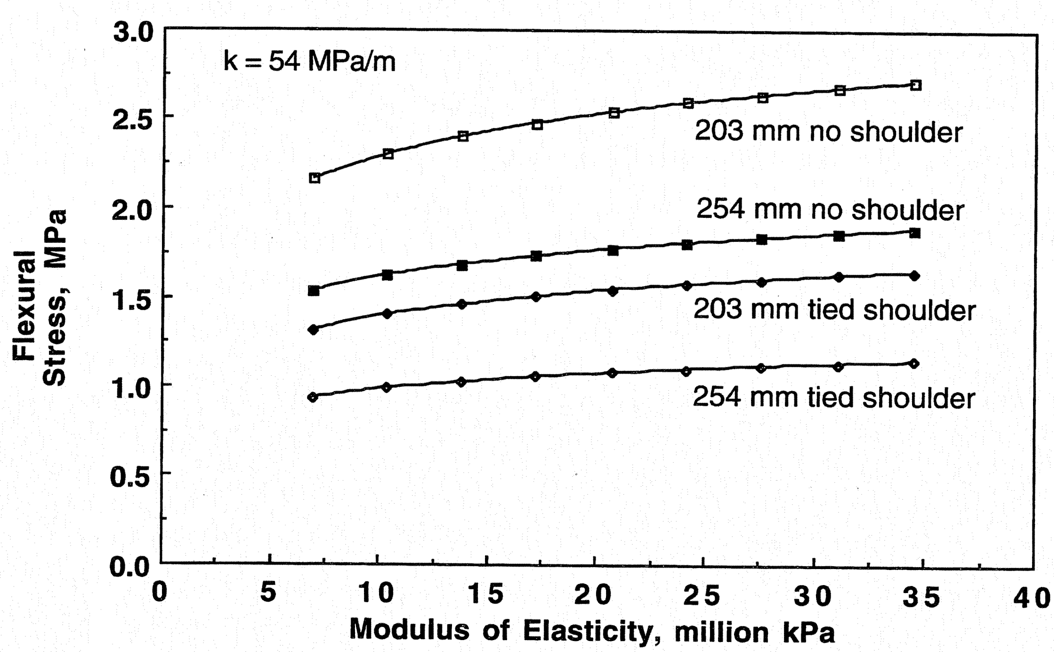

Fig. 1 - Design Flexural Stresses

and 4 percent greater, respectively. Increases in stress at higher modulus of elasticity would be offset by greater percentage increases in flexural strength. An early age modulus of elasticity of 20680 MPa (3 million) (early age) was assumed to simplify the early loading guidelines since stresses are relatively insensitive to elastic modulus.

A stress ratio of 0.50 was used as the opening criteria. For a stress ratio of 0.50, the allowable number of 90 kN (20 kip) SAL calculated using eq. 9 is 762,000. To consume 1 percent fatigue life 7620 axle loads would have to be applied. The fatigue life consumed at stress ratios of 0.50 are essentially zero since:

- Modulus of rupture increases are significantly greater at earlier ages which decreases the stress ratio.
- Loads applied directly at the free edge or shoulder joint are conservatively assumed (no lateral distribution).
- A relatively high single-axle load is assumed.
- A very conservative fatigue model used in design was assumed (eq. 9).

Modulus of rupture values to limit 90 kN (20 kip) SAL stress ratios to 0.50 were calculated incorporating effects of outside subbase support, tied shoulders, and strength variability. Results are summarized in Table 2. For a 254 mm (10 in.) slab with tied shoulders and subbase support of 81 MPa/m (300 lb/in.^{3}), the pavement can be opened to traffic when the flexural strength is a minimum of 2.0 MPa (295 psi). With no tied concrete shoulders the minimum opening strength is 3.4 MPa (490 psi).

Construction Traffic Opening Strengths

An analysis of a 64 kN (14,500 lb) spansaw positioned at the

Table 2 - Minimum Flexural Strength for Public Traffic

t, mm	k, MPa/m	L, (1) mm	Stress, MPa (2) no shoulder	MR, MPa (3) no shoulder	Stress, MPa (2) tied shoulder	MR, MPa (3) tied shoulder
203	27	859	2.58	5.55	1.57	3.38
	41	776	2.45	5.27	1.49	3.21
	54	723	2.36	5.07	1.43	3.10
	81	653	2.23	4.79	1.36	2.93
	136	575	2.07	4.45	1.26	2.69
229	27	939	2.13	4.59	1.29	2.79
	41	848	2.03	4.34	1.23	2.65
	54	789	1.95	4.21	1.19	2.55
	81	713	1.85	3.96	1.13	2.41
	136	628	1.72	3.69	1.05	2.24
254	27	1016	1.79	3.86	1.09	2.34
	41	918	1.71	3.65	1.04	2.24
	54	854	1.65	3.55	1.00	2.14
	81	772	1.56	3.38	0.95	2.03
	136	679	1.46	3.14	0.89	1.90
279	27	1091	1.53	3.28	0.93	2.00
	41	986	1.46	3.14	0.89	1.90
	54	917	1.41	3.03	0.86	1.83
	81	829	1.34	2.90	0.82	1.76
	136	730	1.25	2.69	0.76	1.65
305	27	1165	1.32	2.83	0.80	1.72
	41	1052	1.26	2.72	0.77	1.65
	54	979	1.22	2.62	0.74	1.59
	81	885	1.16	2.52	0.71	1.52
	136	779	1.09	2.34	0.66	1.41

NOTES:
1. Modulus of elasticity 20,700 MPa.
2. 88.96 kN SAL, 0.967 subbase support factor for no shoulders or 1.7 shoulder factor
3. Strength coefficient of variation of 7 percent.
4. 10 in. = 354 mm 100 pci = 27.145 MPa/m 100 psi = 0.69 MPa

center of a slab indicated that fatigue damage is insignificant at early ages.(1) Stresses were computed using relationships developed between flexural strength, compressive strength, and elastic modulus. Load induced stresses varied between 0.2 and 0.6 MPa (36 and 92 psi) for slab thicknesses of 203, 254, and 305 mm (8, 10, and 12 in.) with subbase support of 27 through 81 MPa/m (100 through 300 lb/in.^{3}). Modulus of elasticity ranged from 6895 to 34470 MPa (1 to 5 million psi). Stress ratios in the analysis ranged from 0.05 to 0.34 resulting in an unlimited number of fatigue load applications. Walk-behind saws and spansaws do not induce high enough stresses to cause any appreciable fatigue damage at early ages.

In addition to sawing equipment, construction truck traffic may be driven across the concrete at early ages. Most common heavy construction loads are 80 kN (18 kip) single-axle load water trucks and 151 kN (34 kip) tandem-axle load dump trucks. Since the majority of early age construction traffic is typically dump trucks an early age analysis was conducted for a 151 kN (34 kip) TAL. Since front axle loads are farther from the slab edge they were not included in the analysis. At early ages, before grading operations are completed the slab edge drop-off minimizes pure edge loading conditions. Interior loads 610 mm (2 ft) from slab edges were assumed in the analysis. Since traffic may use shoulders as working platforms the benefits of tied shoulders were not used in the early loading analysis. Stresses for the 151 kN (34 kip) TAL were determined by proportioning stresses from eq. 5.

Modulus of rupture to limit 151.2 kN (34 kip) TAL stress ratios to 0.50 were calculated incorporating effects of outside subbase support and strength variability. Results are summarized in Table 3. For a 203 mm (8 in.) slab with subbase support of 54 MPa/m (200 lb/in.^{3}), the pavement can be opened to construction traffic when the flexural strength is a minimum of 2.4 MPa (355 psi).

Table 3 - Minimum Flexural Strength for Construction Traffic

t, mm.	k, MPa/m	L, (1) mm.	Flexural Stress, (2) MPa	Flexural Strength, (3) MPa
203	27.1	859	1.28	2.76
	40.7	776	1.19	2.55
	54.3	723	1.13	2.45
	81.4	653	1.05	2.28
	135.7	575	0.96	2.07
229	27.1	939	1.08	2.31
	40.7	848	1.00	2.17
	54.3	789	0.95	2.03
	81.4	713	0.89	1.90
	135.7	628	0.81	1.72
254	27.1	1016	0.92	1.97
	40.7	918	0.86	1.86
	54.3	854	0.82	1.76
	81.4	772	0.76	1.62
	135.7	679	0.69	1.48
279	27.1	1091	0.80	1.72
	40.7	986	0.74	1.59
	54.3	917	0.71	1.52
	81.4	829	0.66	1.41
	135.7	730	0.60	1.31
305	27.1	1165	0.70	1.52
	40.7	1052	0.65	1.41
	54.3	979	0.62	1.34
	81.4	885	0.58	1.24
	135.7	779	0.53	1.14

NOTES 1. Modulus of elasticity 20,700 MPa.

2. 151 kN TAL, 0.967 subbase support factor.

3. Strength coefficient of variation of 7 percent.

4. 10 in. = 354 mm 100 pci = 27.145 MPa/m 100 psi = 0.69 MPa

VERIFICATION OF STRESSES AT EARLY AGE

The analysis of early loading fatigue damage is dependent on material properties (modulus of rupture and elastic modulus), pavement design parameters, selected fatigue model, and theoretically calculated load induced edge stresses. Field load tests on mature concrete have established that measured stresses are statistically equivalent to calculated stresses.(3) Measured stresses at early ages from field testing were compared to theoretical stresses calculated using the ILLI-SLAB finite element computer program.

Field Studies

Pavement stresses at early ages were recently measured in Iowa and Utah.(1) Tests were conducted on a 254 mm (10 in.) thick slab over a granular base near Fort Dodge on U.S. Route 169 in Iowa. Strains due to a 90 kN (20 kip) single-axle load were measured at a longitudinal free edge midway between transverse joints, variable distances inward from free edges, and at dowelled transverse joints. Elastic moduli data was used to convert load induced strains to flexural stresses. Slabs were loaded at ages of 2, 3, 7, and 8 days. Tests were also conducted on a 254 mm (10 in.) thick slab over a 102 mm (4 in.) thick lean concrete base near Tremonton, Utah on Interstate 15. Strains due to a 90 kN (20 kip) single-axle load were measured at free edges midway between transverse joints, shoulder joints, variable distances inward from free edges or joints, and at aggregate interlock transverse joints. Slabs were loaded at 3, 4, 5, 6, 7, and 8 days. Tests were also conducted on slabs approximately one year old not yet opened to public traffic on the same project. Load tests were conducted at various time intervals throughout the day. Since maximum daily differences in measured strains were small relative to strain magnitudes and the measured strain did to consistently or significantly change with time, average strain throughout the day was used to calculate stresses. Other field studies have shown that curl influences on load induced strains were significantly less than on load induced deflections.(6)

Measured strains were compared to theoretical stresses for both the Utah and Iowa sites. Due to potential bonding between lean concrete subbase and concrete slab only longitudinal joint and free edge stresses were evaluated for the Utah data. For the 40 (IA) and 16 (UT) pairs of differences between theoretical and measured stresses, the null hypothesis that theoretical and measured stresses are statistically different could not be rejected at the 90 percent confidence level. Therefore, it was not statistically inferred that field measured stresses and calculated stresses came from different populations.

Full-Scale Tests

Full-scale tests were recently conducted on the CTL laboratory grounds to supplement the field testing verification done in Utah and Iowa.(2) Two separate 254 mm (10 in.) thick slab sections 18.3 m (60 ft) long by 7.3 m (24 ft) wide were constructed. Sections were constructed over a 127 mm (5 in.) thick granular base and a 127 mm (5 in.) thick lean concrete base. A 1.5 mm (6 mil) polyethylene layer was used to provide a bond breaker between the concrete and lean concrete base. Modulus of subgrade support was 27 MPa/m (100 lb/in.^{3}) and 81 MPa/m (300 lb/in.^{3}) for the subgrade and granular base, respectively. Transverse sawed joints were either dowelled or undowelled. Ends of the 18.3 m (60 ft) long sections were thickened to induce joint cracking. The sawed longitudinal joint was tied to simulate a 3.7 (12-ft) wide shoulder. Strains due to a 90 kN (20 kip) single-axle load were acquired with surface mounted 120 mm-long strain gages. Both static and creep load induced strain at free edges midway between joints, longitudinal shoulder joints, and at variable distances inward from longitudinal joints and edges were measured.

Stresses were computed using slab elastic moduli determined from the pulse velocity nondestructive testing method. Correlations of field cured cylinder test data with elastic modulus were used to convert strains to stresses. Strains were measured at 5, 6, 9, 14, and 26 days for the slabs constructed over the granular subbase. Slab moduli of rupture determined from insitu pulse velocity testing were 2.7, 2.7, 2.8, 3.0, and 3.4 MPa (385, 390, 400, 440, and 490 psi) for the five test times. For slabs constructed over the lean concrete base, tests were conducted at 6, 9, 12, and 19 days. Slab moduli of rupture corresponding to loading was 2.1, 2.33, 2.4, and 2.9 MPa (300, 340, 350, and 425 psi) for the slab over lean concrete base. Creep load strains were consistently and significantly less than static load induced strains. Average static to creep strain ratios were 1.2 for both slabs. All creep load stresses were therefore multiplied by 1.2 to get equivalent static stresses. Average stress transfer at the longitudinal shoulder joint was 73 and 71 percent for slabs constructed over the granular subbase and lean concrete base, respectively. Load stress transfer efficiencies were used to compute theoretical free edge stresses as shown in eq. 7. Stresses were theoretically calculated using the ILLI-SLAB finite element computer program. For each test time the nondestructively determined elastic modulus of concrete was input to compute theoretical stresses.

A paired t-test analysis was conducted on the difference between theoretical and measured stresses. For the 177 strain readings on the slab constructed over the granular base the null hypothesis that stresses are statistically different could not be rejected at the 90 percent confidence level. Therefore, it can not be inferred that field measured stresses and calculated stresses come for different populations. Results of measured versus calculated stresses for slabs constructed over the granular subbase are shown in Figure 2.

Tests on the lean concrete subbase did not compare quite as favorably. Theoretical stresses were generally higher than measured stresses. For the 229 tests average measured stress was 55 kPa (8 psi) lower than theoretical stresses. Differences between measured and calculated stresses were statistically different than zero. The null hypothesis that measured stresses were 34 kPa (5 psi) lower than theoretical stresses could not be rejected at the 90 percent confidence level. Therefore, it can not be inferred that measured stresses are more than 34.5 kPa (5 psi) lower than calculated stresses.

The analysis uses theoretical stresses to evaluate fatigue damage at early ages. Field tests in Iowa and Utah indicated that it can not be inferred that measured and calculated stresses come from different populations. Analysis of full-scale data over a granular base also indicated it can not be statistically inferred that measured and calculated stresses come from different populations. Since full-scale tests of concrete over a lean concrete base showed that measured stresses were slightly lower (55 kPa average) than calculated stresses a small degree of conservatism in the early loading analysis for slabs over a lean concrete base is expected.

EVALUATION OF DOWEL BEARING STRESSES

The maximum load associated bearing stresses exerted by dowel bars at transverse contraction joints can be critical in evaluating required opening concrete strengths of dowelled pavements and have a large impact on the development of transverse joint

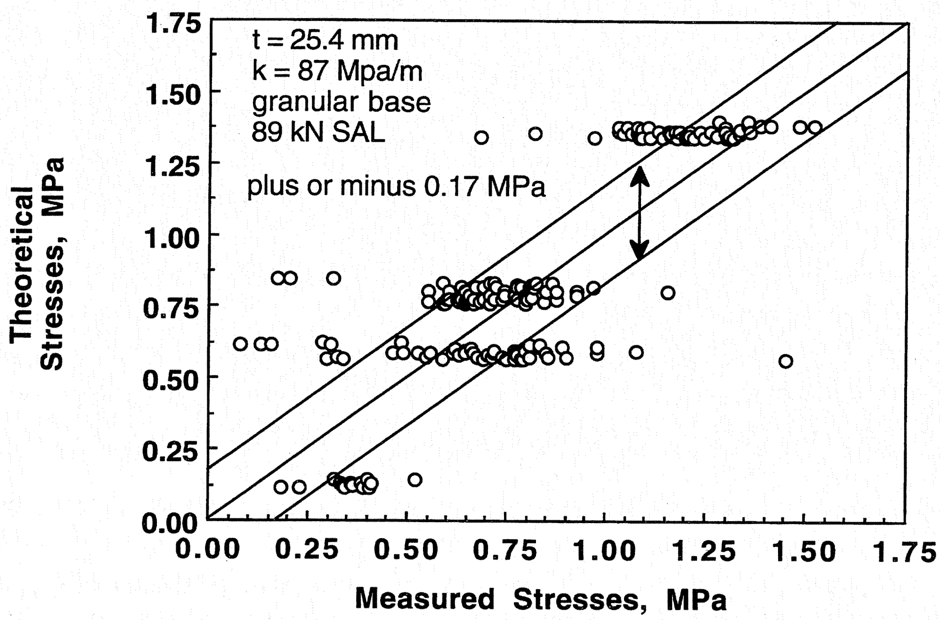

Fig. 2 - Measured vs Theoretical Stresses

faulting.(13) If dowel bearing stresses due to loading exceed early age concrete compressive strength, crushing of the concrete below the dowels may occur. Excessive joint deflections due to concrete crushing below dowels will reduce load transfer. This may lead to pumping, premature joint faulting, and corner cracking.

Friberg Analysis

The Friberg analysis was used to calculate maximum free corner dowel bearing stresses. Maximum bearing stress is calculated as: (14)

$$\sigma_{max} = G * \delta_o \quad \ldots\ldots\ldots\ldots (10)$$

where

σ_{max} = dowel bearing pressure
G = modulus of dowel support
δ_o = deflection of dowel at the face of the joint
$= P_t \ (2 + \beta z) / 4 \beta^3 E_s I$
P_t = shear force acting on dowel
= wheel load * percent transferred load across joint * dowel distribution factor
β = relative stiffness of the dowel concrete system
$= [G d / (4 E_s I)]^{0.25}$
z = width of joint opening
E_s = modulus of elasticity of dowel bar
I = moment of inertia of dowel bar cross-section
$= 0.25 \pi (d/2)^4$ for round solid dowels
d = dowel diameter

Bearing stress and dowel deflection are a function of the modulus of dowel support. Very little literature and few guidelines are available in selecting G-values, especially at early ages. The shear force, P_t, on the dowel is a function of load magnitude, load carried across the joint by dowel bars (transferred load), and the shear load distribution of the transferred shear load.

Modulus of Dowel Support

The modulus of dowel support, G, is difficult to measure or quantify. Values of 81,400 and 407,200 MPa/m (300,000 and 1,500,000 lb/in.^{3}) have been suggested for mature concrete.(15) Load deflection test and embedded dowel tests suggest values of G ranging from 190,000 to 2,334,000 MPa/m (700,000 to 8.6 million lb/in.^{3}) (15,16) Commonly a value of 407,000 MPa/m (1,500,000 lb/in.^{3}) is used in design. Other dowel tests at 14 and 37 days indicate G ranges from 217,000 to 1,276,000 MPa/m (0.8 to 4.7 million lb/in.^{3}) (17) Dowel support was back-calculated from dowel deflections at the joint face for dowel diameters of 19, 25, and 32 mm (3/4, 1, and 1-1/4 in.). Dowels were embedded 229 mm (9 in.) into a 254 mm (10 in.) thick concrete block. As dowel diameter increased, G, decreased. Also values were higher for higher compressive strength at a given time and also increased with age.

A three dimensional finite element analysis was conducted to evaluate effects of concrete elastic modulus on modulus of dowel support. Dowel diameter was set at 1/8 the slab thickness. Modulus of dowel support was back-calculated using eq. 10 from stress data under the dowel at the joint face. Dowel moduli of support are summarized in Fig. 3. Modulus of dowel support ranged from 95,000 through 611,000 MPa/m (350,000 through 2,250,000 lb/in.^{3}) for concrete elastic modulus ranging from 6895 through 34,470 MPa (1 through 5 million psi). Modulus of dowel support increased with increasing elastic modulus and decreasing dowel diameter (slab thickness). Values for concrete modulus of elasticity of 27,580 MPa (4 million psi) ranged from 285,000 MPa/m (1,050,000 lb/in.^{3}) with the 32 mm (1-1/4 in.) diameter bar to 556,000 MPa/m (2,050,000 lb/in^3.) with the 19 mm (3/4 in.) diameter bar, within the range commonly used in concrete pavement

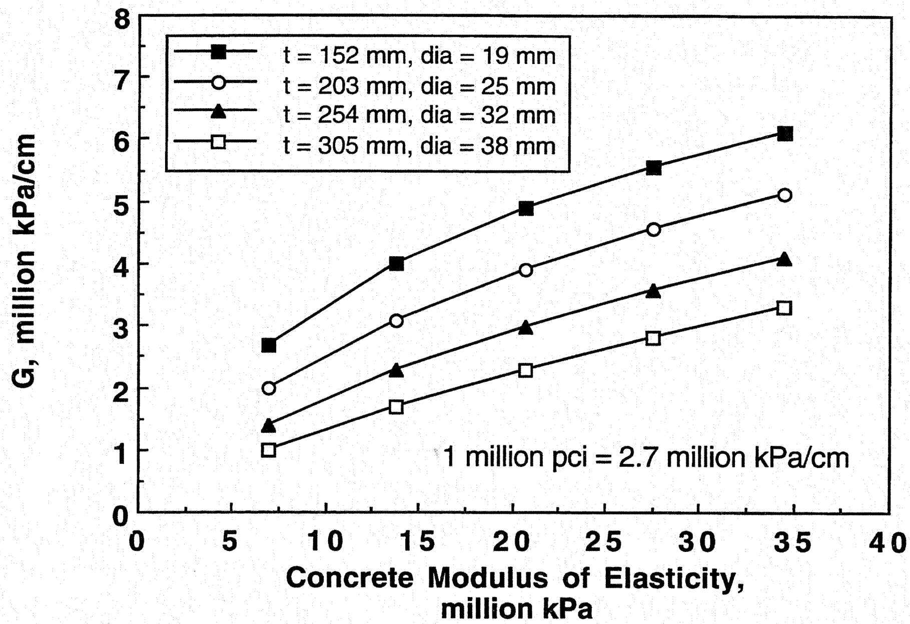

Fig. 3 - Modulus of Dowel Support

design.

Transferred Load Across Doweled Joint

The shear force acting on the critical corner dowel is a function of the load transferred across the joint in shear as well as the dowel bar shear distribution. The finite element program JSLAB was used to determine the percentage of a 44.5 kN (10 kip) single-axle load transferred across the joint in shear.(17) The program calculates shear loads at each dowel bar along the joint. Slab thicknesses of 203 and 254 mm (8 and 10 in.) were analyzed for a load positioned over the critical corner dowel bar. Modulus of elasticity was 13.8, 20.7, and 27.6 GPa (2, 3 and 4 million psi). Modulus of subgrade reaction was 27 and 81 MPa/m (100 and 300 lb/in.^{3}). Dowel diameter was 25 and 32 mm (1 and 1-1/4 in.) for the 203 and 254 mm (8 and 10 in.) - thick slabs, respectively. Modulus of dowel support varied with concrete elastic modulus as shown in Fig. 3. No aggregate interlock load transfer was assumed at the 2.5 mm (0.1 in.) wide doweled joint. Dowel shears were calculated for a 3.7 m (12 ft) wide slab with and without a 3.0 m (10 ft) wide tied concrete shoulder (no dowels at shoulder transverse joint).

Without a tied shoulder the load transferred across the joint in shear ranged from 30 to 50 and averaged 42 percent of the 44.5 kN (10 kip) wheel load. The 42 percent is the same as reported for an analysis of mature concrete. (18) With a tied concrete shoulder the percentage ranged from 22 to 35 and averaged 29 percent. A 45 and 35 percent transferred load factor was assumed in the early loading analysis for slabs with and without tied shoulders, respectively.

Shear Load Distribution

The distribution of shear loads from JSLAB was compared to the distribution used in the Friberg analysis. The Friberg analysis assumes that all dowels within a distance of 1.8 times the radius of relative stiffness, L, from the application of load are effective in carrying shear. For mature concrete it has been determined that only dowels within a distance of 1.0*L are effective in transferring load.(18) The JSLAB analysis of concrete at early ages also confirmed that only dowels within one radius of relative stiffness are effective in transferring shear. The distance where dowels are effective in transferring shear averaged 0.9 and 1.1 times the radius of relative stiffness for slabs with and without tied shoulders, respectively. The Friberg analysis used in the early loading analysis was modified to account for the reduced effective distance of 1.0*L.

The Friberg analysis assumes that the shear load is linearly proportioned as a function of distance away from the load. This assumption, modified for the effective distance of 1.0*L, was compared to the dowel distribution computed from the JSLAB analysis. The JSLAB distribution factor for each dowel was computed by dividing the shear at each dowel by the total shear across the joint. In most of the designs analyzed, the distribution factor was significantly different between those computed with JSLAB and those computed using the modified Friberg analysis. The ratio of JSLAB to Friberg load distribution factors ranged from 0.83 to 1.39. Excluding two conservative outliers (thickness of 254 mm (10 in.), subbase support of 27 MPa/m (100 lb/in.^{3}), and elastic modulus of 13,790 MPa (2 million psi) with and without shoulders) the ratio of JSLAB to Friberg critical corner dowel distribution factors can be determined by:

$$\text{Ratio} = 26.30 * \text{sqrt}(L) - 32.4 \qquad R^2 = 0.868 \quad (11)$$

where

Ratio = JSLAB to Friberg critical corner dowel distribution factor ratio, percent
L = radius of relative stiffness, in. (eq. 3)

For radius of relative stiffness greater than 25.3 the Friberg analysis underestimates the dowel distribution factor (ratio greater than 100 percent). To determine the JSLAB distribution factor on the corner dowel the modified Friberg distribution factor is multiplied by the ratio (divided by 100) determined from eq. 11 as shown in Fig. 4.

EARLY LOADING BEARING STRESS GUIDELINES

The Friberg analysis was used to determine dowel bearing stresses for different moduli of elasticity. Dowel pressures were computed using eq. 10. Modulus of dowel support was determined as shown in Fig. 3. The transferred load was assumed 45 and 35 percent of a SAL positioned at the corner for a slab with and without shoulders, respectively. Effective length was 1.0 * radius of relative stiffness. The Friberg dowel distribution factor for the critical corner dowel was modified using eq. 11. Dowel bearing pressures were computed for 152, 203, 254, and 305 mm (6, 8, 10, and 12 in.) thick slabs with elastic moduli of 6895 through 34,470 MPa (1 through 5 million psi) at 1720 MPa (0.25 million psi) increments. Modulus of subbase reaction, k, was varied between 27 and 136 MPa/m (100 and 500 $lb/in.^3$). Compressive strength was back-calculated at each elastic modulus using a relationship determined from laboratory test data of early age concrete properties (1 through 28 days). (1) Compressive strength was determined from the following relationship:

$$E_c = 62{,}000 * \mathrm{sqrt}\ (f'_c) \quad \ldots\ldots\ldots\ldots\ldots\ldots \ (12)$$

where

E_c = concrete modulus of elasticity, psi
f'_c = concrete compressive strength, psi

Minimum opening compressive strength was determined when concrete strength is greater than dowel bearing pressures. For a given elastic modulus and thickness, the resulting minimum compressive strengths did not significantly increase with modulus of subbase support (decreases in L). As shown in eq. 11, the dowel distribution factor increases with increases in radius of relative stiffness, L. As the distribution factor increases the shear force acting on the dowel, P_t, also increases offsetting the benefits of having more dowels transferring load in shear (effective length of 1.0*L). Minimum opening compressive strengths shown in Table 4 are therefore independent of subbase support. Average coefficient of variation for compressive strength in the study of early age concrete properties was 4.2 percent. (1) To account for material variability Table 4 reflects a 5 percent coefficient of variation reduction in compressive strength.

SUMMARY

Guidelines for determining minimum opening flexural strengths based on a 90 kN (20 kip) single-axle edge load were established. The procedure is easily modified to account for other load magnitudes and tandem-axle-loads. Minimum flexural strengths were also established for 151 kN (34 kip) tandem-axle loads positioned 610 mm (2 ft) from the slab edge. Early loading of doweled transverse pavement joints is also a function of concrete compressive strength. Minimum compressive strength was selected when strength exceeded dowel bearing pressures.

For a 254 mm (10 in.) thick slab with a modulus of subgrade support of 81 MPa/m (300 lb/in^3) the minimum flexural strength, as listed in Table 2, is 3.4 and 2.0 MPa (490 and 295 psi) for pavements without and with a tied concrete shoulder, respectively. As listed in Table 3 the opening strength for construction traffic is reduced to 1.6 MPa (235 psi) for construction traffic. In addition to the minimum flexural strength criteria, if the pavement has 32 mm (1-1/4 in.) diameter dowel bars at transverse joints, the minimum opening compressive strength for a 90 kN (20 kip) SAL is 19.3 and 14.8

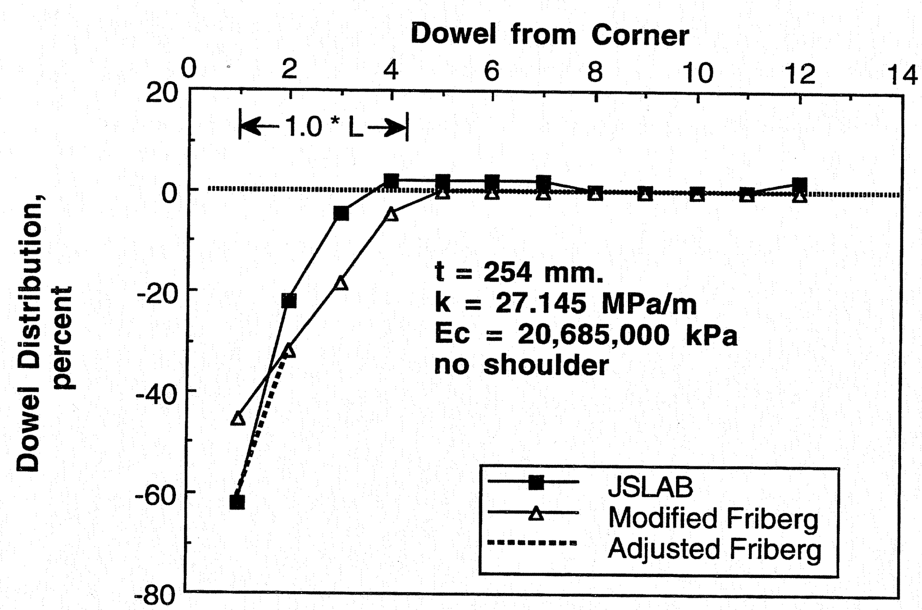

Fig. 4 - Dowel Distribution Factor

Table 4 - Minimum Compressive Strength for Dowel Bearing Pressures

		71 kN SAL		80 kN SAL		89 kN SAL		98 kN SAL	
t, mm	Dowel Dia., mm	No Shoulder f'c, MPa	Shoulders f'c, MPa	No Shoulder f'c, MPa	Shoulders f'c, MPa	No Shoulder f'c, MPa	Shoulders f'c, MPa	No Shoulder f'c, MPa	Shoulders f'c, MPa
152	19	45.4	34.9	> 44.8	40.0	> 44.8	> 44.8	> 44.8	> 44.8
203	25	25.9	19.3	26.1	21.9	31.7	24.5	34.9	27.3
254	32	14.8	11.8	17.0	12.8	19.3	14.8	21.9	17.0
305	38	10.0	7.6	11.8	8.3	12.8	10.0	13.8	10.9

NOTE: 20 kips = 89 kN
1000 psi = 6.895 MPa

MPa (2800 and 2150 psi) for pavements without and with a tied concrete shoulder, respectively.

ACKNOWLEDGMENTS

Work was conducted by the Construction Technology Laboratories, Inc. (CTL) under the sponsorship of the Portland Cement Association. The opinions and findings expressed or implied in this paper are those of the authors.

REFERENCES

1. Okamoto, P. A., P.J. Nussbaum, K.D. Smith, M.I. Darter, T.P. Wilson, C.L.Wu, and S.D. Tayabji, "Guidelines for Timing Contraction Joint Sawing and Earliest Loading for Concrete Pavements," FHWA-RD-91-079, 080, Federal Highway Administration, Washington DC, 1991.

2. P.A. Okamoto, C.L. Wu, and S.M. Tarr, "Early Opening of PCC Pavements to Traffic," report by CTL, Inc. to the Portland Cement Association, Skokie, IL, August 1993.

3. A.M. Tabatabaie, E.J. Barenberg, and R.E. Smith, "Longitudinal Joint Systems in Slip-Formed Rigid Pavements," volume II, Analysis of Load Transfer Systems for Concrete Pavements, report no. FAA-RD-79-4, II, Federal Aviation Administration, Washington DC, November 1979.

4. Thickness Design for Concrete Highway and Street Pavements, EB109.01P, Portland Cement Association, Skokie, IL, 1984.

5. B.E. Colley, C.G. Ball, and P. Arriyavat, "Evaluation of Concrete Pavements with Tied Shoulders or Widened Lanes," Transportation Research Record 666, Transportation Research Board, Washington DC, 1978.

6. S.D. Tayabji, C.G. Ball, and P.A. Okamoto, "Effect of Concrete Shoulders on Concrete Pavement Performance," report by CTL, Inc. to the Minnesota Department of Transportation, St. Paul, MN, July 1983.

7. J.S. Sawan, "Effect of PCC Shoulders on Adjacent Concrete Lanes," Journal of Transportation Engineering, vol. 109, no. 2, American Society of Civil Engineers, New York, NY, March 1983.

8. C.G. Ball and L.D. Childs, "Tests of Joints for Concrete Pavements," RD026.01P, Portland Cement Association, Skokie, IL, 1975.

9. A.M. Ioannides and G.T. Korovesis, "Aggregate Interlock: A Pure-Shear Load Transfer Mechanism," Transportation Research Record 1286, Transportation Research Board, Washington DC, 1990.

10. "Computer Program for the analysis and Design of Foundation Mats and Combined Footings," SR100D, Portland Cement Association, Skokie, IL, 1971.

11. R.G. Packard and S.D. Tayabji, "New PCA Thickness Design Procedure for Concrete Highway and Street Pavements," Proceedings Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West Lafayette, IN, April 1985.

12. "Recommended Practice for Evaluation of Strength Test Results of Concrete," ACI 214-77, American Concrete Institute, Detroit, MI, 1992.

13. K.D. Smith, D.G. Peshkin, M.I. Darter, A.L. Mueller, and S.H. Carpenter, "Performance of Jointed Concrete Pavements," vol. I, FHWA-RD-89-136, Federal Highway Administration, Washington DC, 1989.

14. B.F. Friberg, "Design of Dowels in Transverse Joints of Concrete Pavements," Transactions of the American Society of Civil Engineers, vol. 105, New York, NY, 1940.

15. L.W. Teller and H.D. Cashell, "Performance of Doweled Joints Under Repetitive Loading," Public Roads, vol. 30, no. 1, April 1958.

16. E.A. Finney, "Structural Design Considerations for Pavement Joints,," Proceedings, vol. 53, American Concrete Institute, Detroit, MI, July 1956.

17. S.D. Tayabji and B.E. Colley, "Analysis of Jointed Concrete Pavements," FHWA-RD-86-041, Federal Highway Administration, Washington DC, 1984.

18. K.W. Heinrichs, M.J. Liu, M.I. Darter, S.H. Carpenter, and A.M. Ioannides, "Rigid Pavement Analysis and Design," FHWA-RD-88-068, Federal Highway Administration, Washington DC, 1989.