

Effect Of Built-In Curling On Concrete Pavement Performance And Design In Chile

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Abstract

Built-in curling (BIC) is a permanent upward curvature of the concrete slab produced by irreversible moisture and temperature differences between the top and the bottom of the slab. This residual curvature has been shown to change the performance of concrete pavements. With mechanical loading, built-in curling typically changes the location and magnitude of the maximum tensile stress in the slab as larger regions of the slab are without contact with the support layer. Two existing concrete pavement projects in Chile were selected to characterize their in-situ BIC, document their performance, and determine if existing design procedures can adequately predict and account for their performance differences. In both case studies, BIC had a large effect on pavement performance and the magnitude of the effective built-in temperature difference, which is an important design input to predict the performance life. For one of these projects, premature slab failures resulted due to the initial BIC. The concrete thickness would needed to be increased by 59% or the slab size reduced to 2.00 m x 1.75 m to avoid premature failure of the section due to the high level of BIC. In the second project, the section of the concrete pavement that performed best was constructed at night and had a lower BIC relative to the sections constructed during the daytime in the summer.

KEYWORDS: concrete pavements, built-in curling, pavement performance, curling analysis, falling weight deflectometer analysis, FWD, mechanistic-empirical pavement design guide, M-E PDG

Introduction

Concrete pavements have been built in Chile for over 60 years. Nearly all of the highway concrete pavements existing today were built between 1980 and 1995. The structural design of these jointed pavements followed the AASHTO method. The slabs were normally constructed with thicknesses of 180 to 250 mm over granular bases and without dowels. In some concrete pavement designs, cement-treated base layers were used, which failed prematurely and overall performed poorly. Concrete pavements built over granular base layers had varying performance results. Some sections failed prematurely, but others are still in service with almost double the original design traffic. Construction practices and materials were not very different from one project to the other with the aggregate, cement, water, admixtures, and mixture proportions mostly the same. The difference in pavement performance is not easily explained with existing, empirical design procedures. Measurements made in Chile (Larrain 1985; Poblete 1988) and in other countries, as seen in Table 1, have revealed that many concrete slabs have an initial upward curling condition. When combined with truck loading, this condition normally results in the critical tensile stresses on the top of the slab and thus should be considered in the concrete pavement structural design (Hiller 2007).

Curling in concrete pavements is a combination of five equivalent temperature components as seen in Equation (1). These factors are significantly affected by the local climate conditions, concrete mixture parameters, concrete hardened properties, and construction techniques.

$$T_{\text{tot}} = T_{\text{tg}} + T_{\text{mg}} + T_{\text{bi}} + T_{\text{shr}} - T_{\text{crp}} \quad (1)$$

Where:

- T_{tot} = Total effective linear temperature difference (TELTD);
- T_{tg} = Equivalent temperature difference between top and bottom of a slab due to the nonlinear temperature profile in the slab;
- T_{mg} = Equivalent temperature difference between top and bottom of a slab due to the nonlinear moisture gradients in the slab;
- T_{bi} = Equivalent temperature difference between top and bottom of a slab due to the nonlinear construction temperature profile “built into” the slab;
- T_{shr} = Equivalent temperature difference between top and bottom of a slab due to the irreversible drying shrinkage between top of the slab and bottom of the slab;
- T_{crp} = Part of T_{bi} and T_{shr} recovered through tensile creep of the slab.

The cumulative effect of built-in temperature difference, shrinkage, and creep can be defined as an effective built-in temperature difference (EBITD), as seen in Equations (2) and (3). The EBITD is defined by a temperature difference between the top and bottom of a concrete slab that produces the same deflection response as the cumulative effects of a nonlinear built-in temperature and a shrinkage profile relaxed over time by creep.

$$T_{\text{tot}} = T_{\text{tg}} + \text{EBITD} \quad (2)$$

$$\text{EBITD} = T_{\text{mg}} + T_{\text{bi}} + T_{\text{shr}} - T_{\text{crp}} \quad (3)$$

The total amount of curling in a slab (T_{tot}) due to a combination of the above factors can be represented by an effective linear temperature difference, i.e., total effective linear temperature difference (TELTD). As shown in Equation (4), the EBITD can be calculated from field measurements by measuring the temperature profile (T_{tg}) through the slab during deflection testing and then determining the TELTD from a back-calculation analysis.

$$\text{EBITD} = \text{TELTD} - T_{\text{tg}} \quad (4)$$

Table 1. Summary of Built-in Curling for Various Concrete Pavement Field Studies (after Bordelon et al. 2009)

Location / ID	Built-in Curl (°C)	Comment	Reference
Surface Profiling (using dipstick unless otherwise noted)			
All LTPP GPS3 Sections ^a	Up to -44	Multiple geometries / restraint	Byrum 2000
I-80 in Pennsylvania	-37	Dowels / 20 ft joint spacing	Yu et. al. 2001
AZ1 in Phoenix, Arizona	-15.2 ^b	No dowels / AC shoulder / 4.5m joint spacing	C. Rao et. al. 2001
	-23.9 ^c		
AZ3 in Phoenix, Arizona	-23.1 ^b	No dowels / AC shoulder / 6m joint spacing	
	-29.1 ^c		
AZ4 in Phoenix, Arizona	-11.8 ^b	Dowels / Tied PCC shoulder / 6m joint spacing	
	-20.0 ^c		
Mankato, Minnesota	-22.8 ^b	No dowels / AC shoulder	
	-27.4 ^c		

<i>Deflection Measurements (using FWD/HWD unless otherwise noted)</i>			
I-70 in Colorado ^d	-11,1	Dowels / Tied PCC shoulder	Yu et. al. 1998
Florida ^d	-5	Undoweled	Armaghani et. al. 1987
Chile ^d	Up to -19.2	Multiple sections	Poblete et. al. 1988
I-80 in Pennsylvania ^d	-8,9	Perm. Aggregate base	Yu et. al. 2001
	-6,7	Perm. Asphalt base	
Palmdale, California	-22,7	No dowels / AC shoulder	S. Rao and Roesler 2005b
	-9,8	Dowels / Tied PCC shoulder	
	-17,2	Widened lane / AC shoulder	
Ukiah, California	-10	No dowels / AC shoulder	S. Rao and Roesler 2005b
DIA Denver, Colorado ^e	-5 to -8	430mm airfield slabs	Rufino 2003

^a Using high speed profiling

^b 3 days after construction

^c 40 days after construction

^d Using fixed external surface gages for deflections

^e Using multi-depth deflectometers

Chilean Case Studies: Routes 60 and 78

Two projects were studied in Chile to quantify the magnitude of built-in curling and its effect on the concrete pavement performance. The first project was Route 78 (Figure 1) between the cities of Melipilla and Leyda from Km. 67.350 to 92.930 (elevation 100 to 260 m) in the valley between the Andes Mountains and the coastal ranges. This project had a slab thickness with a trapezoidal cross-section of 260 mm on the outer lane edge and 220 mm on the inside lane edge, a 150 mm granular base, asphalt shoulders, and no dowels. It was constructed from 1992 to 1995 in three different stages. One section was paved during nighttime and the other two during the day, but both in the summer time. The general climatic conditions for this project are a dry, no freeze environment with a maximum temperature in the summer months of 35°C and low temperature approaching 0°C. The difference in the climatic conditions during the paving had an important effect on performance. The section constructed at nighttime is performing very well without cracks and the international roughness index (IRI) and faulting are below the maximum accepted parameters. The two other sections are starting to deteriorate with around 5% transverse cracking and increased faulting and IRI values.



Figure 1. Route 78 Section D (Km 90.000 to Km 92.930) paved during daytime.

The second pavement project was Route 60 from Km. 80.480 to 84.238 (approximate elevation of 2000 m) which is a two lane road connecting Los Andes, Chile, and Mendoza, Argentina, as seen in Figure 2. The roadway was designed for 20 million ESALs using AASHTO 1998, which resulted in a 220 mm concrete slab with a flexural strength of 5 MPa. The pavement support layers consisted of a 450 mm non-frost susceptible granular base (less than 5% passing #200 sieve) with a California bearing ratio (CBR) of 80% over a soil k-value of 137 MPa/m. The typical slab size was 4 m long by 3.5 m width without any joint load transfer devices or stabilized shoulders. A resin-based curing compound was used after casting of the concrete and the pavement was opened to traffic 21 days later. After 24 hours of traffic, 80% of the slabs had transverse cracks and within 10 days, 100% of slabs had fractured. Longitudinal cracks began appearing one month later on the slabs. This project was in a dry freeze zone at a much higher elevation (2000 m above sea level) and therefore the climatic conditions are extreme from day to night and between seasons. The day to night temperature changes can be 25°C with a maximum air temperature in the summer months of 28°C and low air temperature around -10°C in the winter. For this project, the temperature of the concrete at casting was over 35°C.



Figure 2. Route 60 between Los Andes (Chile) and Mendoza (Argentina)

Within different sections on Route 78 and comparing Routes 60 and 78, the concrete pavement performed quite differently, which could not be explained by the empirical AASHTO 1998 design method. The Mechanical-Empirical Pavement Design Guide (M-E PDG) was selected to determine if the performance differences observed could be predicted based on the local climatic conditions as well as the magnitude of the built-in curling. Therefore, the EBITD was measured in the field for both of these projects and then the M-E PDG was utilized to analyze the performance given the accumulated traffic, EBITD, layer and material properties, climate, and slab geometry.

Procedures for Estimating EBITD of Concrete Pavements

To calculate the EBITD, two different measurement methods were employed for each project: one based on falling weight deflectometer (FWD) tests and the other on the slab's surface profile. While researchers have reported built-in curling based on surface curvatures (e.g., Byrum 2000 and Vandenbosshe 2003), other research has noted that surface curvature measurements may overestimate the actual built-in curling (Yu et al. 2001).

In order to back-calculate EBITD with either method, a finite element analysis is required and thus accurate input information is essential. Slab dimensions can be measured by field observations. The pavement thicknesses, layer properties (elastic modulus, unit weight, and Poisson's ratio), and interface conditions can be obtained from construction records and coring.

Route 60: Surface Elevation Differences For this project, only topographic equipment was available to estimate the surface profile of the slab. From this information, the heights between the corner and the center of the slab at two different times of the day could be calculated. With the slab geometric and material properties, ISLAB2000 was used to backcalculate the equivalent temperature difference that would produce the same magnitude of measured curling and corner-interior elevation difference. Thermocouples were also installed in the pavement to measure the actual temperature difference between the top and bottom of the slab. The built-in curling (EBITD) was calculated by subtracting the measured temperature difference from the backcalculated equivalent temperature difference, as shown in Equation (3).

Route 78: FWD Corner Deflections The EBITD was also estimated based on FWD corner deflection testing to determine the response of the concrete slab to the level of residual and reversible curling (Rao and Roesler 2005). Generally this method is preferred, A brief summary of the procedure used to estimate the EBITD is described next.

Step 1: Field Data Collection and Backcalculation The load plate of the FWD was dropped at the slab corner and the deflections under the load plate and across the transverse joint were measured. The deflections on either side of the transverse joint estimate the load transfer efficiencies across the joints. For each slab, the deflection data were collected at both outside corners of the slab for three load levels. An estimate of the modulus of subgrade reaction (k-value) is needed to accurately assess the contribution of the subgrade stiffness to the corner deflection response. FWD drops were made at the center of the slab and the acquired deflection data was used to backcalculate the dynamic k-value of the soil and elastic modulus of the concrete. An estimate of the actual temperature difference in the slab at the time of FWD testing

was also required in order to determine the EBITD. This difference was calculated from the surface temperature sensor on the FWD and a measurement at the bottom of the slab taken from an oil-filled hole.

Step 2: Interpolate Corner Deflections Corner deflections were measured at three FWD load levels. The actual load experienced by the pavement is measured by the FWD load sensor and can differ from the set load levels and also from one drop to another. In order to minimize the number of finite element computations, the deflection data are normalized to obtain deflections at three pre-determined load levels.

Step 3: Finite-Element Analysis The two-dimensional finite-element program ISLAB2000 analyzes the concrete slab responses under combined temperature and mechanical loading. The finite-element analysis was performed given the following inputs for the pavement section:

- Joint spacing and slab width;
- Load (three FWD load levels and no load);
- Subgrade k-value (dynamic);
- Load transfer efficiency;
- Layer properties such as thickness, elastic modulus, coefficient of thermal expansion, Poisson's ratio, unit weight, and slab-base interface bond condition;
- Load geometry and position (a square load with the same area as that of the FWD circular load plate is used).

The analysis was also conducted simultaneously for slab temperature differences ranging from -50°C to 0°C in increments of 3°C . The predicted corner deflection due to the FWD load is the difference between the loaded slab deflection (load plus temperature) and the unloaded slab deflection (temperature only). The result of the analysis is predicted deflections versus total effective linear temperature difference as seen in Figure 3 (solid lines).

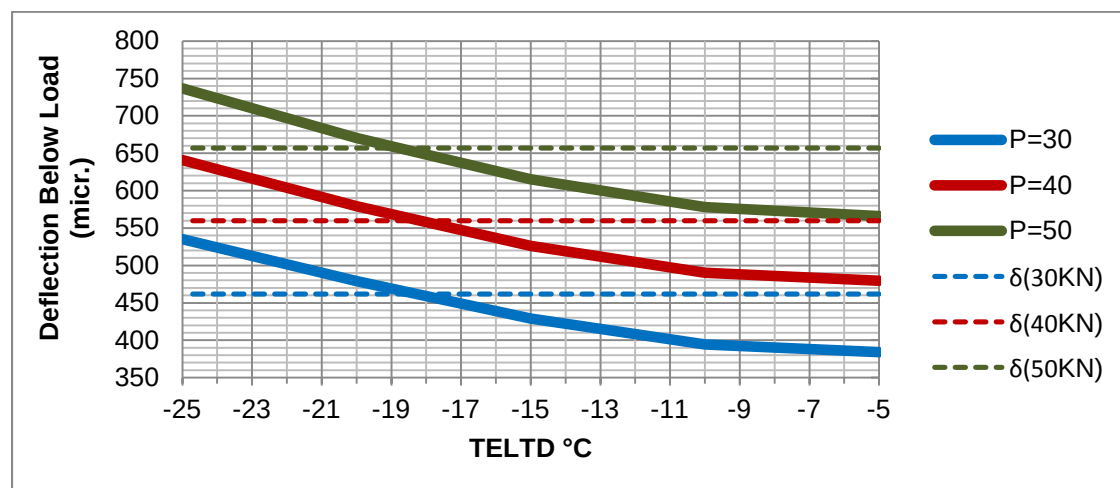


Figure 3. Comparison of measured deflections by FWD (dotted lines) and calculated deflections in ISLAB2000 (solid line) for different loads (kN) and total effective linear temperature difference (TELTD).

Step 4: Estimate TELTD and EBITD With the field-measured deflections, the dotted lines in Figure 3, can be entered and determined where the predicted and measured deflections intersect. The intersection point for each load level is the

estimated TELTD for each load drop. The EBITD is then the difference between the TELTD and the slab's actual temperature difference at the time of FWD testing.

Route 60 EBITD Values The measured elevation differences between the center and the corners of the several slabs at different times of the day are summarized in Table 2 Corner to Center of Slab Elevation Differences for 5 Slabs in the Morning and Afternoon. The level of negative curling on these slabs was extreme and significant slab cracking in this section occurred in less than 24 hours after opening to truck traffic.

Table 2. Corner to Center of Slab Elevation Differences for 5 Slabs in the Morning and Afternoon

Time of Day	Slab Number				
	1	2	3	4	5
8:00	11 mm	11 mm	11 mm	14 mm	8 mm
15:00	8 mm	10 mm	9 mm	14 mm	7 mm

Table 3 shows the backcalculated slab temperature differences, using ISLAB2000, to produce the same corner to center slab elevation difference. Since the temperature profile measurement equipment malfunctioned during the slab curling tests, the temperature difference from the top and bottom of the slab were calculated from the Enhanced Integrated Climatic Model (Larson and Dempsey 1997), which is a module within the M-E PDG. The Palmdale, California, climate station was selected since it had a similar weather, latitude, and altitude as the Route 60 project. Table 3 lists the predicted slab temperature differences [ΔT ($^{\circ}\text{C}$)] during the period of testing. The average EBITD calculated from Route 60 tests was -41°C . This magnitude of EBITD was consistent with several other surface curvature studies shown in Table 1. The high level of residual, negative curling will lead to alternative cracking modes (Hiller 2007) and explains the premature cracking of the section.

Table 3. TELTD and EBITD Values of Five Slabs in Route 60

Time of measure	Slab Number					TELTD $^{\circ}\text{C}$ (Average)	Slab	
	1 ($^{\circ}\text{C}$)	2 ($^{\circ}\text{C}$)	3 ($^{\circ}\text{C}$)	4 ($^{\circ}\text{C}$)	5 ($^{\circ}\text{C}$)		ΔT ($^{\circ}\text{C}$)	EBITD ($^{\circ}\text{C}$)
8:00	-35	-35	-35	-42	-26	-35	+5	-40
15:00	-26	-31	-28	-42	-22	-30	+12	-42
							Average	-41

Route 78 EBITD Values This evaluation divided the 25 km pavement project into four different sections: Section A (Km 67.350 to Km 81.250), Section B (Km 81.250 to 83.670), Section C (Km 83.670 to Km 90.000), and Section D (Km 90.000 to Km 92.930). Section A was paved during nighttime conditions while the other three sections (B, C, and D) were cast during the day. The adjustment made to the EBITD backcalculation method compared to the procedure outlined by Rao and Roesler (2005) was to include in the finite element analysis the axle load of the FWD and the truck that towed the FWD equipment. As shown in Tables 4 and 5, Section A, paved at night, had a less negative EBITD relative to the other three sections paved during

the day. Route 78 project also had a smaller average EBITD than the project on Route 60 but was located in a less severe climate condition and had a different EBITD measurement technique. All Route 78 sections had significantly larger EBITD ($\Delta T = -16.2^{\circ}\text{C}$) than the default value (-5.6°C) assumed in the M-E PDG program.

Table 4. Measured Temperatures with Predicted TELTD, and Calculated EBITD for Section A

Location (Km)	Temperature, Top ($^{\circ}\text{C}$)	Temperature, Bottom ($^{\circ}\text{C}$)	TELTD ($^{\circ}\text{C}$)	EBITD($^{\circ}\text{C}$)
75.000	13.8	18.2 $^{\circ}\text{C}$	-12.8	-8.4
75.003	13.8	18.2 $^{\circ}\text{C}$	-14	-9.6
75.500	12.8	18.1 $^{\circ}\text{C}$	-18.5	-13.2
75.503	12.8	18.1 $^{\circ}\text{C}$	-18.3	-13.0
Average				-11

Table 5. Measured Temperatures with Predicted TELTD and Calculated EBITD for Sections B, C, and D

Location (Km)	Temperature, Top ($^{\circ}\text{C}$)	Temperature, Bottom ($^{\circ}\text{C}$)	TELTD ($^{\circ}\text{C}$)	EBITD($^{\circ}\text{C}$)
82,400	12.4	17.2 $^{\circ}\text{C}$	-14 $^{\circ}\text{C}$	-9.8 $^{\circ}\text{C}$
82,403	12.4	17.2 $^{\circ}\text{C}$	-18.7 $^{\circ}\text{C}$	-3.9 $^{\circ}\text{C}$
82,800	13.4	18.3 $^{\circ}\text{C}$	-26.3 $^{\circ}\text{C}$	-1.4 $^{\circ}\text{C}$
82,803	13.4	18.3 $^{\circ}\text{C}$	-26.6 $^{\circ}\text{C}$	-1.7 $^{\circ}\text{C}$
92,000	12.1	16.7 $^{\circ}\text{C}$	-19.8 $^{\circ}\text{C}$	-5.2 $^{\circ}\text{C}$
92,003	12.1	16.7 $^{\circ}\text{C}$	-19.9 $^{\circ}\text{C}$	-5.3 $^{\circ}\text{C}$
Average				-6.2 $^{\circ}\text{C}$

Route 60 Pavement Performance:

As noted previously, the pavement was opened to traffic 21 days after construction and had 100% slabs transverse cracked in less than ten days with the majority of slabs cracked in less than 24 hours. The section was immediately closed to traffic to analyze why the failure had taken place while recently constructed sections for the same roadway project were not opened to traffic. The transverse cracks can be seen in Figure 4. Longitudinal cracks appeared after one month in about 20% of the slabs, as noted in Figure 4, also. In order to predict the reasons for the premature failure of the section, ISLAB2000 was employed to calculate the critical tensile stress on the top of the slab and the M-E PDG fatigue algorithm was implemented. Table 6 lists the inputs used in the ISLAB2000 runs and tensile stresses including the number of load applications until 20% slabs cracked. Based on the surface profile measurements and backcalculation of EBITD, a value of -25°C was assumed along with -10°C nighttime temperature differential. Clearly, with the constructed slab geometry and thickness, level of built-in curling, and local climate for this section, a tensile stress of 4.9 MPa was generated which resulted in a high probability of early failure. Even when ISLAB2000 is used to calculate maximum tensile for the AASHTO 1998 design thickness with a -6°C equivalent temperature differential and the M-E PDG fatigue

algorithm, the maximum top tensile stress is 2.2 MPa with the allowable load applications of only 192,028 to produce 20% slabs cracked.

Table 6. Fatigue Performance (M-E PDG Fatigue Equation) of Various Slab Designs and Equivalent Temperature Differences

Concrete Pavement Case Analyzed	Max. top tensile stress (MPa)	Allow. Load applications	Thickness (mm)	Slab Length (mm)	Equivalent Temperature Differential (°C)	Axle Distance (mm)	K_c (MPa/m)
Actual Slab Design & curling	4.9	105	220	4000	-35	3500	120
3m slab with front axle of truck	3.3	1785	220	3000	-35	3500	120
3m slab with rear axle of truck	3.4	1329	220	3000	-35	2500	120
AASHTO 1998 Design- No EBITD	2.2	192,028	220	4500	-10	3500	120



Figure 4. Route 60 - 4 m long by 3.5 wide slab failures designed with AASHTO 1998.

To check the original AASHTO 1998 structural design with the M-E PDG, similar inputs were used, as shown in Table 7, except that a default equivalent built-in curling of -6°C was input for the M-E PDG. The required slab thickness of 220 mm for the M-E PDG was very similar to AASHTO 1998 design. If the measured built-in curling of -25°C was input into the M-E PDG, the thickness should have been 370 mm for the same level of slab cracking. Therefore, for the measured level of built-in curling (EBITD), the slabs did not have enough thickness to resist the expected load and environmental stresses. Note, the M-E PDG uses BIC as an input but also has an internally calculated moisture curling calculation which means BIC and EBITD are not exactly defined in the same manner.

Table 7. Comparison of M-E PDG and AASHTO 1998 Designs

	M-E PDG		AASHTO 1998
K-value	120 MPa/m		
Climate File	<i>Palmdale-California</i>		
Built-in Curling	-6°C	-25°C	-----
Concrete slab size (cm)	400x350		
Concrete Thickness (mm)	220	370	220
Serviceability	10% slabs Cracked	10% slabs Cracked	4.5–2.5 (P_f - P_i)

In order to minimize the problem the Ministry of Transportation decided to reduce the curling effect by re-designing the slab geometry. Table 6 shows the effect of slab tensile stresses when the slab length is reduced from 4 m to 3 m. The final slab size dimensions were 2 m long and 1.75 m wide based on the OptiPave design software (Covarrubias 2009, 2010). The roadway performance has dramatically improved, and at this time the pavement is 1.5 years old without signs of distress, as seen in Figure 5.



Figure 5. Route 78 2.00m x 1.75 m short slabs in area of high built-in curling

M-E PDG Design Inputs: The predicted performance of Route 78 was also evaluated with the M-E PDG and the corresponding design inputs for the various sections are shown in Table 8. The four different sections have many similar design inputs with some differences. The most important is in terms of the measured permanent effective temperature difference (°C), which is -11°C for section A (paved during nighttime) and -16°C for sections B,C & D (paved during daytime).

Table 8. Inputs on Design M-E PDG of the Different Subsections

SECTION	A	B	C	D
General Information				
Design life (years)	27	24	25	25
Construction month	April 1992	April 1995	April 1994	April 1994
Analysis Parameters				
Terminal IRI (m/km)	3.5	3.5	3.5	3.5
Transverse cracking (Percent of slabs cracked)	10	10	10	10
Mean joint faulting (mm)	5	5	5	5
Traffic				
Initial two way AADTT	2,305	2,660	2,535	2,535
Number of lanes	2	2	2	2
Design Features				
Permanent Curl/Warp effective temperature difference (°C)	- 11	- 16	- 16	- 16
Joint Spacing (m)	3.5	3.5	3.5	3.5
Erodibility Index	1	1	1	4
Layer 1 JPCP				
Layer thickness (mm)	255	250	250	250
Coefficient of thermal expansion (°C ⁻¹)·10 ⁻⁵	1	1	1	1
Aggregate type	Granite	Granite	Granite	Granite
Ultimate shrinkage at 40% R.H. (microstrain)	704	704	704	704
28 day modulus of elasticity (MPa)	30,286	30,286	30,286	30,286
Measured 28 day modulus of rupture (MPa); after Pfeiffer et al. 1996	4.75	4.75	4.75	4.75
20 years/28 days modulus of rupture ratio	1.25	1.25	1.25	1.25
Base				
Unbound material	A-1-a	A-1-a	A-1-a	A-1-a
Thickness (cm)	15	15	15	15
Modulus (MPa)	276	276	276	276
Subgrade				
Unbound material	A-5	A-5	A-5	A-5
Subgrade Modulus (MPa)	138	55	77	55

In order to have precise information in the climate characterizations of the project, two US stations were used for each location and combined as follows: the first station had the same latitude as the project section location, and in order to simulate the angle of the sun, all the climatic data was erased and exchanged with the climatic information of a second station. This new combined station was used to run M-E PDG for the different sections of the project

- Sections A and B are located near the city of Melipilla, Chile; the station chosen in terms of latitude was Blythe, CA and the station chosen for the climate data was San Jose, CA;

- Meanwhile Sections C and D are located on a coastal climatic zone, which are better represented with the city of Santo Domingo, Chile. The station of Santa Ana, CA was chosen in terms of latitude and Monterey, CA was selected as the station to best-fit the climate data

M-E PDG Results The M-E PDG performance results for the four sections are shown in Figures 6 to 8, in terms of IRI, mean joint faulting and percentage of cracked slabs, respectively. Each figure contains the M-E PDG results with the measured EBITD and default EBITD of -6°C (-10°F). Figure 6 includes an IRI-PSI correlated curve derived from a relationship developed in Chile for jointed plain concrete pavements (Solminihaç et al. 2003):

$$PSI_{AASHTO} = 5.769 - 1.589 \cdot (IRI)^{0.5} \quad (R^2 = 0.943) \quad (4)$$

The inputs used to calculate the slab thickness using the AASHTO 1993 design procedure are listed in Table 9. The IRI and mean faulting records measured on Route 78 are included in Table 14. IRI and Faulting Measurements for

Table 8. AASHTO 1993 Design Inputs

	A	B	C	D
ESALS on design lane	24,080,000	22,710,000	23,180,000	23,180,000
Standard Deviation	0.29	0.29	0.29	0.29
Initial Serviceability	3.76	3.64	3.7	3.7
K Value [MPa/m]	69.4	40.8	43.5	33.2
Measured 28 day MOR [MPa]	4.75	4.75	4.75	4.75
28 day E [MPa]	30,286	30,286	30,286	30,286
Drainage coefficient	1.25	1.25	1.25	1.25
Load transfer coefficient (J)	3	3	3	3
PCC Thickness [mm]	255	250	250	250

Table 9. IRI and Faulting Measurements for Route 78

	IRI (m/Km)						Faulting (mm)			
	2002	2003	2004	2006	2008	2009	2002	2003	2004	2005
Section A		2.3	2.3	2.4	2.5	2.6	2.4	2.6	1.7	1.7
Section B	2.6	2.7	2.8	3.1	3.1	3.1	2.4	2.8	3	-
Section C	2.5	2.5	2.6	2.8	2.9	2.9	2.8	2.6	2.7	-
Section D	2.6	2.7	2.9	3.1	3.2	3.3	3.1	4	3.5	-

Figure 6 shows a clear correlation between the IRI measurements and the IRI predicted curves from the M-E PDG using the field determined EBITD with the FWD procedure. Using the default value of EBITD (-6°C) for the M-E PDG, the roughness predicted on the pavement is considerably lower. The AASHTO 1993 curve obtained from the IRI-PSI correlation above is similar to the M-E PDG default curve. Since the AASHTO 1993 method does not consider built-in curling, the prediction capacity of the model can give variable results in places, such as Chile, where the EBITD is above average or high. The mean faulting performance of Route 78 shows a similar behavior to the IRI trends as seen in Figure 7. The correlation between the measured and predicted faulting curves that considers the actual EBITD value of the pavement is much greater than using the M-E PDG default value measured and predicted faulting curves that considers the actual EBITD value of the pavement is much greater than using the M-E PDG default value.

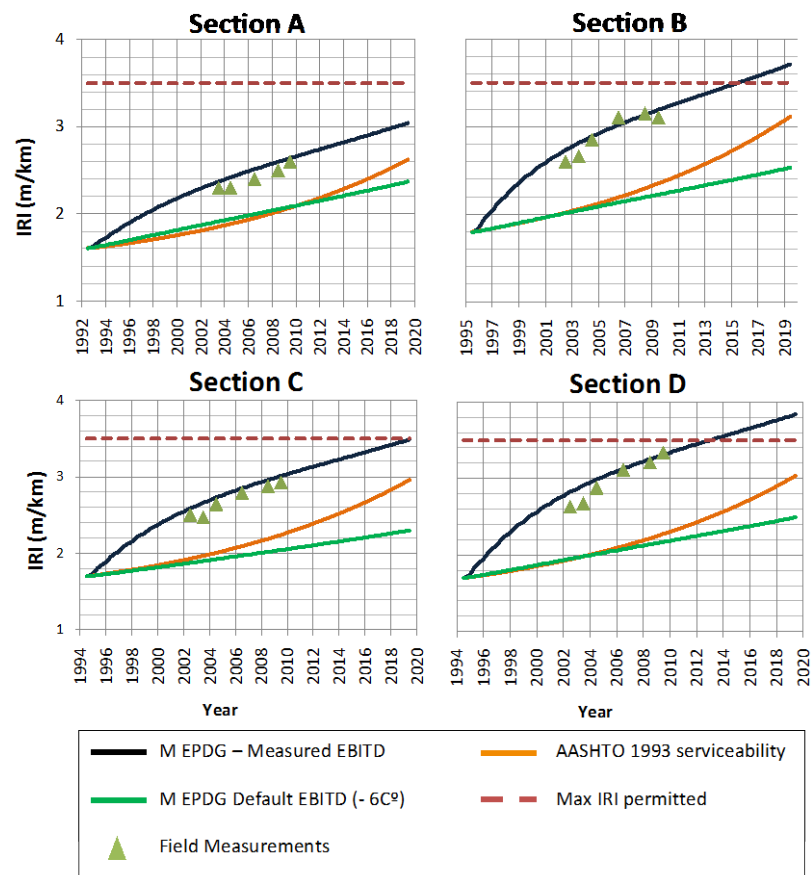


Figure 6. Predicted and Measured IRI curves for Sections A, B, C, and D, respectively, on Route 78.

With the default EBITD value in the M-E PDG, no slab cracking is expected on any of the four sections at the end of the analysis period as shown in Figure 8. The concession company that owns, operates, and maintains the roadway has reported that Section C has between 3% and 5% slabs cracked. Since no systematic collection of slab cracking data was available on this pavement project, an accurate comparison of the M-E PDG model prediction and actual measured cracking data on all sections was not possible.

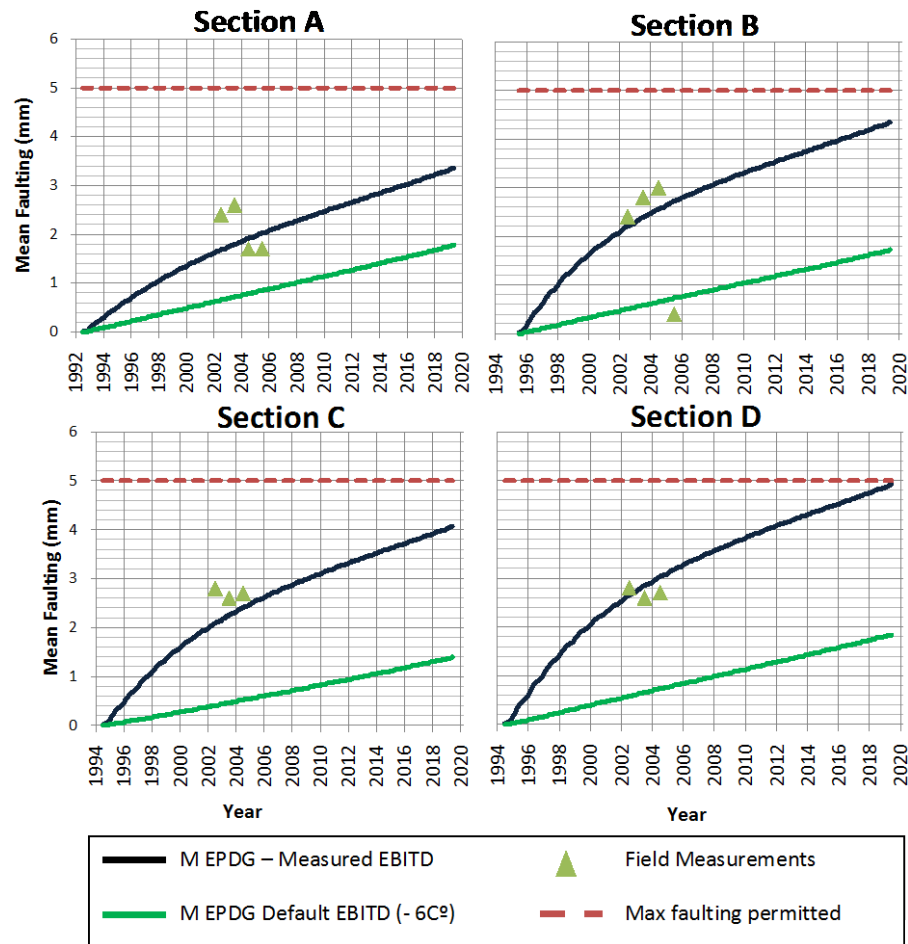


Figure 7. Predicted and Measured Faulting curves for sections A, B, C, and D, respectively, on Route 78.

Hypothetical pavement analyses with the measured EBITD for each section were completed to determine what may be their predicted cracking. Section A had a measured EBITD of -11°C and the M-E PDG reported cracking was 0.4%, similar to the concessionaires reported verbal performance. However, Section C had around 5% of slabs cracked as verbally reported by the concessionaire and when a measured EBITD of -16°C was used, the percent slab cracking was closer than the default EBITD used in the M-E PDG.

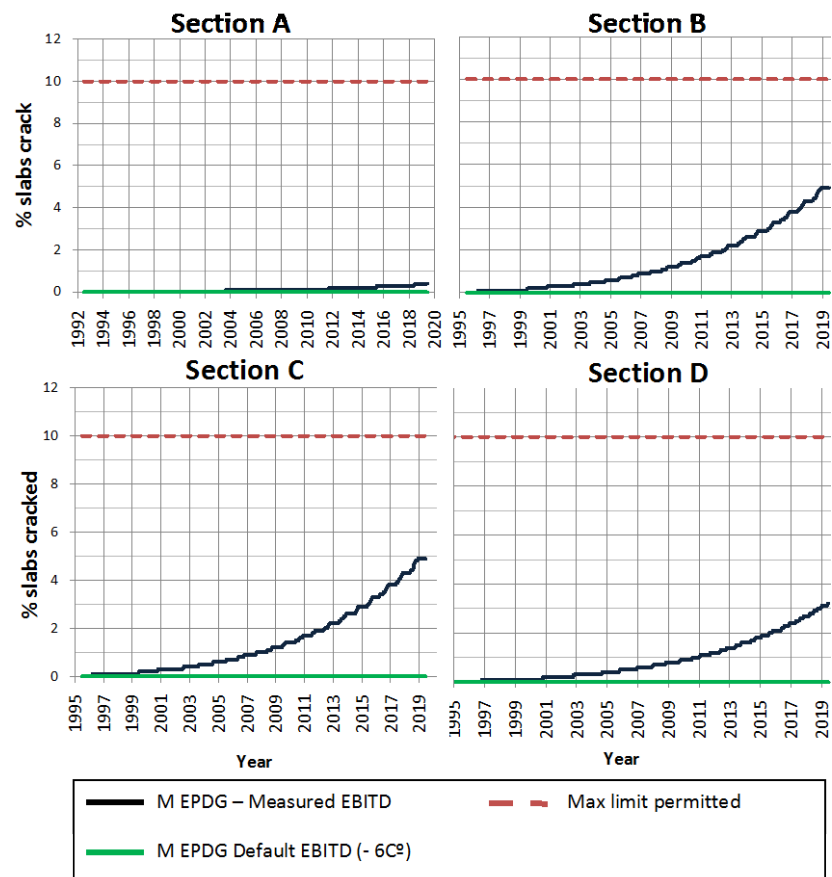


Figure 8. Predicted and Measured Faulting curves for sections A, B, C, and D, respectively, on Route 78.

Hypothetical pavement analyses with the measured EBITD for each section were completed to determine what may be their predicted cracking. Section A had a measured EBITD of -11°C and the M-E PDG reported cracking was 0.4%, similar to the concessionaires reported verbal performance. However, Section C had around 5% of slabs cracked as verbally reported by the concessionaire and when a measured EBITD of -16°C was used, the percent slab cracking was closer than the default EBITD used in the M-E PDG.

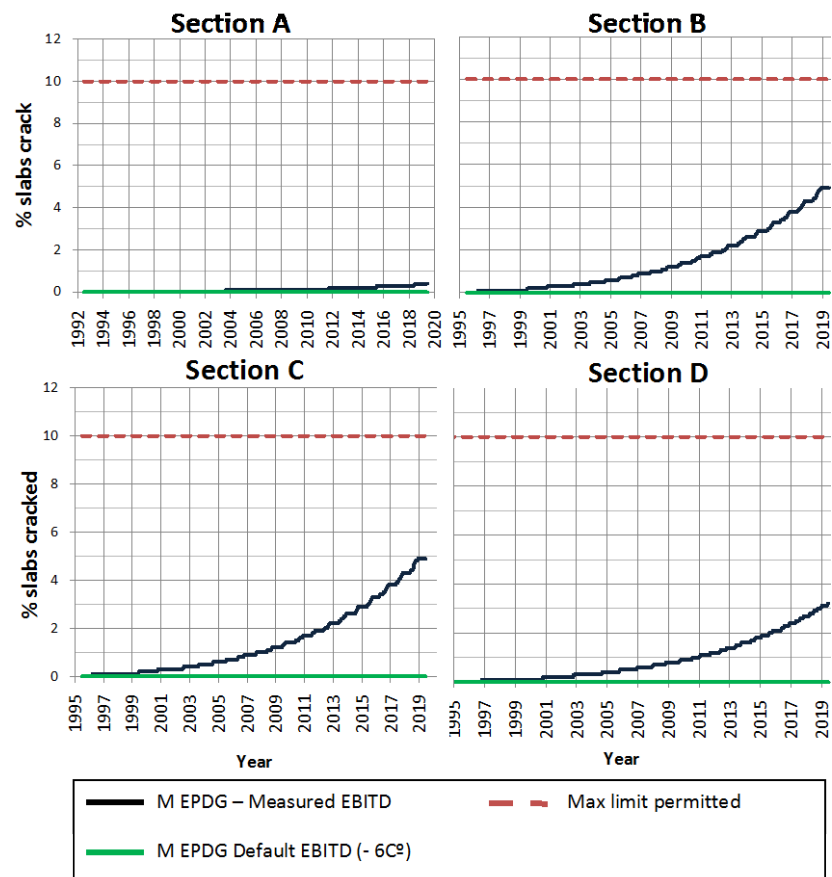


Figure 9. Predicted percent slabs cracked for sections A,

The same sections on Route 78 were re-analyzed with more favorable construction conditions for Sections A and B, e.g., nighttime conditions with EBITD of -11°C , and less favorable conditions for Sections C and D, e.g., daytime conditions with EBITD -16°C . Figure 9 clearly shows that the predicted pavement performance, measured in terms of IRI, faulting, and cracking, improved when the built-in curling was decreased. The terminal IRI decreased between 3.65 m/km to 3.05 m/km while the terminal faulting decreased from 4.5 mm to 3.0 mm. Finally, the percent slabs cracked with lower EBITD is close to zero while with less favorable construction conditions (EBITD -16°C) the cracked slab prediction was between 3% and 8%.

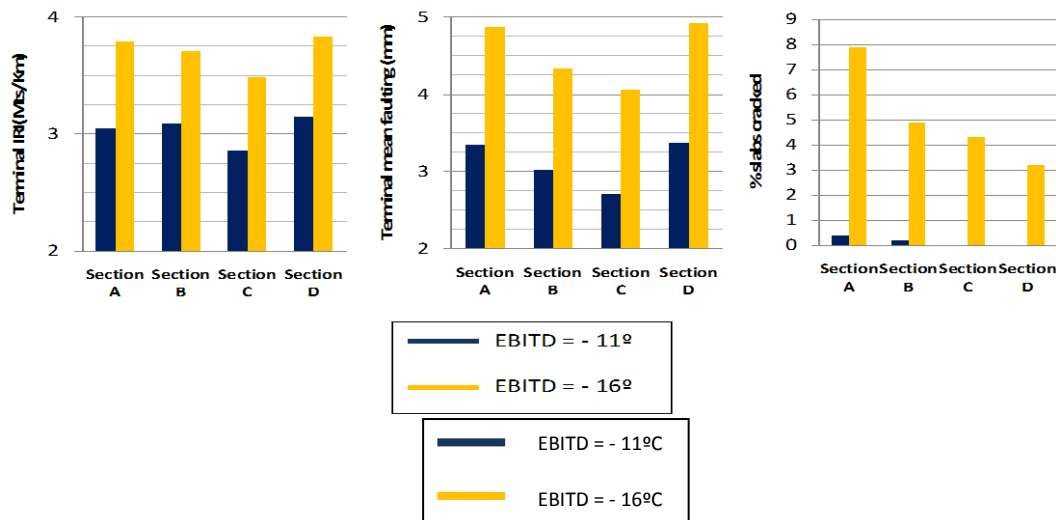


Figure 10. Final distress and serviceability of the pavement relative to the initial built-in curling conditions.

Additional M-E PDG runs were completed to compare the required structural design for Section A on Route 78 at several levels of EBITD (-11°C, -16°C, -25°C). These levels of EBITD were selected based on the least negative EBITD measured in this study, intermediate value, and the lowest EBITD on Route 60. Figure 10 demonstrates the reduction in the required slab thickness for various failure criteria when concrete slabs are built with lower levels of built-in curling. For the -11°C EBITD and a slab thickness of 255 cm, the terminal IRI was 3.05 m/km, the mean faulting was 3.35 mm, and the percentage of cracked slabs was only 0.4%. These values were compared with predicted pavement performance at different EBITD values (-16°C and -25°C). As expected, a greater thickness is necessary to counteract the effect of greater built-in curling. The concrete pavement with EBITD of -16°C needed a 285 cm thickness slab to have similar terminal IRI and faulting, compared to a 255 cm thickness slab with an EBITD of -11°C while a required thickness of 295 cm is needed to achieve the same level of transverse cracking, a 15.6% increase. If the concrete pavement is constructed with a large EBITD of -25°C, a slab thickness of 31.5 cm is required at the same cracking performance (23.5% increase), and 330mm slab thickness is needed to satisfy both the terminal IRI and mean faulting criteria.

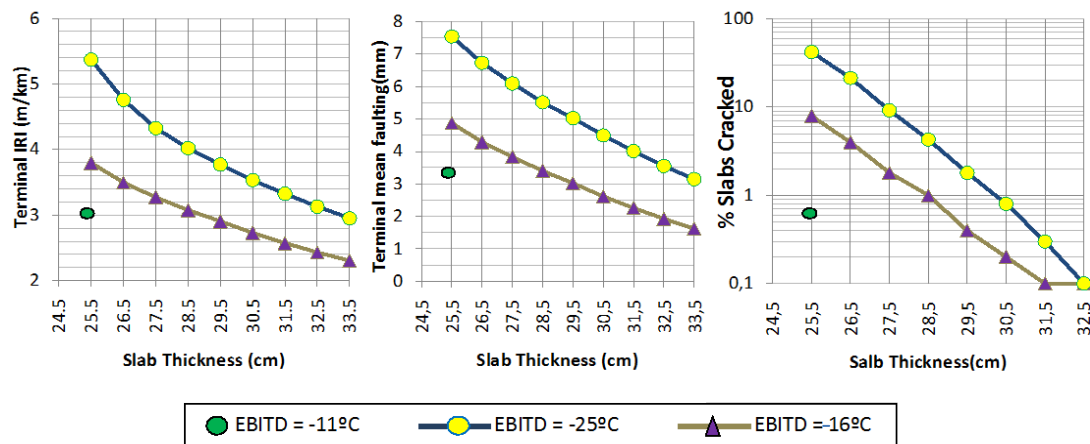


Figure 11. Required slab thickness for failure criteria of IRI, faulting, and slab cracking at different levels of EBITD.

The design analysis quantified in Figures 6 through 10 can lead to better understanding of the importance of the interaction between curing practices, time of casting, and the concrete pavement structural design. EBITD can be minimized by reducing the thermal gradient set into the slab after concrete hardening and the reduction in the moisture loss near the upper surface of the concrete slab. Construction procedures, materials selection, and climatic variables are very important in the final EBITD. Climatic conditions at the time of paving are especially key factors in the performance of the concrete pavement. High temperatures, strong wind, high solar radiation, and low ambient humidities are factors that have to be considered in construction procedures and timing of paving.

Conclusions

The AASHTO 1993 and AASHTO 1998 (used in Chile) rigid pavement design methods do not consider built-in curling (BIC) as a design input while the new mechanistic-empirical pavement design guide (M-E PDG) uses a default value of -6°C (-10°F). A study was undertaken in Chile to measure the in situ built-in curling, called effective built-in temperature difference (EBITD) and performance for several concrete pavement projects and compare it to the predicted performance from the M-E PDG. The EBITD was measured on two projects Routes 60 and 78, using the surface elevation of the slab and FWD testing, respectively. The EBITD measured in Chile ranged from -11°C to -41°C depending on the geographic location (local climate) and principally on the time of paving and construction practices. The EBITD was considerably higher than the default value assumed in the M-E PDG of -6°C . The project with the highest EBITD showed extensive cracking within one week after opening to truck traffic while the other project showed higher levels of IRI, faulting, and cracking with larger values of measured EBITD. Significantly thicker slabs were required for regions that had high, measured EBITD values. Future designs of concrete pavements in Chile must consider the real magnitude of EBITD as a design input and the use of short slabs (2.00 m x 1.75 m) could be a solution to minimize the

effect of BIC and to avoid premature failures. Likewise, construction activities such as curing, time and season of paving must be adjusted to minimize the level of BIC.

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