

A Case Study of Slab Cracking and Delamination in BCO Construction at the Bush International Airport (IAH) – Part II: Causative Factors

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Abstract

Bonded Concrete Overlay (BCO) construction at the Bush International Airport (IAH) has been subjected to incidences of cracking and delamination that is speculated to be mainly climatically driven due the extremely dry and windy weather conditions prevalent throughout most of the construction period. An investigation was undertaken to determine the causes of the failure of the bond between the overlay and existing pavement as well as the mechanism associated with that. This paper reports portions of the details of the laboratory and field investigation into the distress mechanism but focuses mainly on the analytical approach used to assess the probabilities of the types of cracking and delamination observed.

Introduction

In the latter half of May 2011, the engineering staff of the Houston Airport System (HAS) noticed extensive cracking in portions of a bonded concrete overlay (BCO) construction on Taxiway (T/W) “WA” at the Bush International Airport that was 22.9 m (75 ft) wide with a 7.6 m (25 ft) wide keel section. The outer 7.6 m (25 ft) lanes consisted of a 203 mm (8 in) BCO on the existing 457 mm (18 in) concrete pavement while the keel section was paved full-depth. Paving operations commenced in February 2011 and terminated in the latter part of April 2011 and it was not long afterwards that extensive cracking in the overlay concrete had taken place. The existing concrete that was overlaid consisted of a 762 mm (30 in) section of concrete pavement with joints every 7.6 m (25 ft) that were matched to the joints in the BCO with full-depth sawcuts. Many of the details of the construction and the subsequent cracking distress are presented in Part I of this paper but the purpose here is to examine the factors associated with the incidence of noted cracking and delamination distress.

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Failure Mechanism

Many factors can synergistically combine to affect early age stress development leading to cracking and delamination in portland cement concrete (PCC) pavement overlays. Jang (2005) and Lange et al, (2001) put many of these factors into perspective incorporating the key aspects of early age jointed concrete stress and strength development over time. The stresses associated with the observed cracking are thought to mainly be caused by curling/warping behavior of the overlay slab which is of course volumetric strain related. A portion of the volumetric moisture and temperature induced changes take place within the concrete contribute to a vertical strain gradient causing the slab to deform (Figure 2a); while other components of it may also contribute to horizontal contraction between the slab and the supporting medium in the case of an overlay.

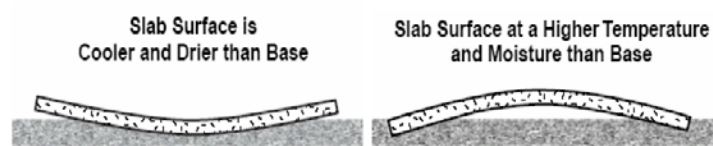


Figure 2a Slab Curling and Warping Action.

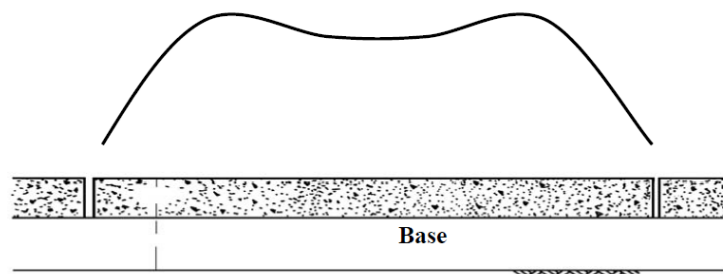


Figure 2b Slab Curling and Warping Pattern of Stress.

During hardening, the shrinkage of the surface concrete (principally due to evaporation with some self-desiccation if the water content is low enough in the concrete) is often greater than those near the bottom of the overlay. The drying action of the concrete causes the top of the slab to contract with respect to the bottom thus warping the slab in a concave configuration. This action could cause one or a combination of the following to occur:

- 1) Lifting of the overlay and the underlying slab at the corners and along the edges before delamination occurs. Slab movement in this manner would be driven by the developing moisture gradient in the overlay while the action of the slab would be dominated by the structural composite made up of the existing, underlying 457 mm (18 in) layer and the 203 mm (8 in) overlay. Tensile stresses would be induced at the top of the slab and along the interface primarily due to the dead weight of the concrete in the overlay and in the existing slab counteracting the lifting caused by the shrinkage induced volume changes in the BCO.
- 2) Enough upwards lifting at the slab corners and edges to cause the tensile stress at the slab/overlay interface to exceed the interfacial strength and to separate the BCO from the supporting medium. To the degree that the delamination strength of the interface exceeds the delamination stress in

the first case will be the degree that the composite slab effect will dominate the curling action. Otherwise, if the delamination strength is exceeded by the tensile stress due to warping action' the overlay will be allowed to separate from the underlying support.

- 3) Warping of the slab on the underlying 457 mm (18 in) slab after delamination occurs. In this case, the overlay concrete has broken free of bonding to the substrate allowing very stiff support conditions to affect the tensile stresses that would be introduced in the top of the slab and compressive stresses at the bottom of the slab when temperature effects cause a convex shape to occur.

It is postulated that crack development in a BCO is a dynamic split between the three possibilities noted above. In this sense, the assessment of the cracking should be configured in the form of the probability associated with each and the concomitant variability associated with such factors as materials, curing quality, and weather conditions at the time of paving. Thus the following discussion outlines an approach to analyze the probability of these effects as part of an investigation into the causative factors to cause the type of cracking and delamination associated with the taxiway BCO paving in this instance to occur.

Methodology to Assess the Probability of Slab Cracking and Delamination

Over the days that concrete placements were made on T/W WA, the ambient temperatures at the time of placement were estimated to range from approximately 15°C (60°F) to a high of 21°C (70°F) during the March time frame and from approximately 21°C (70°F) to a high of 26°C (80°F) in the April time frame. Wind conditions varied but averaged from 8 (5) to 24 km/h (15 mph) and had gusted often over 48 km/h (30 mph). During setting, the concrete would be subject to thermal and drying shrinkage strain gradients which, if high enough, would be expected to induce a high probability of damage to occur. Relative to that, the following circumstances and factors are considered:

- 1) The development of strength over time. Both early-age and long-term concrete strength is a function of mixture characteristics such as cement and water content, aggregate type as well as the method of curing and the ambient climatic conditions at the time of placement.
- 2) Assessment of the ultimate concrete shrinkage and creep strain. These effects are interrelated on several levels of behavior but are also influenced by many of the factors that affect the gain of strength over time. Shrinkage strain contributes to cracking potential and creep strain diminishes it over time.
- 3) During the hardening process, concrete typically undergoes early age temperature and humidity changes applied at the slab surface which lead to temperature and drying shrinkage gradients and a combination of curling - induced thermal (ϵ_{th}) strains and warping - induced shrinkage (ϵ_{shr}) strains in the concrete layer. These effects tend to deform the slab and induce stress levels that are often sufficient to induce cracking due to the dead weight of the slab.
- 4) The assessment of curling and warping behavior in a bonded overlay as it is related to climatically-induced stresses (in comparison to the strength of the concrete over time) can perhaps be broken down into the three general categories of potential cracking: before delamination, after delamination, and delamination itself.

- 5) The assessment of the variability in the development of strength and stress in the concrete bonded overlay relative to the determination of the probability of cracking over time.

The overlay concrete was not a typical paving mixture in that it included a mid-range water reducer calling for a w/cm of 0.23. However, upon delivery, the w/cm may have been closer to 0.31. The mixture consisted of 6 sacks of cement with 25% Type F fly ash.

Concrete Strength Gain

The overlay concrete was specified to achieve a compressive strength (f'_c) of 52 MPa (7500 psi) at an age of 28 days. Using the following model:

$$f_s = a \log(t) + b$$

Where:

- f_s = compressive or tensile concrete strength, MPa (FL⁻²)
- a, b = calibration parameters
- t = time since placement

Assuming the compressive strength gain doesn't begin until the concrete is 6 hours old, the resulting ascension of strength gain with time for the concrete would result in the calibration parameters being set at:

- a = 25.23
- b = 15.19

Ultimate Shrinkage and Creep Strain

Although the mechanisms of creep and shrinkage are enormously complex, simplified procedures to estimate the ultimate value of shrinkage strain ($\epsilon_{sh\infty}$) as well as its development over time are outlined by the American Concrete Institute (ACI 209 R2-08). The factors considered in these procedures for shrinkage are:

- Ambient relative humidity (~60%)
- Element thickness (8 in)
- Concrete slump (~1 in)
- Percent of fine aggregate in the mixture (~30%)
- Cement content (~6 sacks)
- Air content (~5%)

Ultimate drying shrinkage ($\epsilon_{sh\infty}$) is defined as the shrinkage strain long after the end of the initial curing. Bazant and Wu (1974) suggested the following equation for $\epsilon_{sh\infty}$:

$$\epsilon_{sh\infty} = 1330 - 970y$$

Where

$$y = (390z^{-4} + 1)^{-1}$$

$$z = 0.381\sqrt{f'_{c-28}} \left[1.25\sqrt{\frac{a}{c}} + 0.5\left(\frac{g}{s}\right)^2 \right] \left[\frac{1 + \frac{s}{c}}{\frac{w}{c}} \right]^{1/3} - 12$$

- a/c = aggregate-cementitious material ratio (~5.78),
- g/s = gravel-sand ratio (~1.42),
- s/c = sand-cement ratio (~2.45),
- w/c = water-cementitious material ratio (~0.22),
- f'_{c28} = compressive strength at 28 days in units of ksi (7.5)

Although, not all the same parameters are directly involved, many are related to those that are listed by the ACI method and are used to make the determination. The ACI formula yielded a value of 436 $\mu\epsilon$ which was reduced to 200 $\mu\epsilon$ in this analysis of cracking potential based on shrinkage test results referred later in the paper. Unfortunately, little direct information was available for the determination of the early aged creep strain but it is important to make a selection for it since there is a certain amount of relaxation of the restrained strain levels (approximately one-third of the ACI determined value) that occurs in hardening concrete during the early ages. A value of 125 $\mu\epsilon$ was used in the analysis subsequently described.

Early Age Environmental Strains over Time

The interaction of drying shrinkage (ϵ_{shr}) or temperature strain (ϵ_{temp}) within the concrete and the level of restraint inherent to the curling and warping of the slab system was evaluated in terms of the vertical humidity and temperature gradients as a function of temperature change and drying that took place at and in the vicinity of the surface of the slab. According to Bazant and Wu (1974), the drying shrinkage of concrete can be described by the following function of humidity as:

$$\epsilon_{shr} = \epsilon_{shr\infty} (1-rh)^3 \quad (1)$$

Where, rh is the internal concrete relative humidity and $\epsilon_{shr\infty}$ is a material parameter, which is the ultimate concrete shrinkage at a reference relative humidity of 50%. Since the drying gradient strain is of interest, equation (1) is expressed as:

$$\epsilon_{shr} = \epsilon_{shr\infty} \Delta(1-rh)^3$$

Where $\Delta(1-rh)^3$ is referred to as a humidity coefficient and is the difference in $(1-rh)^3$ between the pavement surface and its value at the bottom (which is typically calculated by moisture diffusion analysis subsequently described). A similar expression is used to assess the temperature related strain for curling behavior (Mohamed, and Hansen, 1996):

$$\epsilon_{th} = \alpha_c \Delta T$$

Where ΔT is the difference in temperature between the pavement surface and the bottom and α_c is the coefficient of thermal expansion of the concrete (a value of 4.5 $\mu\epsilon$ was selected for this parameter based on the Limestone coarse aggregate type). Parrot (1988) suggested the following equation to determine the relative humidity (rh_i) and various depths below the surface of a pavement as:

$$rh_i = rh_a + (100-rh_a)f_{ti}$$

Where

rh_a = is the ambient relative humidity.

f_{ti} = is the moisture depth and age factor

$$= \frac{1}{\left(1 + \frac{tc}{b_i}\right)}$$

c = is the curing calibration coefficient determined as a function of effective curing thickness ($L \sim 1$ in.) which represents the assumed quality of curing where an L = value is determined based on actual curing monitoring results for a given set of evaporative conditions.

$$= 123.33 - 70.89 \cdot \text{Log}(L)$$

$$b_i = \frac{(y_i)^{1.35} (70 - \% \text{ flyash})(w/c - 0.19)}{8}$$

y_i = depth below surface

%flyash = % of fly ash in the concrete mixture

t = concrete drying age

rh_i is used to calculate the humidity coefficient $\Delta(1 - rh_i^3)$.

Temperature Strain

ACI 207 lists a simple approach to determine the change in temperature with depth by the following:

$$\frac{R_i}{R_0} = e^{-x \sqrt{\frac{\pi}{h^2 \gamma}}}$$

Where

R_i = calculated range in temperature at depth i

R_0 = range in temperature at the pavement surface

h^2 = thermal diffusivity of the concrete ($\sim 1.39 \text{ ft}^2/\text{day}$)

γ = frequency of the temperature cycle (1 day)

The temperature gradient (ΔT) in the slab was found from:

$$\Delta T = T_0 - T_i = T_{\text{avg-air}} - T_{\text{avg-sbt}} + \frac{1}{2}(R_i - R_0)$$

and

$$\varepsilon_{th} = \alpha_c \Delta T \text{ (previously noted)}$$

Where

T_0 = surface temperature

T_i = temperature at depth i

$T_{\text{avg-air}}$ = average air temperature

$T_{\text{avg-sbt}}$ = average substrate temperature

Although, the temperature of the slab at the bottom is controlled to a large extent by the temperature of the substrate or existing concrete, there most likely is an effect due to the set temperature of the concrete which in this case varied from 80 to 100°F over the time frame from February to April. Table 1 lists much of the data that was needed to carry out an analysis of the temperature and moisture gradients contributing to curling and warping stresses in the overlay concrete for two specific days of paving where cracking levels were very different. Both temperature and humidity data that is listed for these days of placement was weighted with respect to the potential evaporation (PE – ACI evaporation nomograph) to represent the curing condition over the hardening periods of the concrete, simplifying the consideration of the effect of climate in the analysis.

Net Strain and Stress Computations

A simplified form of the Westergaard theory (Bradbury, 1938) suggests that curling and warping stress (σ) can be estimated as:

$$\sigma = C\sigma_0$$

Where

C = slab restraint factor (function of slab length and the radius of relative stiffness (ℓ))
= 1.043 for these project conditions

$$\sigma_0 = \frac{E\varepsilon_T}{2}$$

ε_T = net strain due to drying shrinkage, temperature change and creep effects
= $\varepsilon_{shr} + \varepsilon_{th} - \varepsilon_{crp}$

E = modulus of elasticity of concrete
= $4730\sqrt{f'_c}$ MPa

The net strain (ε_T) varies considerably during the early-age strength gain phase due to the effects of moisture loss, strength gain, and hardening. It's consideration in this analysis is simplified to represent an average state of the conditions prevalent throughout the early stages of hardening. Creep effects typically dominate over the first 2 to 3 days resulting in low cracking probabilities over that time period but diminish soon after that increasing the cracking potential until the strength gain is sufficient to lower it. The change in shrinkage and the creep components of the net strain change are assumed to change with time due to the effects of aging on concrete properties but age has a lesser effect on temperature induced strains. Since the climatic effects are being averaged over the analysis period, the time effects on temperature induce strain are not as prevalent in this analysis.

The value of E is commonly taken to be a function of the strength of concrete and as such is subject to increase as the concrete ages particularly during and immediately after the hardening stages. The increase in modulus with time results in an increase in stress with time as well but this is again offset to some extent by the increase in tensile strength (f_t) of the concrete. The split tensile (f_{st}), compressive (f'_c), and tensile (f_t) strength of concrete has been found to be related as:

$$f_t = 0.85 \cdot f_{st} = 0.85 \cdot 81\sqrt{f'_c}$$

The parameter 'C' in the determination of σ has a key role in describing the effects of slab dimensions and configuration on cracking potential. 'C' is effectively only a function of the slab length and the slab radius of relative stiffness (ℓ) - which has similarities to the moment of inertia of a beam such as involving the cube of the slab thickness (ℓ -values for concrete slabs typically range anywhere from 635 mm (25 in) to 1524 mm (60 in)). At slab lengths equal to or greater than 4.5ℓ , the potential for slab cracking is at its greatest level since that is where the value of 'C' is the greatest.

Table 1 Construction Weather and Placement Data.

Weather	Level	Mar 7	July 18
Temperature °C (°F)	Low	7.7 (46)	26.7 (80)
	Avg	13.3 (56)	30 (86)
	High	18.9 (66)	33.9 (93)
Humidity (%)	Low	58	46
	Avg	78	70
	High	93	87
Wind Speed Km/h (mph)	Avg	16 (10)	6.6 (4)
	High	35.4 (22)	12.9 (8)
	High	46.7 (29)	48.3 (30)
PE Kg/m ² /h (lbs/ft ² /h)	Avg	0.88 (0.18)	0.88 (0.18)
	High	1.71 (0.35)	1.47 (0.30)
	Gust	2.15 (0.44)	4.44 (0.91)
%Cracking		5.6%	100%
Delamination		33%	75%
Effective Thickness mm (in)		102 (4.0)	19 (0.75)

Slab Delamination Stress

Even though slab delamination is largely a result of slab curling and warping behavior, closed-form solutions for the stress between the slab and the substrate are essentially non-existent suggesting that a detailed finite element analysis would be in order to determine such stresses. Nonetheless, it is possible to formulate a solution based on a few simple concepts introduced in a paper on analysis of curling and warping by Tang and Zollinger (1993) regarding the formulation of curling stress due to dowel bar stiffness. The moment in the slab (M_0) was found a

$$M_0 = \theta_0 K_r$$

Where

θ_0 = dowel bar rotation due to bending

$$= \frac{w_0}{\frac{\ell}{\sqrt{2}} + \frac{K_r}{k\ell^2}}$$

K_r = dowel bar rotational stiffness

w_0 = edge or gap displacement at the slab edge = $\ell^2 \left(\frac{1+\nu}{h} \right) \varepsilon^T$

ν = Poisson's ratio

h = slab (i.e. overlay) thickness, L

k = foundation modulus, FL⁻²/L

$$\ell = \text{radius of relative stiffness} = \sqrt[4]{\frac{Eh^3}{12(1-\nu^2)k}}$$

$$E = \text{Concrete modulus, FL}^{-2}$$

The slab only rotation (θ_0) could be formulated as $\frac{w_0}{\ell \sqrt{2}}$ while assuming that the

existing substrate acts as a restraint to rotation (similar as a dowel bar would), one could formulate an expression for K_r as:

$$= \left(\frac{\ell_e - \ell}{\sqrt{2}} \right) k \ell^2$$

Which takes into account the stiffness differences between the overlay (by itself - ℓ) and the overlay plus the existing slab (ℓ_e). Assuming the moment in a cantilever beam (of length L) subjected to a linearly distributed load with a peak value of σ_0 , can be formulated as:

$$= \frac{1}{3} \sigma_0 L^2$$

And substituted for M_0 , an expression for σ_0 as the imposed loading or the bond stress between the concrete overlay and the substrate can be formulated as:

$$\frac{\sigma_0 L^2}{3} = \frac{\sigma_0 s^2}{3} = \frac{w_0 \sqrt{2}}{\ell_e} K_r = \frac{w_0 \sqrt{2}}{\ell_e} \left(\frac{\ell_e - \ell}{\sqrt{2}} \right) k \ell_e^2$$

$$\sigma_0 = \frac{3w_0}{s^2} (\ell_e - \ell) k \ell_e$$

Where 's' represents the length of the delamination from the slab edge.

Probability of Cracking

Using the above formulations, the probability of cracking or delamination can be assessed by determining the probability of $\sigma_{net} > 0$ where net stress (σ_{net}) can be determined as essentially the difference between the stress level and the strength of concrete or interface:

$$\sigma_{net} = \sigma - f_t$$

Where

σ = either the curling and warping or the bond stress

f_t = concrete strength

The probability that $\sigma_{net} > 0$ can be formulated in terms of the normal standard deviate z (assuming σ_{net} is normal distributed) as (Ang and Tang, 1975):

$$z = \frac{\sigma_{net} - 0}{\text{sqrt}[Var[\sigma_{net}]]}$$

$$Var[\sigma_{net}] = \sum \left(\frac{\partial \sigma_{net}}{\partial X_i} \right)^2 Var[X_i] + \sum_i \sum_j \left(\frac{\partial \sigma_{net}}{\partial X_i} \cdot \frac{\partial \sigma_{net}}{\partial X_j} \right) \sigma[X_i] \cdot \sigma[X_j] \rho_{ij};$$

Where $\sigma[]$ is the standard deviation of X_i which are the factors (strength (f_t), strain (ϵ_T), slab thickness (h), modulus, (E), etc.- see Table 2) which affect the variance of the net stress which is found as:

$$\text{Var}(X_i) = (\overline{X_i} \text{Cov}[X_i])^2$$

The correlation factor, ρ , was assumed to be zero in this analysis. The breakdown of the listed factors affecting the variability of the net stress is provided in Table 2. On this basis, the probability of cracking can be found as a function of time.

Table 2 Cracking Probability Components.

X_i	$\frac{\partial \sigma_{net}}{\partial X_i}$	$\text{Cov}[X_i]$
f_t	$\frac{\partial \sigma_{net}}{\partial a} = (\text{Log}(t))^2; \frac{\partial \sigma_{net}}{\partial b} = 1$	$\text{Cov}(a)=0.10$ $\text{Cov}(b)=0.10$
ϵ_T	$\frac{\partial \sigma_{net}}{\partial \epsilon_T} = \frac{\sigma}{\epsilon_T}$	0.15
k	$\frac{\partial \sigma_{net}}{\partial k} = \frac{\sigma}{k}$	0.20
s	$\frac{\partial \sigma_{net}}{\partial s} = -\frac{2w_0}{s}$	0.40
ℓ	$\sum \left(\frac{\partial \sigma_{net}}{\partial \ell} \right)^2 \text{Var}[\ell];$ $\text{Var}[\ell] = \sum \left(\frac{\partial \ell}{\partial x_i} \right)^2 \text{Var}[x_i];$ $x_i = E, h, k$	$E = 0.15$ $h = 0.10$ $k = 0.20$

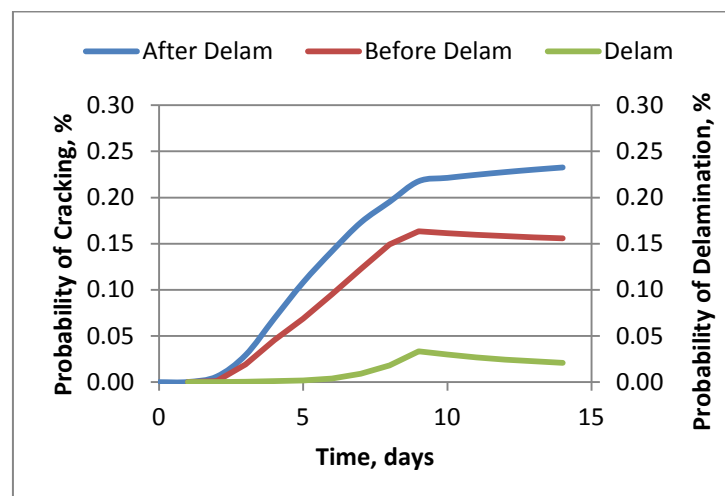


Figure 3 Distress Probabilities for Mar 5-7th.

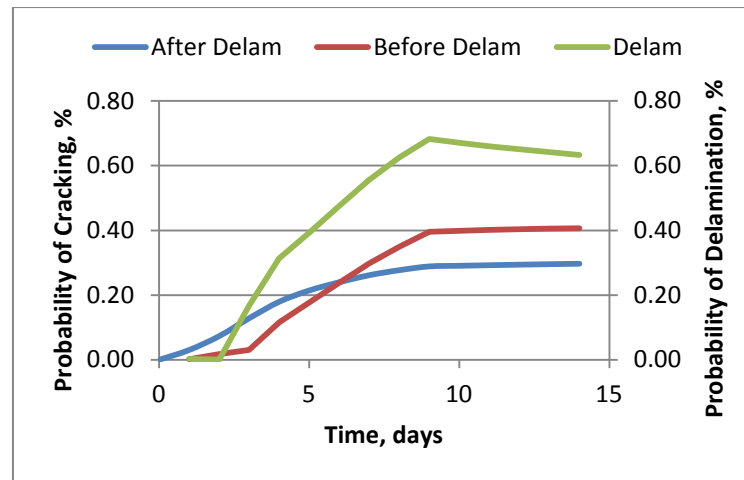


Figure 4 Distress Probabilities for July 18th.

Assessment of Causes and Effects

Field observations of the incidence of cracking and delamination as well as examination of cracking and delamination combinations suggested that some cracking occurred before delamination and some afterwards (as noted in the previously described cases) but that much of it occurred at daily temperature cycles below 30°C (85°F) (temperature and cracking data was not included due to space limitations). As a consequence, the potential

or the probability of cracking or delamination according to the three scenarios described previously were considered in detail for a relatively moderate weather paving day (i.e. Mar 7th) as well as a relatively severe weather paving day (i.e. July 18th). The weather data relative to these conditions are listed in Table 1. Following the methodology of assessing the cracking or delamination stresses as outlined above yields the results shown in Appendix A. The assessment of the probability associated with either cracking or delamination is shown Figures 3 and 4 with respect to:

1. The quality of curing; one measure of quality is the effective curing thickness which is the equivalent thickness of concrete that provides the same amount of curing as the applied curing compound. The data to calibrate the effective curing thickness for the days represented in Table 1 was taken from collected concrete and surface humidity data similar to that shown in Appendix B.
2. The amount of cracking and delamination as indicated in Table 1.

The results summarized in Figures 3 and 4 show the following:

1. Comparing the analysis of the March 2011 paving to the July 2011 paving, it is clear that the harsher the weather conditions the higher the probability for cracking for the same level of curing quality.
2. The potential for cracking before delamination is greater than the potential for delamination under milder PE conditions but is vice-versa for more severe conditions.
3. The potential for slab cracking is greater after delamination occurs due to the stiffness of the substrate.
4. Based on the probability charts, it appears that a certain amount of delamination should be expected in BCO construction especially for harsher evaporative conditions; and that there is a lower risk of it occurring relative to slab cracking prior to delamination for less evaporative conditions.

Other measures of curing were considered as part of the data collection effort during curing testing using the curing monitoring system (described in Appendix B) under both field and laboratory conditions (sample of the data is shown in Appendix B); one such parameter is referred to as the evaluation index (EI):

$$EI = \frac{t_{e-filtered} - t_{e-open}}{t_{e-sealed} - t_{e-open}}$$

Where:

$$t_{e-I} = \sum \beta_h \beta_T \Delta t = \sum_i \frac{1}{5 + 5rh_i} \cdot \frac{T_i - (-10)}{T_{ref} - (-10)} \Delta t_i$$

i = indices for filtered, sealed, or the open temperature or relative humidity
 Δt = time increment
 T_{ref} = reference temperature ($\sim 20^\circ\text{F}$)

In some respects, the $t_{e-filtered}$ also serves as an indicator of quality curing particularly in reference to the actual elapsed time. Keeping in mind, the $t_{e-filtered}$ parameter is an equivalent time (or even a ‘maturity’) determined relative to the temperature and relative humidity of the concrete at the surface – immediately below the curing medium. The closer the value of $t_{e-filtered}$ to the actual elapsed time, the better the quality of curing that has been achieved. Finally, the amount of drying shrinkage is again another indicator of curing quality that was recorded since the lower the average relative humidity the greater the amount of drying shrinkage that results as would be measured with a strain gauge. As part of this study, a limited laboratory and field study of curing quality and shrinkage was carried out and summarized in Figure 5 as a function of the rate of curing associated with each tested specimen. As expected, the $t_{e-filtered}$ parameter decreases as the rate of application decreases as does the EI. What is surprising but perhaps not unexpected, is the decrease in drying shrinkage as the rate of curing application increases but then appears to reverse that trend as the rate of application exceeds a certain threshold amount. It is not entirely clear as to why the data manifest this trend but it is interesting and might suggest that curing quality (at least with respect to the use of curing membranes) involves the loss of a certain amount of moisture from the concrete. On this basis, the data in Figure 5 suggest the optimum rate of curing is between 1 gallon per 120 to 100 sq ft and the minimum EI for paving should be 0.80. With respect to the EI, conducting paving operations at night appears to cause an increase in the EI of approximately 15 to 20%.

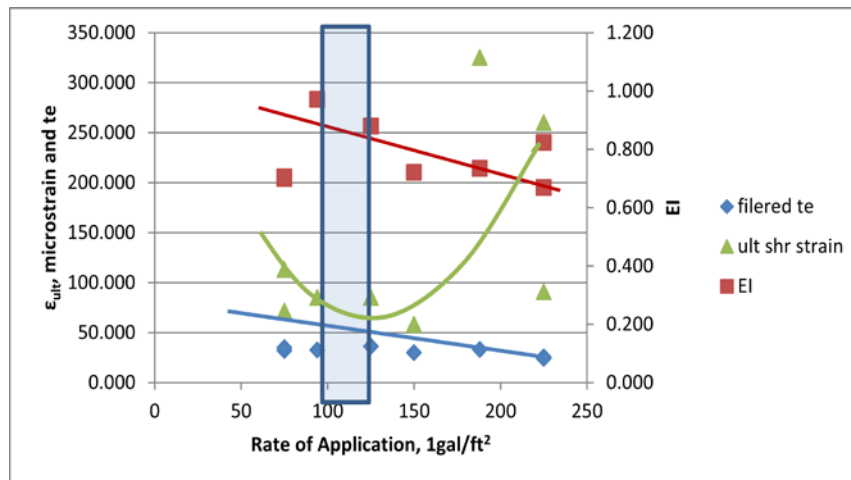


Figure 5 Summary of Curing Quality Testing Data from Appendix G and H.

Summary

Based on the comparison of the data shown in Figures A.1 through A.4 and the probabilities shown in Figures 3 and 4, it appears that under certain combinations of wind and relative humidity that BCO construction may be subject to a relatively high risk of cracking in the overlay (i.e. before delamination takes place) - and if the weather conditions are severe enough, the risk of delamination becomes greater. Even with the daily ambient temperature below 30°C (85°F), the probability of slab cracking discussed above was high enough for cracking to be initiated. Examination of the probability of cracking before and after delamination leads one to conclude that the establishment of a temperature limit of 30°C (85°F) pertains mainly to the former and that limit was not consistently exceeded until the latter half of April after a large percentage of the T/W WA paving had been completed. Construction guidance on mitigation measures during construction based solely on temperature appears to be insufficient and is, at best, limited. Other factors need to be included in view of the analysis described in this paper particularly with respect to the risk of cracking involved, the measures to reduce it, or the construction related thresholds associated with them.

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Appendix A

Analysis Results

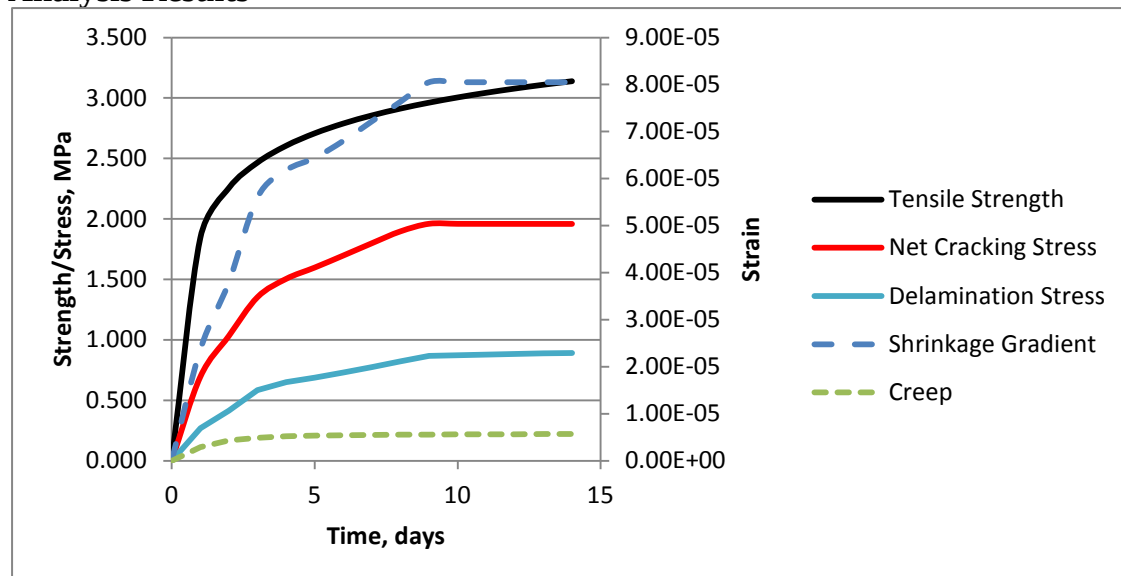


Figure A.1 Summary of Curling and Warping Related Stresses for Mar 5-7th Paving – Before Delamination.

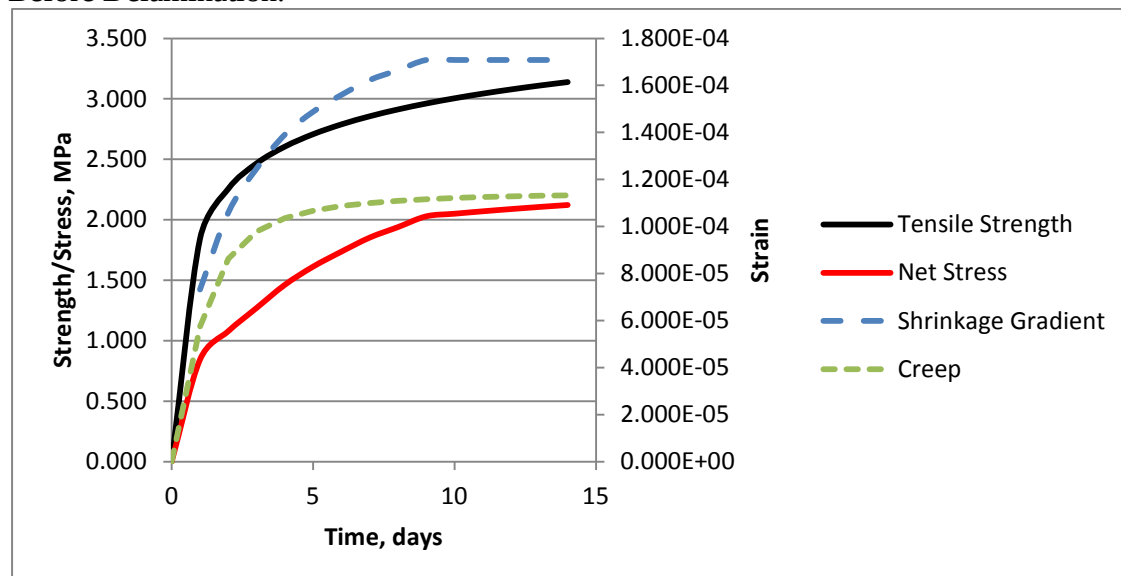


Figure A.2 Summary of Curling and Warping Cracking Stresses for Mar 5-7th Paving – After Delamination.

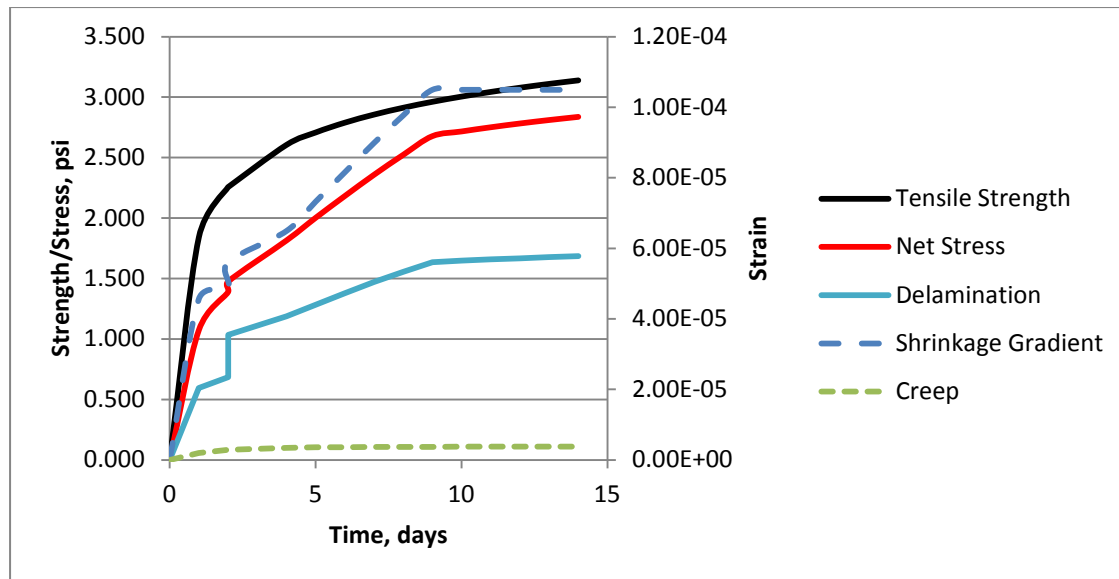


Figure A.3 Summary of Curling and Warping Related Stresses for July 18th Paving – Before Delamination.

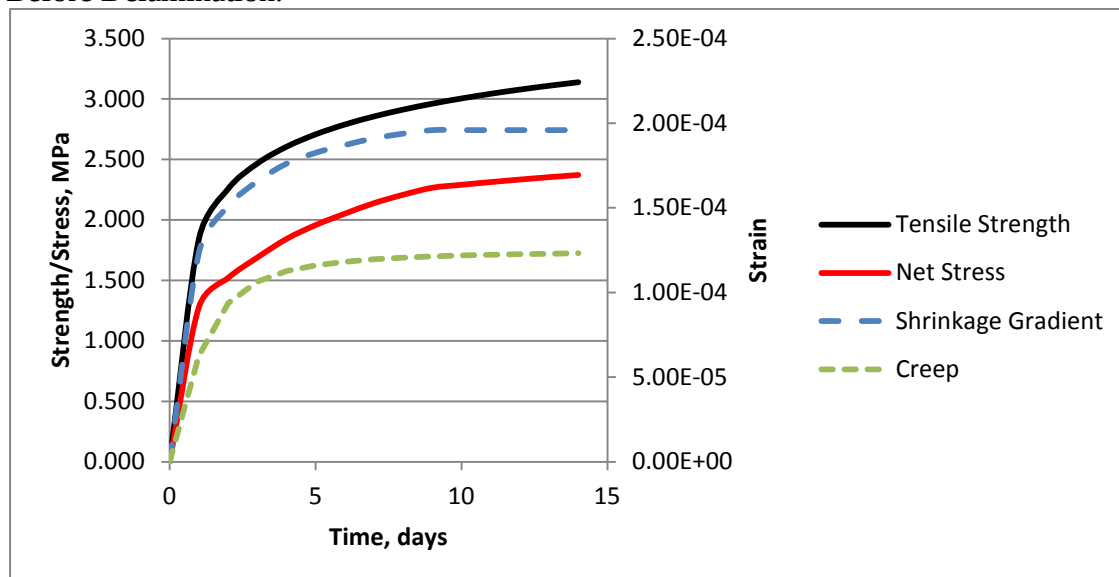


Figure A.4 Summary of Curling and Warping Cracking Stresses for July 18th Paving – After Delamination.

Appendix B

Curing Monitoring System

A square chamber assembly or curing plate with two chambers is placed on a concrete pavement surface or disc specimen (Figure B.1) and sealed using commercial silicon adhesive to prevent loss of vapor pressure during measures of relative humidity (RH). The chamber on the left side in Figure L.1 is the filtered chamber and provides the means to measure humidity just below a membrane formed by a curing compound but above the hardening concrete; the right chamber allows for measurement of RH under sealed conditions. The filtered chamber creates an environment that closely mimics the situation immediately below the curing compound in the field. The filtering can be created in a number of ways using filter paper or even placing a thin mortar layer to support the curing compound that creates a ‘membrane’ effect to suppress the passage of vapor from the concrete to the surrounding environment. Curing compound is applied to the filtering surface after casting at designated rates but

typically 1 gallon/180 ft². If in a laboratory setting, the mortar disc and the humidity chamber can be placed on a scale to measure moisture loss inside an environmental chamber maintained at 104 ±5 °F, 30% relative humidity, and 10 mph wind speed.



Figure B.1 Profile View of ACMM System and Curing Plate.

The curing compound is applied both directly to the surface of the concrete in the filtered chamber and on a mortar filter since some curing mechanisms depend on being in direct contact with the hardening concrete. Obviously, some large differences in RH are expected between the RH in sealed chamber and the RH of the filtered chamber. Nonetheless, these differences could be considered in characterizing the effects attributable to the effectiveness of a curing compound.

Laboratory Monitored Curing and Shrinkage Data

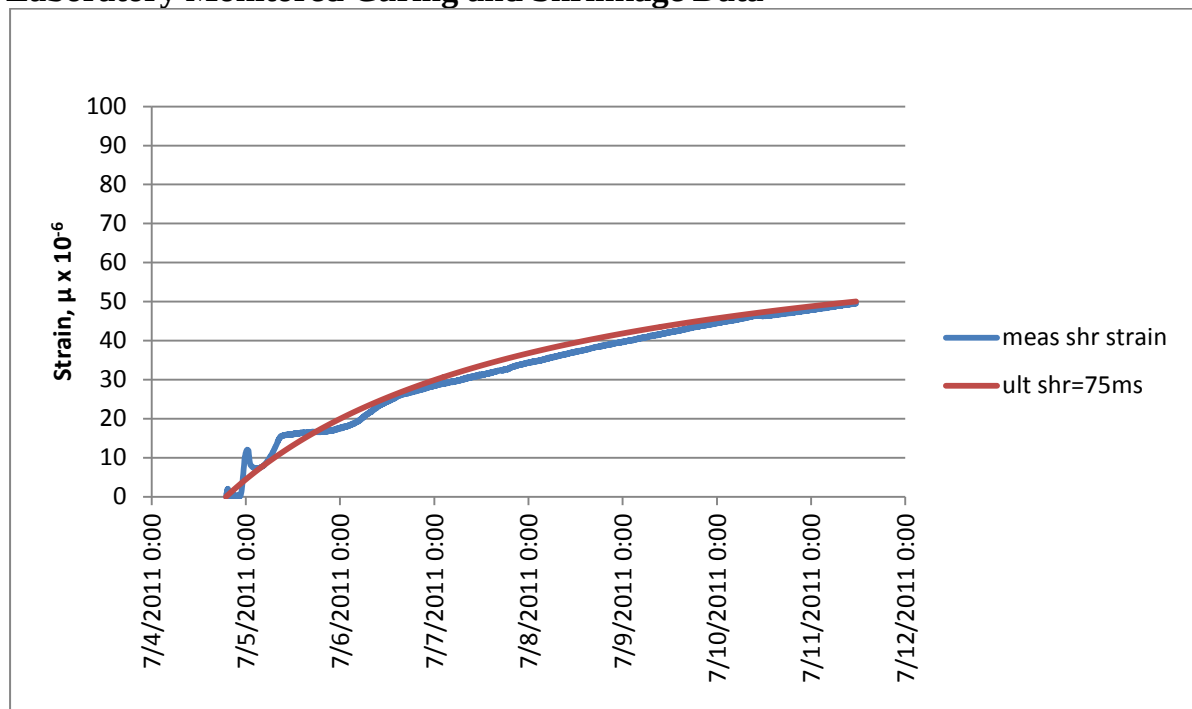


Figure B.2 Shrinkage Strain Data for Lithium Relay (1g/75sf).

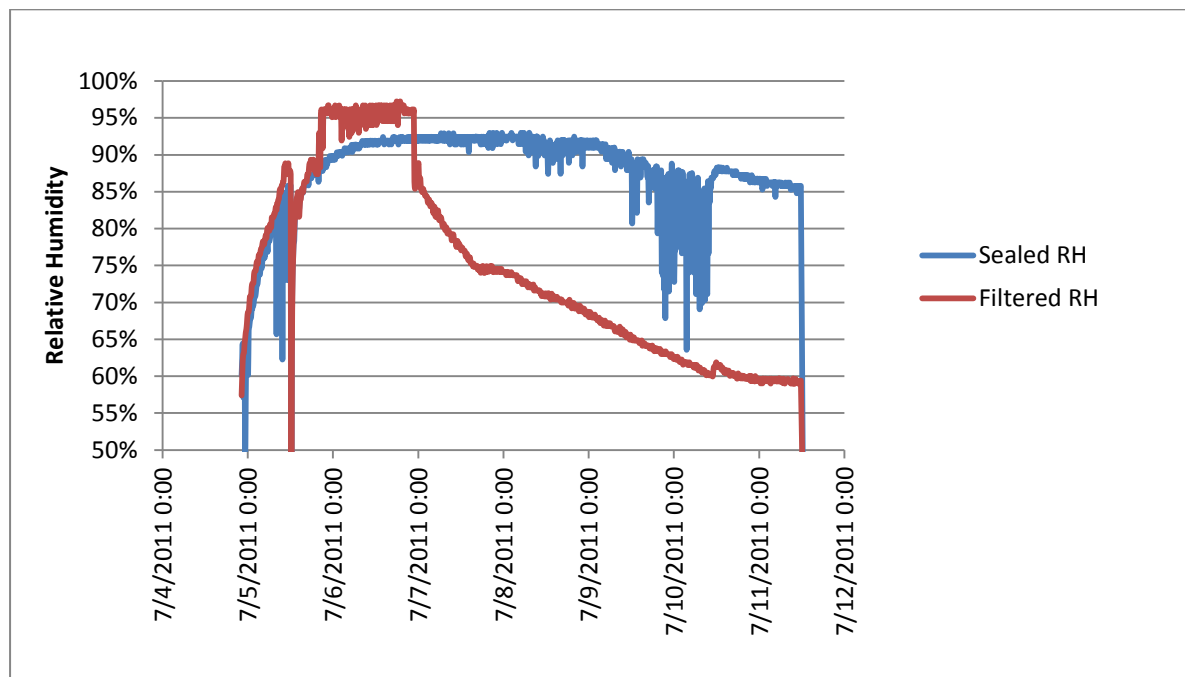


Figure B.3 Curing Monitoring Data for Lithium Relay (1g/75sf). 5