Development of Safe Construction Temperature ranges to avoid blow-ups in Ultra Thin Concrete Pavements

J.D. Mentz: Engineer, Aurecon, Pretoria, South Africa
A.M. Hartman: Technical director, Aurecon, Pretoria, South Africa

***Abstract*—After the construction of an ultra thin concrete pavement, ambient temperatures may induce an axial force within the pavement due to thermal expansion that can lead to the formation of a blow-up failure. By analyzing the mechanism of a blow-up, temperature differentiates at which a blow-up failure in ultra thin concrete pavements may occur were identified. Safe construction temperature ranges were developed for different ultra thin concrete pavements to serve as a guideline that would potentially limit the occurrence of blow-ups.**

***Keywords*— Blow-up, buckling, thermal expansion, ultra thin concrete pavement**

INTRODUCTION

The South African road network is an ageing part of the country's infrastructure. With 76% of the network exceeding the 20year design life, innovative repair and construction methods were required to rectify this problem [1].

Ultra Thin Continuously Reinforced Concrete Pavements (UTCRCP) were developed as one such solution. The most significant difference between UTCRCP and conventional Continuously Reinforced Concrete Pavements (CRCP) is the thickness of the pavement structure. While CRCP typically varies between 150mm up to 300mm, UTCRCP is only 50mm thick. In addition, UTCRCP utilizes concrete with a very high strength (exceeding 80MPa compressive strength) and a high percentage of steel (usually 50x50x5.6mm steel mesh) [2].

During the early development of UTCRCP, the potential of using thin concrete in low volume roads were identified and this lead to the development of Ultra Thin Reinforced Concrete Pavements (UTRCP). UTCRP differ from UTCRCP in that a lower volume of steel (typically 100x100x4mm steel mesh) and lower strength concrete (typically 30MPa compressive strength) is used [3].

The failure modes of ultra thin concrete pavements were interrogated during initial Heavy Vehicle Simulator (HVS) testing [4]. Unfortunately, the trial strips were generally short, did not always incorporate anchor end stops and failure modes were mainly load associated cracking and potholing. These conditions did not favour thermal expansion related failures and accordingly it did not receive significant research attention. The focus of this investigation is blow-ups, also referred to as pop-ups, as typically shown in Figure 1.There is general consensus that blow-ups are caused by axial compressive forces within the pavement structure. The axial forces are induced into the pavement through a rise in temperature or moisture changes [5].

Figure 1: Typical example of a buckling blow-up failure in an ultra thin concrete pavement.

The blow-ups typically occur at transverse cracks or joints in the pavements. The thermal expansion of the pavement is critical to activate this mechanism. Thermal expansion is dependent not only on temperature of the pavement, but also the moisture within the pavement. The increase or decrease of the moisture within the pavement may be represented by an equivalent increase or decrease of temperature [6]. This is due to the thermal expansion coefficient increasing or decreasing with a corresponding increase or decrease in moisture [7].

The use of safe temperature construction limits is well known in the road construction industry, with authorities restricting the construction of bituminous seals to warmer periods of the year. The calculation of safe construction temperature ranges to prevent thermal buckling failure is primarily used in rail construction [7]. It is of crucial importance for Continuously Welded Rails (CWR) to ensure that excessive tension and compression forces are not present in the rail to avoid either rail breaks or kick-outs. This investigation attempts to establish such temperature ranges for the construction of ultra thin concrete pavements.

Background Theory

The development of safe temperature construction range graphs for ultra thin concrete pavements, both UTRCP and UTCRCP, may reduce the occurrence of blow-ups within the pavement. If a pavement is constructed and adheres to the temperature limits prescribed, it should ensure that no excessive strain is developed in cold weather or excessive axial forces are induced by warm weather.

The blow-ups of CRCP pavements have been extensively researched [8], [9], [10], [11], [12], [13], [14], [15], [16] and [17]. The analysis of the phenomenon focus on the axial force induced into the concrete pavement by a rise or fall in temperature and/or moisture. Research conducted by Kerr [6], [5] and [18] identified three modes (cases) of buckling of a concrete pavement as indicated in Figure 2:

- continuous pavement with no joint, also referred to as unhinged;
- continuous pavement with a joint, hinged;
- continuous pavement adjoining a rigid structure.

(a) Case (I)

(b) Case (II)

(c) Case (III)

Figure 2: The three modes investigated by Kerr [18]

Kerr confirmed that the most critical case is where a joint or a large spoiling crack is located in a continuous pavement (Case II). As continuously reinforced concrete pavements are designed without joints but with regular narrow (less than 0.5mm in width) transverse cracks, a load case somewhere between an unhinged (Case I) and hinged (Case II) is applicable for the current study.

The use of conventional beam-buckling theorem, to determine the force at which the concrete pavement will buckle, is considered as erroneous. This is because beam-buckling does not account for the resistance to buckling provided by the self-weight of the pavement that influences the solutions significantly [18].

The analysis of Kerr represented the concrete pavement as a rectangular beam on a rigid base as indicated in Figure 3. The axial force induced by a temperature increase is represented by N_t, the reduced axial force after a blow-up is represented by $\tilde{N}_t$ in the lift-off region of 2l. In the adjoining regions, the axial force, N, varies due to the shearing resistance to axial displacements. The joint at x is where the largest lift-off may be expected after a blow-up.

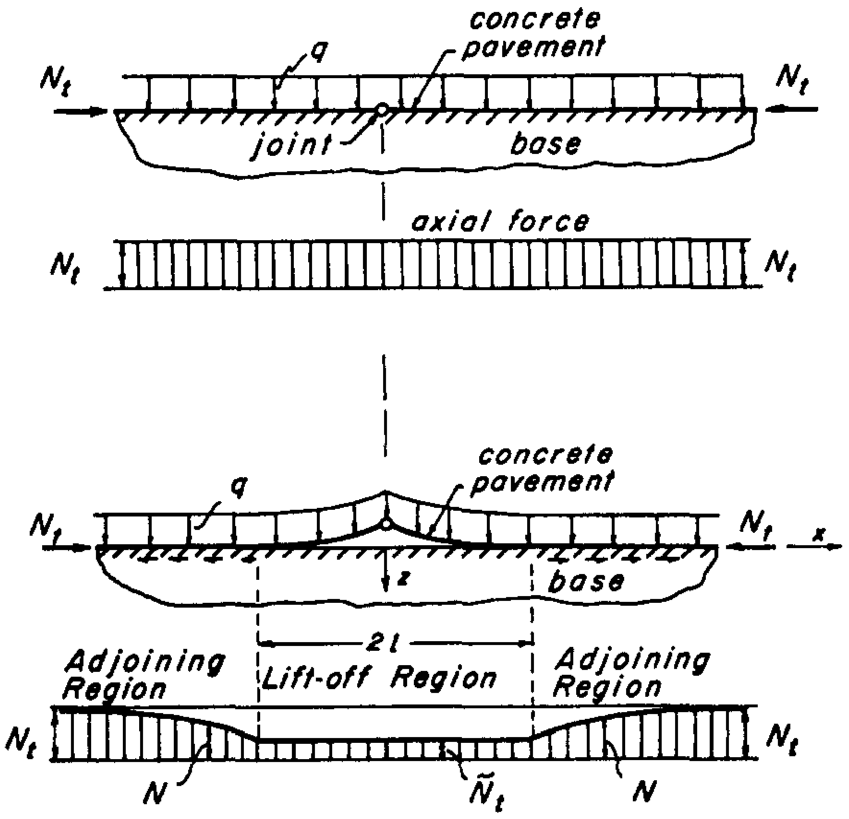


Figure 3: Axial forces in a concrete pavement after a blow-up [18]

The temperature at which a blow-up could occur was determined by further analysis of this model. The maximum temperature without a blow-up occurring was presented by ΔT_L. The temperature increase in the pavement was represented by ΔT_0. Thus, for a range $0 < \Delta T_0 < \Delta T_L$, no blow-up in the concrete pavement is expected, but any instance at which $\Delta T_0 > \Delta T_L$, a blow-up is possible.

For ultra thin concrete pavements, the formulas used to represent the mode of failure for a concrete pavement without a joint/crack were the following [18]:

$$\lambda l = 4.4934 \tag{1}$$

$$\alpha \Delta T_0 - \frac{\tilde{N}_t}{EA} = \sqrt{\frac{2r_0}{\mu EA} ln\left(cosh\left\{\mu\left(\alpha \Delta T_0 - \frac{\tilde{N}_t}{EA}\right) l - J\right\}\right)} \tag{2}$$

Where

$$J = 0.001023 q^{*2} l^7 \tag{3}$$

$$\lambda = \sqrt{\frac{\tilde{N}_t(1-\upsilon^2)}{EI}} \tag{4}$$

$$q^* = \frac{q_0(1-\upsilon^2)}{EI} \tag{5}$$

With

- α = coefficient of linear thermal expansion for the concrete ($\alpha = 9 \times 10^{-6}$/°C)
- E = Young's modulus for the concrete (GPa), refer to Table 1
- A = cross sectional area of the pavement (b x h)
- r_0 = sliding frictional resistance at the interface of the pavement and the base per unit length and width of pavement
- μ = curve fitting parameter for the sliding frictional resistance to test data (μ = 1000/m)
- $\tilde{N}_t$= axial force in the pavement
- q^* = unit weight of the pavement per meter length
- γ = unit weight of the pavement material (γ = 23.6 kN/m³)
- l = half the length of the lift-off region of the pavement since the uplift is symmetric

The Young's Modulus of the concrete differs for UTRCP and UTCRCP due to the difference in compressive strength of the concrete used for each. The value of the Young's Modulus is not the same for all the design codes. For the analysis the values given by Euro code [19] was used.

Table 1: Concrete modulus of elasticity

Cube Strength (MPa)		**Young's Modulus (GPa)**			
		Euro code [19]	SANS 10100 [20]	BS 8110-1:1997 [21]	ACI 318M-05 [22]
UTRCP values	25	30	26	21	21
	30	31	28	23	23
	35	32	29.5	24	25
UTCRCP values	70	41	-	36	36
	80	42	-	39	38
	90	44	-	43	41

The expected vertical displacement of the pavement can be calculated by using the equation (6). For the maximum deflection of the pavement, x is equal to zero since the maximum deflection occurs in the middle of the lift-off region.

$$w_1(x) = \frac{q^*}{2\lambda^4}\left[\frac{2\lambda l(cos\lambda x - cos\lambda l)}{sin\lambda l} + (\lambda x)^2 - (\lambda l)^2\right] \tag{6}$$

With the above equations, a positive value is assumed for the axial force within the pavement. With the now known axial force, lambda (λ) can be calculated along with l (length), the term J and a solution can be found for ΔT_0 through equation (2). From the solution of the equations over a multitude of axial forces, graphs can be developed for a concrete pavement without a joint.

The formulas used to represent the mode of failure for a concrete pavement with a joint/crack were the following, with equation (2), (4) and (5) unchanged:

$$\lambda l = 2.3311 \tag{7}$$

$$J = 0.0063 q^{*2} l^7 \tag{8}$$

$$w_1(x) = \frac{q^*}{2\lambda^4}\left[cos\lambda x + \frac{sin\lambda l - \lambda l}{cos\lambda l} sin\lambda x - 1 + \frac{(\lambda x)^2}{2} - \frac{(\lambda l)^2}{2}\right] \tag{9}$$

For each of the modes of failures, a minimum temperature increase to cause lift-off, i.e. a blow-up, can be determined for different thicknesses.

DETERMINATION OF BLOW-UP FAILURE TEMPERATURES

The relationship between temperature and vertical displacement has a parabolic shape representing the post-buckling equilibrium displacements as indicated in Figure 4 and Figure 5. The equilibrium branch to the left of the minimum value represents the unstable mode of failure that may occur described as a shattering blow-up. The equilibrium branch to the right represents the stable failure mode or a buckling blow-up. Shattering type blow-ups are more likely to occur in UTRCP pavements due to the lower compressive strength concrete used, with buckling blow-ups more common in UTCRCP pavements. Figure 6 and Figure 7 indicate the relationship between the axial force and vertical displacement and confirm high axial compressive forces with minimal up lift for shattering blow-ups.

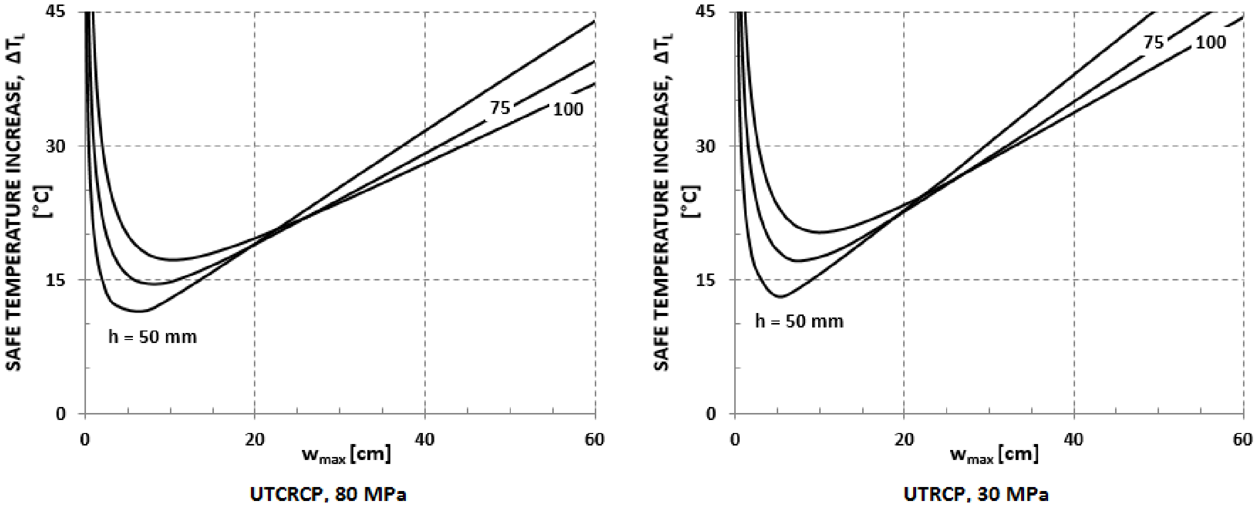


Figure 4: Equilibrium branches for UTCRCP and UTRCP with a joint/crack

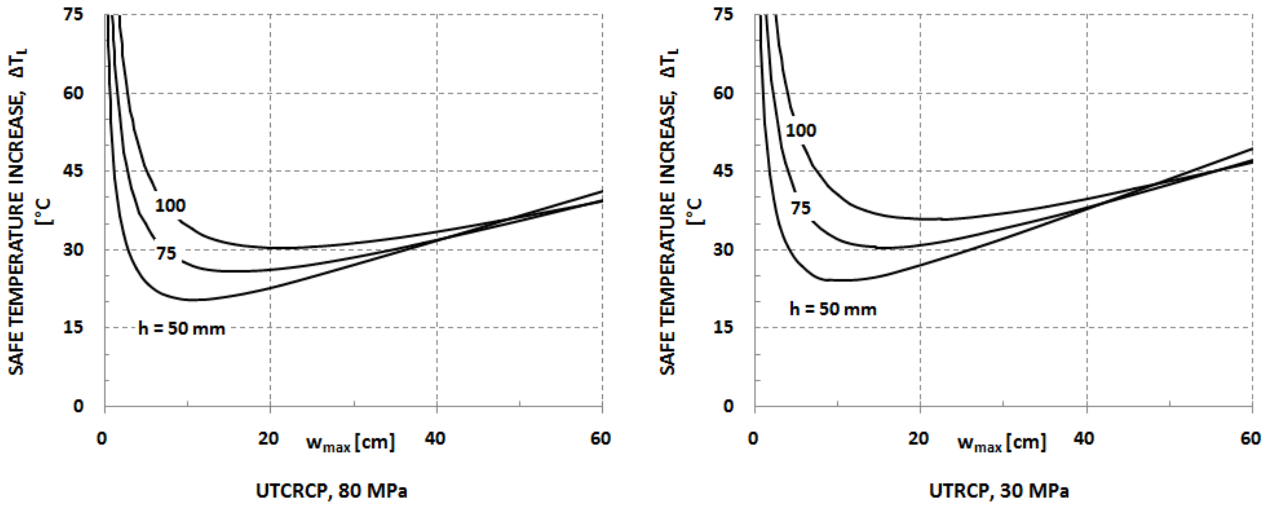


Figure 5: Equilibrium branches for UTCRCP and UTRCP without a joint/crack

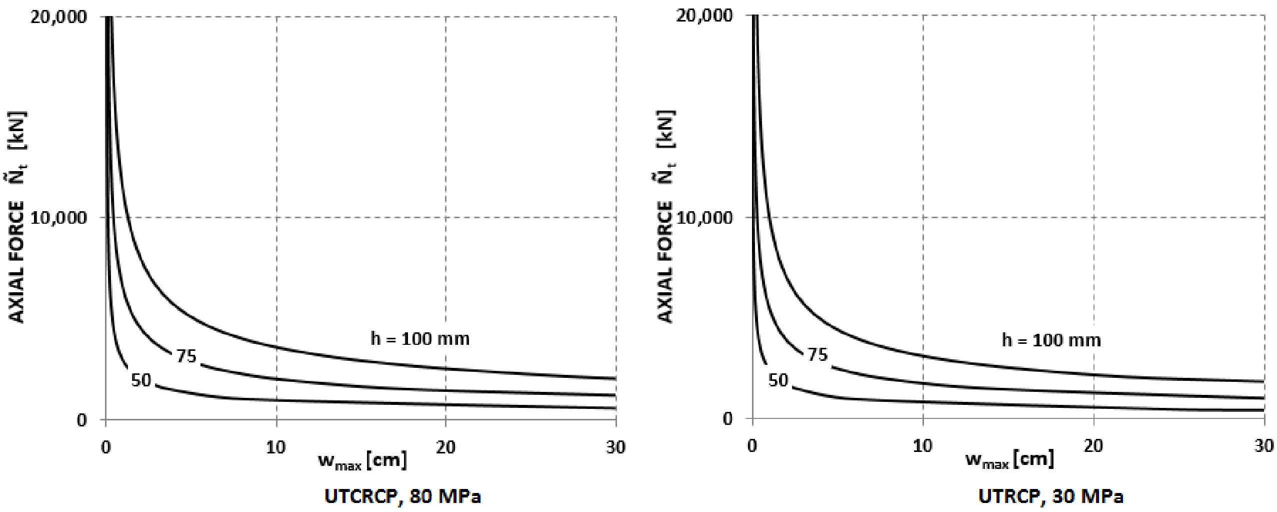


Figure 6: Axial forces, Ñt, for UTCRCP and UTRCP with a joint/crack

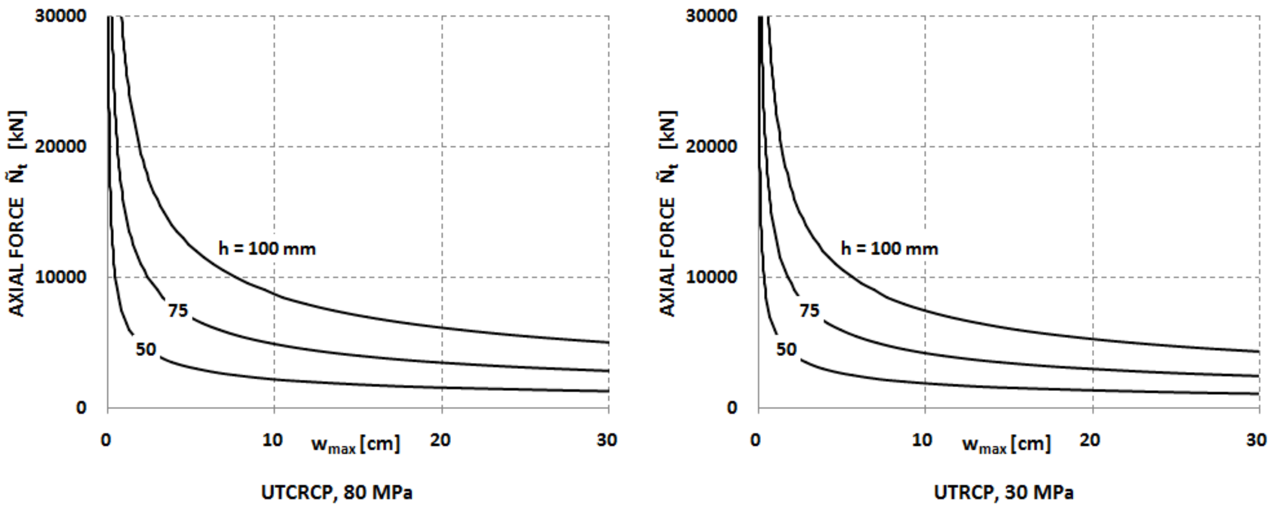
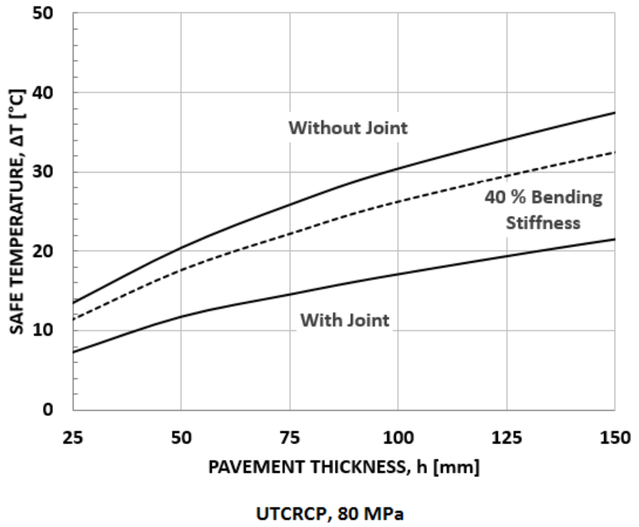


Figure 7: Axial forces, Ñt, for UTCRCP and UTRCP without a joint/crack

The relationship between concrete thickness and the maximum safe allowable temperature increase that the pavement can experience before a blow-up is indicated in Figure 8. It is clear that the temperature increase required to cause a blow-up in an ultra thin concrete pavement is considerably lower than for conventional CRCP pavements. The temperature increase required for a blow-up in ultra thin concrete pavements with a thickness of 50 mm, can be less than 15°C as indicated above. It is therefore of importance for the designer of an ultra thin concrete pavement to take into consideration the minimum temperature increase required to cause a blow-up and limit the construction temperature accordingly.

The deterioration of cracks in an ultra thin concrete pavement may further increase the risk of a blow-up at even lower temperatures than that of a newly constructed (without a joint/crack) pavement. Transverse cracks in an ultra thin concrete pavement occur and reduce the bending stiffness of the concrete, depending on the depth of the crack. This will cause the temperature increase for a blow-up to occur, to lie between the above-mentioned values with and without a joint. The effect of 40% reduced bending stiffness due to a possible crack in the pavement is also shown in Figure 8.



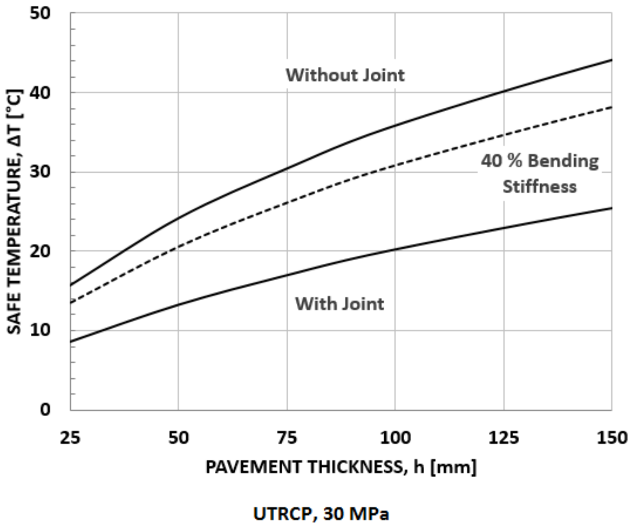


Figure 8: Pavement thickness influence on safe temperature increase

Safe Temperature Ranges

In rail track design, more specifically the design of CWR, the rails need to be fixed and destressed within certain temperature ranges. This will ensure that tension and compression forces within the rail are limited to reduce the risk of buckling failures.

With ultra thin concrete pavements being prone to temperature related failures, the need to identify possible safe temperature ranges for casting of the concrete was recognised. The lower temperature limit is dependent on axial thermal expansion forces within the pavement while the upper temperature limit is dependent on concrete shrinkage strain.

The main factor influencing the upper temperature limit is strain within the concrete. If the concrete pavement exceeds the allowed strain in thermal contraction, the pavement may not be able to recover the strain and therefore create a permanent defect [23]. In South Africa [24], hot weather concreting at temperatures above 35°C is usually not permitted as water evaporation and early age cracking becomes a problem [25].

To determine the lower temperature limit, the expected maximum temperature of the pavement needs to be known. While asphalt pavement maximum temperatures are well documented in South Africa, limited information on concrete pavements temperatures are known. Ambient temperatures are generally slightly lower than that of the pavement surface due to concrete's thermal absorptivity from the sun's radiation [26]. Although a relationship between ambient temperature and surface temperature of concrete pavements has been established, ambient temperatures were assumed in this study.

If the maximum temperature that an ultra thin concrete pavement can experience on a specific site is assumed to be 40°C, as indicated by SCT 22 [27], the lower limit (LL) is determined as follows:

$$LL = MaxT - \Delta T_L \tag{11}$$

As shown earlier, for a 50mm thick ultra thin concrete pavement with a transverse crack (40% reduced stiffness) the value of ΔT_L is equal to 20.5°C for UTRCP and 17.5°C for UTCRCP. The lower construction temperature limit is thus equal to 19.5°C and 22.5°C respectively. The sensitivity of the pavement to temperature and the condition thereof is clear from this analysis.

Discussion

The specifications supplied by the Standard Specifications for Road and Bridge Works for State Road Authorities (COLTO), state that concrete may be placed at temperatures as low as 5°C [28] and was adopted for UTCRP [29]. Although such low temperatures may be expected on site and even lower, to construct the ultra thin concrete pavement at such temperatures may lead to significant blow-ups occurring within the pavement.

Following the analysis above, a safe construction temperature graph for an ultra thin concrete pavement can be produced for different possible site temperatures and a range of possible pavement construction thicknesses to establish a guide for minimum construction temperatures. The use of a reduced bending stiffness of 40% was applied to represent possible imperfections and cracks that may occur within the pavement and considered appropriately conservative for design. Table 2 and Table 3 indicates minimum construction temperatures for a range of concrete pavement thicknesses for UTRCP and UTCRCP respectively.

Table 2: Minimum construction temperatures of UTRCP associated with the maximum expected site temperature and pavement thickness

Pavement thickness [mm]	Maximum expected site temperature [°C]					
	24	28	32	36	40	44
50	5	8	12	16	20	24
60	5	5	9	13	17	21
70	5	5	7	11	15	19
80	5	5	5	9	13	17
90	5	5	5	7	11	15
100	5	5	5	5	9	13

Table 3: Minimum construction temperatures of UTCRCP associated with the maximum expected site temperature and pavement thickness

Pavement thickness [mm]	Maximum expected site temperature [°C]					
	24	28	32	36	40	44
50	6	10	14	18	22	26
60	5	9	13	17	21	25
70	5	7	11	15	19	23
80	5	5	9	13	17	21
90	5	5	8	12	16	20
100	5	5	6	10	14	18

CONCLUSION AND RECOMMENTATIONS

With an increase in temperature, an axial force is induced within a concrete pavement. The axial force may then lead to a blow-up in the pavement where there is a vertical displacement of the pavement over a certain length. The magnitude of the temperature increase, consequent induced axial force and magnitude of displacement was calculated for ultra thin concrete pavements.

An interim guideline was developed that will allow designers in future to specify the minimum safe construction temperature for UTCRCP and UTRCP pavements for specific site temperatures. The safe temperature range can also be referred to as a neutral temperature range where the pavement is not in danger of experiencing excessive temperature induced axial forces or excessive strain with a temperature decrease. The lowest allowable construction temperature as stated by COLTO [28] is 5°C, but to construct ultra thin pavements at this temperature could be detrimental and may lead to blow-up failures.

The temperatures at which blow-ups occur need to be confirmed by laboratory and/or full-scale testing to substantiate the analysis done. The validation of the calculated low construction temperatures is the focus of a study that is currently being undertaken by the authors on a number of recent blow-up failure sites.

References

[1] South African Institution of Civil Engineers, 2017. 2017 Infrastructure Report Card for South Africa. SAICE, Midrand, South Africa.

[2] Mukandila, M.W.K.E., Milne, T.I. & Horak, E., 2009. Constructability aspects of Ultra thin continuously reinforced concrete pavement. 28th Southern African Transport Conference, (July), pp.234–244.

[3] Brink, H. & Road, M.N., 2012. Ultra Thin Reinforced Concrete Pavement: Practical Applications. (July), pp.168–175.

[4] Kannemeyer, L. et al., 2007. Ultra Thin Continuously Reinforced Concrete Pavement Research in South Africa. International Conference on Concrete Roads, p.27.

[5] Kerr, A.D., 1994. Blow-up of a concrete pavement adjoining a rigid structure. International Journal of Non-Linear Mechanics, 29(3), pp.387–396.

[6] Kerr, A.D., Shade, P.J. & Park, F., 1984. Analysis of Concrete Pavement Blow-ups. Acta Mechanica, 52, p.201-224.

[7] US Army Corps of Engineers, 2000. Technical Instructions Railroad Design and Rehabilitation.

[8] Woods, K.B., Sweet, H.S. & Shelburne, T.E., 1946. Pavement blow-ups correlated with source of coarse aggregate. Highway Research Board Proceedings, 25.

[9] Hveem, F.N., 1958. Types and causes of failure in highway pavements. Highway Research Board Bulletin (187).

[10] Hensley, M.J., 1966. The study of pavement blow-ups, Little Rock, Arkansas.

[11] Bowers, D., 1966. A study of the failures occurring in the concrete pavement of the Connecticut Turnpike and roads of similar design, [Wethersfield Conn.]: Connecticut State Highway Dept. Division of Research and Development.

[12] Graham, M. & Chamberlin, W., 1967. Summary of 1966 rigid pavement blow-up survey. Engineering Research and Development Bureau, New York..

[13] Stott, J.P., & Brook, K.M., 1968. Report on a visit to USA to study blow-ups in concrete roads. Road Research Laboratory, British Ministry of Transport.

[14] Gordinier, D.E. & Chamberlin, W.P., Pressure relief joints for rigid pavements. Albany, New York.

[15] Burke, M.P., 1972. The world’s most expensive pavement expansion joints. In Ohio Highway Engineering Conference Proceedings.

[16] Foxworthy, P.T., 1973. State wide survey of blow-ups in resurfaced concrete pavements. West Lafayette, Indiana: Purdue University.

[17] Gress, D., 1974. Blow-ups on resurfaced concrete pavements. Joint Highway Research Project, West Lafayettte, Indiana.

[18] Kerr, A.D., 1997. Assessment of Concrete Pavement Blow-ups. Journal of Transportation Engineering, 123(2), pp.123–131.

[19] Uni En, 2004. BS EN 1992-1-1:2004 - Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings. Eurocode 2, 1(2004).

[20] South African Bureau of Standards, 2000. The Structural Use of Concrete - Part 1: Design. SABS 10100-1.

[21] British Standard, 1997. Structural use of concrete - Part 1. Code of practice for design and construction.

[22] ACI Committee, 2010. Report on High-Strength Concrete (ACI 363R-10).

[23] Alexander, M. & Beushausen, H., 2009. Deformation and volume change of hardened concrete. In: Owens, G. (ed.), Fulton’s Concrete Technology, Ninth, Midrand: Cement and Concrete Institute, p.111–154.

[24] South African Bureau of Standards, 2012. Ready-mixed concrete SANS 878:2012.

[25] Rasmussen, R., Rogers, R. & Ferragut, T., 2009. Continuously Reinforced Concrete Pavement: Design and Construction Guidelines. FHWA/CRSI Final Report, Washington, DC.

[26] Van den Bossche, J.M., 2001. The measured response of ultra-thin and thin white topping to environmental loads. 7th International Conference on Concrete Pavements. The Use of Concrete in Developing Long-Lasting Pavement Solutions for the 21st Century, 2001, Orlando, Florida, p.807–823.

[27] Cement and Concrete Institute, 2011. SCT 22 Concrete Road Design and Construction, Cement and Concrete Institute.

[28] Committee of Land Transport Officials, 1998. Standard Specifications for Road and Bridge Works for State Road Authorities.

[29] CSIR, 2008. Guidelines for the construction of a 50mm thick Ultra Thin Reinforced Concrete Pavement (50mm UTRCP). Version 1. Council for Scientific and Industrial Research.