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Mitigating Concrete Pavement Cracking

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Abstract

One of the main concerns about concrete pavement, mainly the industrial one, also called industrial floor, is the cracking issue. This could be caused by structural aspects or by the concrete shrinkage. The latter is not usual and when it occurs, it is easily identified and frequently associated with other problems. However, the cracking caused by shrinkage are more frequent and could represent more than 95% of the total observed.

Although frequent, the cracking causes are related only to ordinary hydraulic shrinkage; this kind of cracking is more complex and could be separated in: initial and final hydraulic shrinkage, autogenous or even thermal shrinkage.

The shrinkage is connected to the concrete materials - cement, aggregates, admixtures and water - and their relative quantities. To face the cracking issue, we must know the influence of each one on it. In this paper, our goal is to focus on the crack formation's mechanisms.

The cracking control is also related to the ways you could come to control it, like choosing more adequate additions and admixtures, the concrete mix, water content and also employing some kind of materials into concrete to minimize the shrinkage. The concrete warping and the stresses related to it will also be discussed.

Keywords: concrete pavement, slab on ground, drying shrinkage, plastic shrinkage, autogenous shrinkage, warping, curling, thermal coefficient, concrete, pavement design, bonded overlay.

1 Introduction

Concrete pavement is considered the most durable and lowest cost alternative for roads, streets and industrial floors, but has showed some types of pathologies that have caused insecurity among investors and dissatisfaction among users.

The main pathology is the non-structural cracking, caused by secondary stress. The secondary stress comes from the environment actions, like temperature variation or concrete humidity exchange, that occurs during pavement's life which become more evident during the first year.

In this work, emphasis shall be given to the stresses to which the concrete is submitted by drying shrinkage and thermal movements, by mainly considering differential movements of the concrete slab and, in some cases, how such stresses may be increased by the simultaneous action of static or dynamic loads.

Oftentimes such stresses wind up being involuntarily incremented by some actions of structural engineers or contractors, who had the initial objective of improving the performance of the pavement, but ended up reducing its useful life.

One example is the use of mechanical resistances that are too high, which end up worsening warping stresses; under the item for concrete, the care that must be taken regarding concrete specification shall be addressed, by suggesting upper and lower limits of some inputs used in its confection.

Finally, adhered overlays shall also be discussed, which have systematically presented smaller pathologies due to concrete shrinkage in the contact surface, which results from the complexity of the system.

2 Drying and Autogenous Concrete Shrinkage

Concrete technologists are frequently taken by surprise by questions such as: does the concrete today present greater shrinkage? Why do so many cracks appear? Are the causes of cracks simply the result of inadequate construction techniques?

The first question can be easily answered, since it is widely documented that changes that take place during cement manufacturing, such as fineness and levels of addition, as well as the difficulty in obtaining good quality aggregate, especially fines, has resulted in significant increases in concrete shrinkage (Burrows, 1988) basically caused by the increase of water in the mixture, reduction in internal shrinkage restriction that the concrete presents, etc.

Even more important is that autogenous shrinkage, which only a few decades ago almost did not have a practical effect on construction works, is currently considered to be an important overall shrinkage component of concrete, especially in those with active hydraulic additions and in concrete with a low water-cement ratio. There is also the aggravation of employing concretes with a high mechanical resistance in pavement, following the trend of high performance concretes used in structures, or even exaggerated specifications on part of the normative entities or of opinion formers, such as the mistaken impositions of elevated cement contents.

The second issue is more complex, but is the main objective of this work, since it also deals with the association of shrinkage stresses with those of loading. As to the latter, currently the execution ends up being blamed even though the failure is not necessarily due to it.

In order to better understand, concrete shrinkage can be divided into two phases: that which occurs during the initial stages, 24 hours after pouring the concrete, called initial shrinkage, and that which takes place at later ages, which is complementary shrinkage.

In principle, during initial shrinkage, plastic, autogenous and hydraulic shrinkage may occur simultaneously or isolatedly, where the last two effectively promote the shortening of the concrete. In complementary, it is possible that hydraulic and autogenous shrinkage occurs, where the latter depends on certain factors in order for it to take place.

2.1 Plastic Shrinkage

Plastic shrinkage occurs very prematurely when the bleeding rate is less than the evaporation of water from the concrete, thereby promoting a dissection of the upper pavement layer, which depth will depend on the atmospheric conditions and concrete characteristics, such as the water/cement ratio, cement content and mainly the evaporation conditions. Environments with low relative humidity of the air, subject to winds and mixtures rich in cement are more subject to plastic shrinkage, followed by traditional short fissures that appear in groups that are parallel among themselves

Such fissures are caused by the water suction into the pores (pore with negative pressure) initially generated by the drying of the pores, but which is also influenced by the hydration process, which promotes an increase in the system rigidity, thereby reducing mobility. Generally, they have a large opening, measurable in millimeters, but the depth is a direct function of the dissection depth and of the concrete Poisson's ratio. Whenever such is close to 0.5 no fissures will occur, but the depth tends to coincide with the dissection depth once the Poisson's ratio is reduced.

Even though all the generating forces cease after the setting of the concrete, the fissure dimensions may evolve depending on the magnitude and direction of the strains present in the pavement (Morris & Dux, 2006).

Besides climatic variations, fissure control, which is not always possible of being avoided, may be done with the use of fibers. Wire mesh or conventional rebar, which are not efficient in preventing plastic cracking.

In general, any type of fiber capable of controlling the formation of plastic cracking, whether of plastic or steel and their needed quantity (volumetric fraction), will depend on the elasticity modulus of the material of which they are made, the aspect ratio of the fibers – diameter/length ratio, quantity of fibers per unit mass, surface properties, cross section and specific strain (Balaguru, 2001).

Fiber choice may also be made by considering the type of source being used. For concrete, longer fibers with larger diameter are more recommendable, while for grout, short fibers with smaller diameters present better performance.

Fiber performance evaluation is unfortunately a complex process. One of the difficulties that can be observed in various fiber performance researches is the type of test employed, since normalized ones, such as ASTM C157, are not adequate for determining shrinkage in early stages and in reality, each researcher ends up adopting a different procedure. Therefore, the tests have a comparative value but are not, in most cases, interchangeable (Rodrigues and Motardo, 2002).

Such tests have in common the use of controlled humidity, temperature and wind chambers and the sample is submitted to some type of restriction, such as an o-ring, adherence at the base simulating a bonded overlay, or other movement restraints. The currently consolidated conclusions are (Balaguru, 1994):

a) The addition of synthetic fibers, even at rates as low as 0.45kg/m³ promotes some reduction in the quantity of fissures;
b) More marked reductions are obtained with doses between 0.45kg/m³ and 0.90kg/m³;
c) Long fibers, those presenting a lower elasticity modulus, are the ones that offer the best performance;
d) For 0.9kg/m³ doses, both for nylon or polypropylene fibers, plastic shrinkage fissures were practically not observed in the experiments;
e) The amount of fibers – number of fibers per kilogram – is an important parameter for dosing;
f) Long fibers present better performance in leaner mortar and concrete, while micro-fibers present better results in richer mixtures
g) With synthetic fibers, the quantity of fissures is not only reduced, but their opening is smaller.

There is a consensus among various researchers that low modulus fibers – a characteristic of synthetic fibers – present better performance. For example, in order for the steel fibers to also act in the control of plastic fissures it is necessary to use a content three times greater than that which is used in structural purposes, which makes concrete mixing become very difficult.

Plastic fibers, in dosages used to control plastic shrinkage, varying between 0.6 kg/m³ and 0.9 kg/m³, do not aggregate a structural value, since the tenacity observed in the concrete is negligible. However, it is believed that they are still efficient in controlling fissures at earlier stages, less than 24 hours. Recently, high diameter fibers have been used, mainly of polypropylene and PVA, made with larger diameters and that allow elevated dosages between 2 and 5kg/m³. In these cases, it is possible to obtain a structural increment in the pavement, with tenacity values varying between 15% and 25%, or even higher.

2.2 Drying shrinkage of concrete

The drying shrinkage is the classic shrinkage that occurs in concrete, oftentimes subdivided into initial, which occurs during the first 24 hours, and complementary.

Its occurrence is related with the loss of water that is not combined during cement hydration and from this, the first important control lesson may be obtained, which is the quantity of water added to the concrete and also the water/cement ratio that it presents, where the former is more significant, as shown in figure 1 (PCA, 2002).

Actually, the water in cement paste has four different forms (Metha and Monteiro, 2006): the chemically combined water, the interlayer water, the adsorbed water and the capillary water. The last one is responsible for drying shrinkage. The pore size can be higher than 50×10^{-6} mm or between 5 and 50×10^{-6} mm; the evaporation of the water of the last range causes shrinkage; the former one has no influence over drying shrinkage.

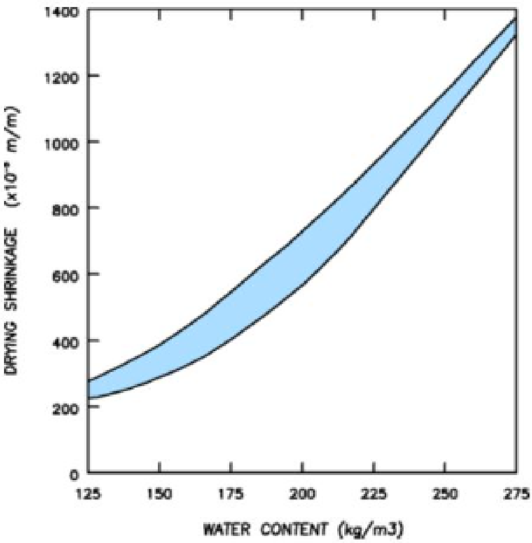

Figure 1: Influence of water content in drying shrinkage (PCA, 2002)

Influence of cement

Since the cement paste is responsible for the shrinkage, it becomes evident that its quantity directly influences this property. For this reason, cement consumption should be limited, since elevated consumption, besides increasing shrinkage, will make the concrete more susceptible to thermal variations mainly in the presence of moisture.

Specifically regarding cement, fineness seems to be an important factor since the increase of cement specific area will not only imply in greater water consumption for wetting the particles, but also in the restrictive effect that the coarser particles may promote.

Some decades ago, when grinding systems were open, granular curve of cement was continuous and it was possible to find cements having specific areas of around 3000 cm^2/g and a detectible residue on the 0.075mm sieve varying between 2 to 5%. With the advent of closed grinding systems in which coarse particles are sent back to grinding, started presenting narrower granular curves, tending towards the finer part of the initial curve.

Currently, grinding systems are operated to obtain maximum mechanical resistance and quick develop, but emphasis has not been given to the problem related to paste shrinkage.

In broader granular curves, the reaction velocity is lower, which maintains a lower elasticity modulus of the paste during initial stages, thereby allowing for a relief of the shrinkage stress promoted by creep. Besides this, the larger cement particles would act as restriction elements, thereby reducing paste shrinkage (Burrows. 1998).

There is no clear correlation between the type of cement and the rate of addition with hydraulic shrinkage of concrete, where there are finer cements that promote less retraction than others, which apparently are less problematic. This subject was discussed in the ASTM (*American Standard Tests of Materials*) studies, which developed a relatively simple test (ASTM C1581 – 04: Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage), which proposes to assess the potential of concrete shrinkage, basically linked to chemical and micro-structural aspects of the paste.

In the test, circular specimens are molded with a rigid steel core, which is maintained during all the test period, promoting a restriction to the shrinkage. The test finished when the first crack occurred, and the elapsed time is recorded. The shrinkage potentiality is established in the table 1.

There is no knowledge about tests of this kind made with national cements. However, it is beyond doubt that this knowledge, associated with job site checks, could be a valuable tool for the correct evaluation of Brazilian cements that present expressive variations comparing to the North-American ones. Concerning to the pavements, only the cements with moderate-low or low shrinkage potentials should be used.

Elapsed time (days)	Shrinkage Potential
0 < t < 7	Hight
7 < t < 14	Medium Hight
14 < t < 28	Mediun Low
t > 28	Low

Table 1: Shrinkage Potential (ASTM, 2004)

The importance of the chemical composition of the cement in the shrinkage process can be found in the test developed by R. L. Blaine in the end of the 50's (Burrows, 1998). He used cement rings, with a central restriction, and showed that besides the high fineness, cements with high C_3A and alkalis content, are more likely to crack and they are more predominant than fineness.

Aggregates Influence

As already seen in the former item, the most of aggregates can be considered non-shrinkable, and therefore it becomes easy to figure out that the concrete shrinkage shall be proportional to the quantity of the aggregate. T.C. Powers presented the following expression to correlate the variables (Mehta and Monteiro, 2006):

$$\frac{S_c}{S_p} \qquad -g)^n 1(\qquad (2.1)$$

where S_c and S_p correspond respectively to the concrete and the cement paste shrinkage, and g to the aggregate fraction of the concrete. The exponent n refers to the modulus of elasticity of the aggregate, and could vary, according to L'Hermite, between 1.2 and 1.7 (Mehta and Monteiro, 2006).

Although there are other factors such as particle texture, paste – aggregate interface, which can influence on the concrete shrinkage, the modulus of elasticity of the aggregate prevails, as can be seen in figure 2 (Troxell, 1958). There, the aggregates with higher modulus of elasticity, such as granite and limestone, eventually show lower concrete shrinkage than the ones made with sandstone. It can also be seen that the difference increases significantly with aging or drying time of the concretes.

Admixtures Influence

Regarding admixtures, a distinction can be made according to function: water reducing admixtures and shrinkage reducing admixtures. The first group includes plasticizers and super-plasticizers, which are products that promote water reduction in the concrete. However, they should also reduce the concrete shrinkage as an additional benefit.

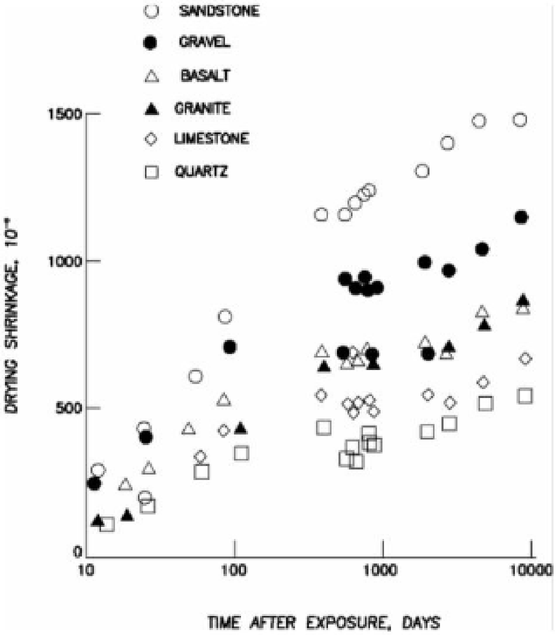

Figure 2: Influence of aggregate modulus of elasticity on shrinkage (Troxell, 1958)

Nevertheless, in the past, some plasticizers presented slightly greater shrinkage for the same water content, which nowadays has been overcome by super-plasticizers, a viable alternative when it becomes necessary to use more fluid mixtures without increasing the water content.

The second group, shrinkage reducer admixtures, is still not so widely used because of high cost in our country, although such admixtures are a powerful tool for shrinkage control. These products have a chemical action on the water, by changing the superficial water tension and providing reduction in drying shrinkage, which varies between 35% and 70% depending on the water/binder factor (Mokaren, 2002).

Pavement Thickness

Pavement thickness is an important factor in the development of drying shrinkage. The water loss occurs through the surface of the slab. Figure 3 shows this trend, since moisture diffusion in the concrete is low, mainly in more impermeable concretes.

This fact will be important to the concrete slab warping study, as seen in item 4, where the differential shrinkage, either from drying or thermal effect, generates high stresses that, when associated with loading, might lead to its structural failure.

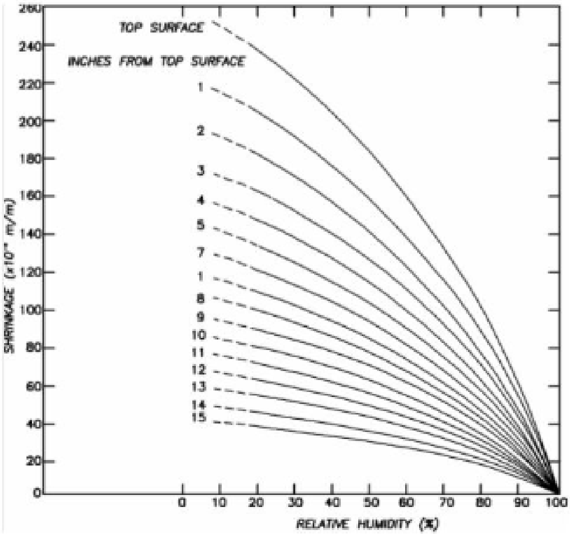

Figure 3: Influence of slab thickness on drying shrinkage (Suprenant, 2002)

Environmental influences

The concrete drying principle is phase equilibrium. It means that the concrete can exchange humidity with the air until it reaches the point of saturation. It is a function of the relative humidity of the air. Thus, when it is low, the concrete will dry faster. For example, in the Southwest of Brazil, concrete dries faster between May and August, while during the rainy season there is low humidity exchange in the concrete.

2.3 Autogenous shrinkage

The autogenous shrinkage happens without loss of mass in the concrete. This means that there is no water lost to the air. The cause of this is the shift of capillary water to adsorbed water in a spontaneous process (the free energy or Gibbs' energy of adsorbed water is smaller than the free energy in capillary water) during cement hydration

When water supply is abundant, autogenous shrinkage does not occur. However, when the water supply is limited, hydration occurs using capillary water, thereby causing concrete shrinkage. In light of this, concrete pavement that is chemically cured will be more subject to this type of shrinkage

One way of avoiding autogenous shrinkage is by using self-curing concrete, which is made with saturated porous aggregates that provide additional hydration water by gradually releasing internal supplementary water. Cellulose fibers, due to their high capacity of water absorption, can also play this role. Self-curing concrete also presents a little understood feature, which could have a valuable effect in shrinkage control of the concrete paste, since it allows the formation of a very well hydrated paste microstructure, which is less shrinkable than deficiently cured ones.

Since it is conditioned to cement hydration, autogenous shrinkage occurs faster in concretes cured under higher temperatures, following the maturity rule. Hendlund, et al. showed that, for a water/binder ration between 0.4 and 0.45, the autogenous shrinkage was three times higher than that of similar age, when the curing temperature changed from 20 °C to 50°C (Hedlund and Jonasson, 2001).

Concretes with water/cement [1] ratio below 0.42 present high autogenous shrinkage (Holt, 2001), and when the water/binder ratio is close to 0.3, this kind of shrinkage corresponds to 50% of the total (PCA, 2002). The hydraulic additions like fly-ash, blast slag furnace and silica fume result in higher values of autogenous shrinkage where such property is also closely related to fineness.

3. Thermal variations

When analyzing pavement projects, it can be observed that little attention has been given to concrete thermal variations in Brazil, essentially due to modest thermal gradients, which results in small variations in the slab length. Nevertheless, one should be aware of the differential variations in plate length caused not only by the thermal difference between the top and the base of the slab, but also by the variation of the concrete thermal expansion coefficient as a function of humidity.

Nevertheless, there are important issues about thermal variation that can be considered in the design, like the difference between thermal variation coefficient of aggregate and cement past, or the changes in concrete thermal coefficient due to concrete humidity. This condition can promote the warping slab without temperatures changes, because the humidity is not uniform in the slab's thickness.

The importance that the coefficient of thermal variation has on concrete, can be seen in figure 4 (NHRP, 2003), where there is an expressive variation of slabs cracking when the thermal coefficient changes.

In order to have an adequate evaluation over the thermal variations of concrete it is necessary understand the coefficient of dilation. As it deals with a compound material, this property shall depend on the individual coefficients of its components, separated in paste-hydrated cement, air and water – and the aggregates.

[1] Cement or cementicios materials

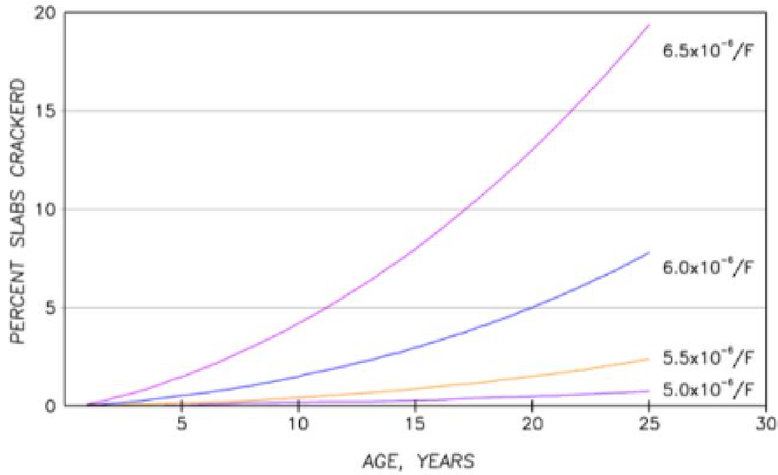

Figure 4: influence of thermal coefficient on road durability (NHRP, 2003)

The paste coefficient might vary in a relatively wide range, approximately between 10 and 22 x 10^{-6} / °C, depending on the chemical constitution and the cement fineness as much as the humidity, considering this the most relevant factor [2](Emanuel and Hulsey, 1977). The figure 5 shows this behavior.

This variable is particularly important to the pavements because the superficial concrete tends to be normally less expansive than the base, except when the base happens to be saturated. This effect collaborates with the slab warping. The analyses of tensions has to be done regarding not only the thermal gradient, but also the effective one, that can be up to 80% higher in concrete with 70% of relative humidity, if compared to the dried or even saturated one.

The age also influences the thermal coefficient: a new concrete has thermal coefficient bigger that an old concrete. It can be 30% smaller in concretes with over ten years. This fact should be considered, mainly in bonded overlays.

In order to have an adequate evaluation over the thermal variations of concrete it is necessary understand the coefficient of dilation. As it deals with a compound material, this property shall depend on the individual coefficients of its components, separated in paste-hydrated cement, air and water – and the aggregates.

[2] The thermal coefficient can be considered by the association of the true one, which is based on kinetic movement and the apparent thermal coefficient, which is caused by absorptive mass attraction forces and capillary stress (Emanuel and Hulsey, 1977).

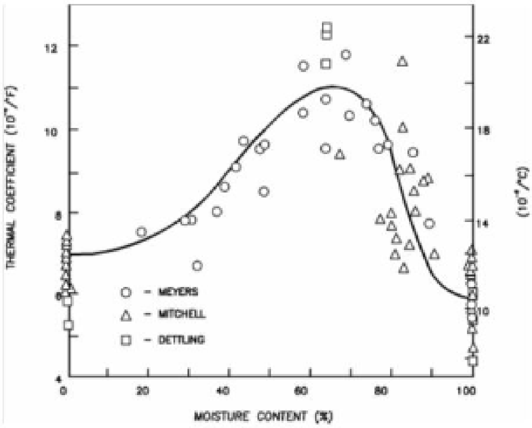

Figure 5: Variation of thermal coefficient of expansion of neat cement past with moisture content, combined plot w/c ratio between 0,12 to 0,4 an age up to 6 month (Emanuel and Hulsey,1977)

The aggregates, mainly due to their dosage in concrete, end up having an important influence on the expansion coefficient of this material. Among a wide range of rocks used, arenite is the most sensitive to temperature variations, with approximately 11 x 10^{-6} / °C, while granite and gneiss have a coefficient of 6 x 10^{-6} / °C. Limestone behave differently and may present values as low as 3 x 10^{-6} / °C, which in some cases may cause concrete durability problems due to the great difference from the cement paste.

4. Warping stress in concrete pavement

It can be affirmed that all concrete pavements warp, with no exceptions. In most cases, it can be easily identified by the deflection of the edges when under load, later returning to the original shape. At other times, this deformation is not so noticeable due to the slab weight, which maintains itself close to its original position.

For the time being, whether the warping is visible or not is not relevant. However, what must be checked is if the stresses that it has caused will have an influence on the life of the pavement. Such stresses are the result of thermo-hygrometric variations between the top and the base of the slab.

Even when there is no thermal gradient, warping can occur because of the temperatures at which the concrete is poured and that of the environment at the time of the pouring (Hansen, 2001). For instance, if concrete is poured at a temperature below ambient, after only a few hours, with the concrete still in a plastic state and therefore in perfect contact with the sub-base, there will be a temperature rise on the upper surface of the slab. This is due to the heat exchanged with the environment, which generates a positive thermal gradient called pavement construction gradient. When the ambient temperature drops, the slab becomes warped, even if the thermal gradient is equal to zero! This can give us an idea of the complexity of the problem.

The author can testify the importance of this issue: in a road construction made with regular concrete, with a maximum slab length of 5m, in slabs where the concrete pouring process occurred during the hottest hours of the day (between 11 a.m. and 3 p.m.), the cracking of sawed joints happened frequently. This anomaly can be attributed to the stresses caused by warping, even with a zero gradient.

Although warp studies began almost a century ago with Westergaard (Huang, 2004), still the stresses and deformations of this process are not fully known. Westergaard's (Huang, 2004) work was done with the intention of knowing the stresses involved in temperature variations. The numeric resolution for his equation was proposed by Breadbury (Breadbury, 1938) whose resolution model, based on figure 6, became widely known. This numeric resolution allowed calculating the warping stresses at any point of the slab.

$$\sigma = \frac{E \times \alpha \times \Delta t}{2(1-\mu^2)} (C_x + \mu C_y) \qquad (4.1)$$

Where E, α and μ are the elasticity modulus, coefficient of thermal expansion and Poisson coefficient for concrete. The C_x and C_y factors depend on the relation between the stiffness radius ratio of the slab and distance between analyses point and slab edges (figure 6).

Breadbury calculated the stresses regarding half the height of the slab and considered that the slab could rotate freely and that the thermal gradient is linear. Thus, the tension at the center of the slab is zero and the tensile or compression tension is calculated using equation 4.1. Losberg (Losberg, 1961) considered that the warp tension is obtained from expression 4.2, where the edge effect is negligible and considering the maximum value thereof. The outermost fibers were exposed to a tension twice as big as in 4.1.

$$\sigma = \frac{E \times \alpha \times \Delta t}{(1-\mu^2)} \qquad (4.2)$$

Where E, α and μ are the elasticity modulus, coefficient of thermal expansion and Poisson coefficient for concrete.

It is implicit in both equations that the slab weight restricts edge lifting, thereby allowing for tension relief, and that such equations do not consider that the elastic deformation of the compressed zone promotes a reduction in the specific deformation of the region under traction, thereby reducing tensile stress. There is also creep that promotes tension relief. Therefore, in practice, the resultant will be slightly lower.

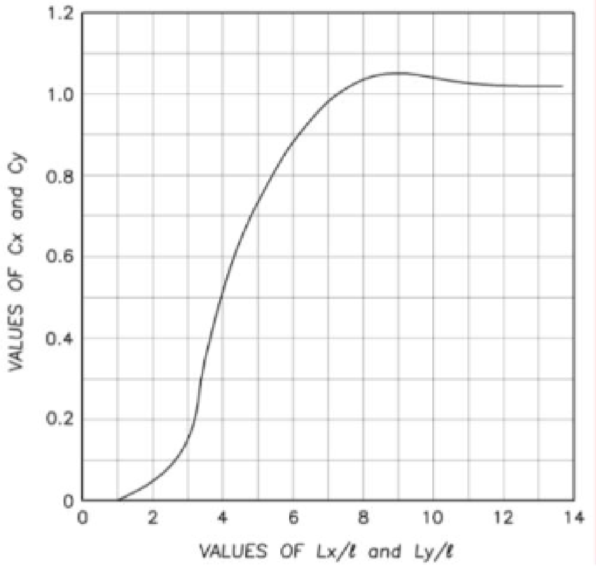

Figure 6: Coefficients C_x e C_y (Bradbury, 1938)

When the edges are lifted warping stresses are reduced, but under load, the slab returns to its original position and the stress generated by the elongation of the fibers that were initially shortened in order to return to the original position will be added to the tension set in by warping.

Thus, both the slab weight and the loading will promote a restriction to warping. In the simplest case where the thermal-hygrometric gradient is not great enough to promote slab lifting, an action similar to that which occurs in concrete pouring operations can be considered, with restriction to base movement, as in adhered overlays. However, in this case, the final tension will be smaller since the bottom layer is also free to move, and may therefore eventually retract as well as present elastic deformations.

This tension tends to diminish over time due to concrete creep. It is important to stress that in linear slab variations, the stresses generated by slab friction must also be considered, which is represented by the following expression:

$$\sigma_f \quad 5,0 \quad \gamma \times L \times f = \qquad (4.3)$$

Where γ is the specific weight of the concrete, f the friction coefficient between slab and sub-base, and L the slab length.

An equivalent thermal gradient that simulates differential shrinkage is normally used for drying shrinkage; a useful standard is to assume 1.3 °C/cm (Leonard & Harr, 1959); the linear system is far from the reality encountered in concrete, mainly during the initial periods in which the shrinkage differential observed at the surface of the concrete is really high.

More precisely, shrinkage can be indirectly measured by the relative humidity of concrete (Shin e Lange, 2004); the variation is not linear due to humidity diffusion in the concrete, mainly during the initial stages of drying. However, this characteristic does not change significantly with older concrete, as can be observed from figure NN (Foos et al, 2002). In these cases, the relative humidity begins to be constant at depths greater than 50mm from the surface, stabilizing at around 85% of relative humidity in the concrete.

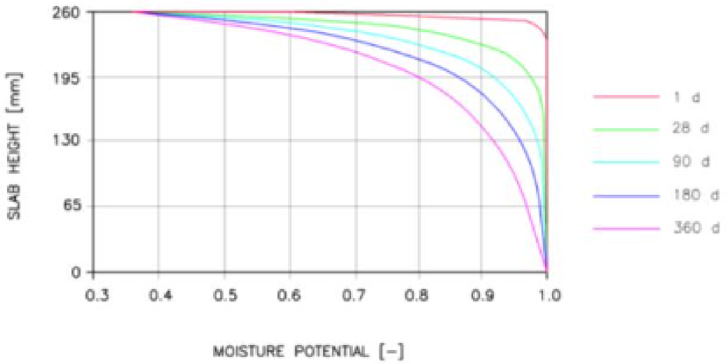

Figure 7: Moisture distribution over the cross-section of the slab as a function of moisture potential

Values obtained for stresses due to differential shrinkage can be significant, as is shown in figure 8, where tensile stresses greater than 2MPa occur (Foos et al., 2002). Similar warping tension values have been frequently reported in technical literature.

When the warping action occurs together with heavy loads, the added tension values may lead to premature rupture of the concrete slab. In case of the thermal gradient is positive, i.e., if the top of the slab is warmer than the base, thereby forming a convex surface, the action of the loads passing between the joints might promote a base up crack (NCHRP, 2003).

In case the thermal gradient is negative – the upper part is cooler than the bottom – a convex surface is formed and a load passing over the joints will generate a negative moment at the center of the plate, which may result in a top down fissure. The differential shrinkage of the plate always generates a negative equivalent thermal gradient.

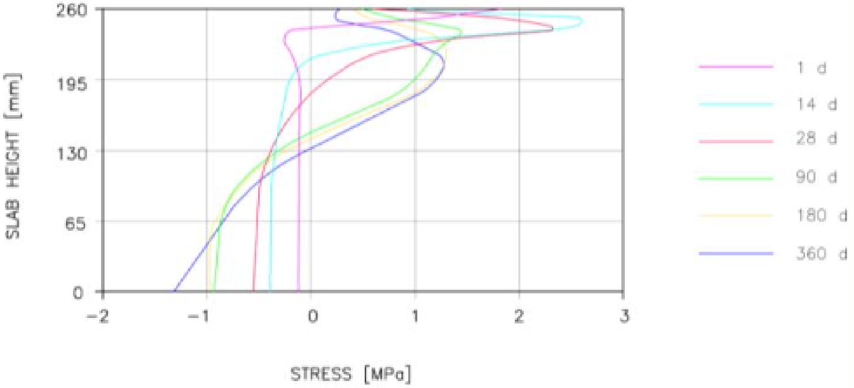

Figure 8: Stress distribution over the cross-section of a concrete pavement slab subject to drying on the upper side

4. Bonded overlay

A very common pathology in new constructions nowadays is the intense presence of fissures observed in bonded overlays. The execution of coating with the same standard of joints as that of the concrete pavement is almost the rule in such constructions, as if the behavior of the two systems were equal, but in reality they are completely different.

The bonded overlay is the most widely used system in new constructions to create an appropriate surface layer on bridges, overpasses and tunnels. This is also used in the recovery of old pavements, thereby promoting considerable structural capacity since the increase in thickness of the slab results from the direct addition of the adhered overlay and dictates that, in this case, the existing layout of the old pavement must be rigorously followed by the overlay and any fissure existing in the substratum will appear in the overlay.

Since in bridges and overpasses the rigidity of the structure is much greater than that of the bonded overlay, it is not common to consider such as a structural element, even though it is subject to the action of negative moments that appear in the bridge floor and that must be considered. The consideration of the hydraulic shrinkage of concrete, which is the greatest cause of fissures and other pathologies in adhered overlays, is much more important in fissure control.

This occurs because the overlay is adhered to older concrete, which normally has already developed most of the deformations, and will not follow the shrinkage of the overlay and will also try to prevent it, as can be observed in figure 9. Since the overlay is in fact adhered to the substratum it is prevented from freely retracting, as occurs in conventional pavements. Therefore, it must obey the following equilibrium equation for deformations (**Pigeon & Bissonete**, 1999):

$$\varepsilon_{free\_sh} \quad \varepsilon_{elastic} \quad \varepsilon_{creep} + \varepsilon_{crack} = \quad (4.4)$$

where $\varepsilon_{free\ retr.}$ is the hydraulic shrinkage that the overlay would suffer if it could move freely, $\varepsilon_{elastic}$ is the elastic deformation, ε_{creep} is the deformation due to concrete creep and ε_{fiss} is the deformation caused by micro-fissures; figure 9 (Pigeon & Bissonete, 1999) illustrates this issue.

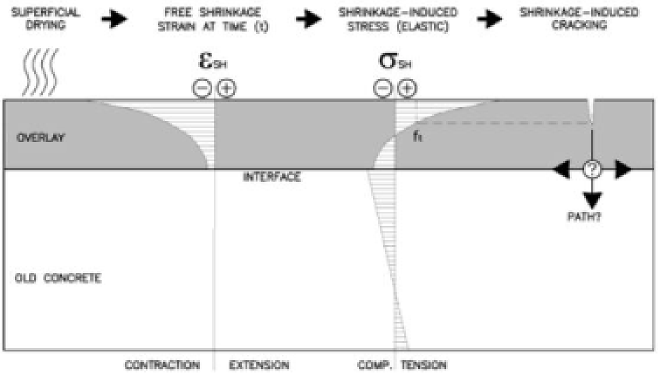

Figure 9: Simplified representation of shrinkage-induced stresses and cracks in a bonded overlay – tension is positive (Pigeon and Bissonnette, 1999).

In summary, when the sum of the elastic and creep deformations is less than free shrinkage, the overlay will crack. What happens after the fissure occurs is that there is an immediate relief of the shrinkage stress in the overlay, but this winds up being transferred to the interface, and tends to emphasize it [3].

When considering such issues, the most obvious question arises: which spacing shall be used for adhered overlay joints? The answer is simply: it is not necessary to place joints, since such are not efficient in controlling fissures, as will be demonstrated further on, besides favoring detachment, the number two factor regarding the pathologies of this system.

Despite the joint issue being worrisome, it can be easily demonstrated that since the deformations are high and there is no slab displacement, since it is adhered, the stresses are above the admissible level. For example, an excellent concrete presents hydraulic shrinkage of around 400 x 10^{-6} m/m; the elastic deformation and creep portions are around

[3] This fact can be easily proven in the adhered overlay regions where there are fissures or sawed joints if there is a hollow sound indicating the detachment.

150 x 10^{-6} m/m (**Burrows**, 1998), leaving a residual deformation of 350 x 10^{-6} m/m to be dissipated. Therefore, when considering the concrete module as being around 25GPa, an acting tensile stress of 6.25 MPa is obtained, which is well above those admissible for concrete. The conclusion is that the concrete will crack and the size of it should be prevented with adequate reinforcement (Rodrigues, 2006).

Near the joint region, the tensile stress in the upper fiber of the overlay is null and the question arises as to for what distance it will be less than the stress admissible for concrete. This distance can be calculated using Carlson's equation, which allows calculating the stress between two joints where there is restriction to movement of the base, i.e., full adherence to the substratum. Besides deformation, this stress depends on the degree of restriction to movement which, in turn, shall be a function of the $L/H_{overlay}$ ratio, where L is the spacing between joints. Roy Carlson, a great researcher who acted both in Brazil and in the USA during the second half of the last century, was one of the pioneers in studying such stresses, and developed equations to determine the restriction coefficient K_R (figure 10). His model is still currently used (ACI, 1995).

What can be understood from figure 10 is that when the L/H ratio decreases, i.e., the distance between the joints decreases to a same thickness of the adhered overlay, the restriction coefficient decreases, thereby reducing the tensile stress in the concrete, since: The stress in the upper layer depends on the relation L/H, where L is the length between the joints and H is the overlay thickness. The figure 10 shows the relationship between L/H and K_r, a restriction coefficient to be used in the expression:

$$\sigma_{tc} = \ _{R}\ \varepsilon E_c K \qquad (4.5)$$

However, if the previous example is considered where the tensile stress is of 6.25MPa, it would be reduced to 4.5MPa, which value could be supported by the concrete when K_R is around 0.7, which would imply in an L/H ratio of 9. If the overlay is 10cm thick the joints should be spaced every 90 cm, which is not a practical value to be executed. Also, this solution would impose an elevated risk since the displacement stress of the overlay would be even higher.

The crack control in bonded overlay is providing it with appropriated reinforcement to keep the cracking open under 0.2mm. To reach this condition using steel fiber, a minimum fiber content of 25kg/m³ (Shah and Yang, 1998) should be used; with the wire mesh, reinforcement rate of about 0.4% should be used but the more appropriate value can be calculated. The reinforcement location is very important and should be done the nearest from surface, respecting the corrosion protection of it.

5. Conclusions

5.1 Knowing the magnitude of warping stresses is important in concrete pavement design, because such may exceed those resulting from loading.

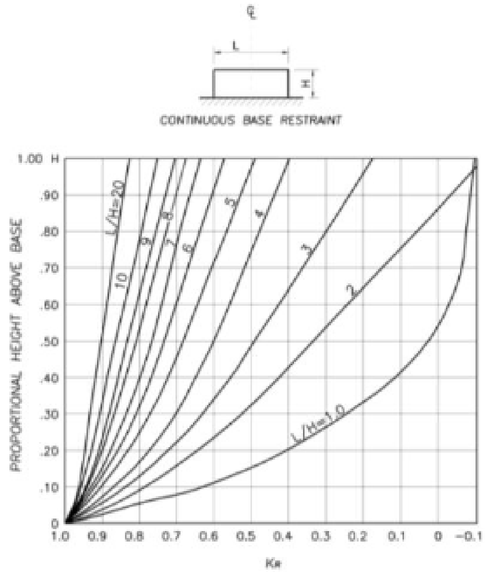

Figure 10: Restriction coefficient K_R (ACI, 1995)

5.2 Concrete characteristics capable of influencing upon secondary stresses, such as the elasticity modulus, thermal expansion coefficient and the maximum hydraulic shrinkage, must be made explicit in the pavement design.

5.3 The concrete temperature and environmental conditions at the moment of pouring may significantly influence the occurrence of fissures. In hot climates, nighttime concrete pouring should be considered.

5.4 Exaggerate specifications concerning the concrete mechanical properties and the cement demand might be harmful to the pavement durability instead of improving it. The abrasion resistance is not function of cement content; it is function of water/cement ratio.

5.5 Plastic fibers are powerful tools to control plastic fissures.

5.6 Adequately positioned steel rebar assists in warping control; steel fibers increase concrete ductility thereby improving its deformation capacity

5.7 In bonded overlay, the reinforcement or steel fiber must be considered for the crack control, besides adopting a low shrinkage concrete.

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