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The effects of the early-age concrete behaviour on the in-service performance of Jointed Plain Concrete Pavements

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The effects of the early-age concrete behaviour on the in-service performance of Jointed Plain Concrete Pavements

Proefschrift

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door

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Dedicated to The Road
Jesus said, "I am the Road, also the Truth, also the Life..."
(John 14:6)

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Summary

The characteristics of concrete as paving material make it particularly valuable in critical traffic hubs as intersections or roundabouts, roads with predominant trucks traffic or bus corridors, bus stops, bus stations, industrial floors, yards and airport aprons. In particular, in Jointed Plain Concrete Pavements (JPCPs) cracks are produced under the joints as a result of the restricted deformations in the concrete at early-age. The magnitude of these crack widths can have a positive or negative effect on the JPCP in-service performance. This effect is bigger in non-dowelled JPCPs, where the transfer of loads between slabs depends on the aggregate interlock (as in the innovative short slabs JPCPs). Hence, wider cracks can produce low Load Transfer Efficiency (LTE) and also contribute to functional deterioration like joint faulting that affects the comfort for the road user (JPCP roughness). Another result of the concrete behaviour at early-age is the presence (or absence) of Uncracked Joints (UnCrJ). The presence of UnCrJ certainly can affect the in-service performance of JPCPs because the designed slab length is different from the effective one.

The objective of this thesis is to evaluate the effects of the early-age concrete behaviour on the in-service performance of JPCPs. For that, a system approach is applied where the modelled cracking process of JPCPs is not only time-dependent but also space-dependent (i.e. considering the interaction of the group of joints). The Average Crack Width of the 1st series of cracks (AvCW1st) and the UnCrJ are considered the relevant results of the early-age behaviour affecting the structural and functional in-service performance of JPCPs. In fact, not only the structural pavement performance is part of the analysis but also the functional one because it is directly related with the pavement clients' satisfaction. Finally, an integral economic evaluation is made, i.e. considering the structural and functional performance of traditional and short slabs JPCPs.

In order to obtain realistic results of the modelling of the early-age concrete behaviour, a calibration procedure is defined taking into account the intended uses of the model and the necessity of being practical and useful for pavement clients as public agencies related with different JPCP applications (as urban, interurban, airports). In total, 10 test sections were considered to compare the modelled AvCW1st with the real-world AvCW1st. 2 test sections of traditional JPCPs (1 in Belgium and 1 in Chile) were used in the model calibration and 8 test sections located in Chile were considered in the post-calibration analyses. Between these 8 test sections, 3 correspond to traditional JPCPs and 5 to short slabs JPCPs. Besides, 3 of these 8 test sections are urban JPCPs, 2 interurban JPCPs, 2 test sections are on an industrial floor and 1 on an airport apron. After the calibration procedure, the agreement between the modelled and the real-world AvCW1st was improved from 70% until 100% (average).

The structural analysis was focussed in developing the relation LTE-AvCW1st. For that, field measurements were performed in Chile. This relation allows incorporating the direct cause of the LTE by aggregate interlock in mechanistic-empirical design methods. This is especially important for the innovative short slabs JPCPs where the LTE relies on aggregate interlock. Actually, due to the small crack widths (≤ 1.2 mm) the short slabs JPCPs are able to provide adequate LTE (≥ 70%) even without dowel bars. Besides, the application of high quality coarse aggregates provides even higher values of LTE. Actually, constructing with such kind of aggregates can even provide adequate LTE in traditional non-dowelled JPCPs.

The reduction of the crack width at joints in short slabs JPCPs produces a radical increase of the LTE. In terms of functional performance, this is fundamental to the low joint faulting observed in short slabs JPCPs, which is not predicted correctly by models developed for traditional JPCPs. For that, it is recommended not only to develop a new model to predict the joint faulting in short slabs JPCPs, but also that it includes the LTE. Furthermore, the relationships LTE-AvCW1st, developed originally for structural purposes, can be useful for functional purposes as well. The development of a deterioration model to predict the joint faulting of short slabs JPCPs should be a priority because the joint faulting is the major contributor to the JPCP roughness. Moreover, the effective aggregate interlock restricts the changes of slab curvature and thus contributes to the stability of the ride quality of short slabs JPCPs, measured in terms of the International Roughness Index.

Although avoiding the presence of UnCrJ is important for both traditional and short slabs JPCPs, in this last case it is crucial because the postulated benefits of this innovation are valid only if the slabs are effectively short. A practical, economic and effective method to avoid the presence of UnCrJ is the regulation of the relative joint depth in the construction of the pavement.

The analyses presented in this thesis (i.e. including the effects of the early-age concrete behaviour on the in-service performance of JPCPs) show the structural, functional and economic advantages of short slabs JPCPs compared to traditional JPCPs, resulting from the reduction of the slab length, the new traffic load configurations, the reduction of slab curvature (and the variation of it) and the finer cracks at joints.

Samenvatting

De karakteristieken van beton maken het een bijzonder geschikt verhardingsmateriaal voor kritieke infrastructuur zoals kruispunten en rotondes, wegen met voornamelijk vrachtverkeer, busbanen, bushaltes, busstations, industriële vloeren en terreinen, en platforms op vliegvelden. In ongewapende betonverhardingen (OGB) ontstaan scheuren onder de voegen als gevolg van verhinderde vervormingen in het jonge beton. De wijdte van deze scheuren kan een positieve of negatieve invloed hebben op het lange termijn gedrag van de verharding onder verkeer. Dit effect is groter in niet-verdeuvelde OGBs, waar de lastoverdracht tussen de betonplaten afhankelijk is van de 'aggregate interlock' (zoals in de innovatieve OGBs met korte platen). Wijde scheuren leiden tot een lage lastoverdracht en dragen ook bij aan de achteruitgang van de functionele eigenschappen van de verharding, zoals trapjesvorming die invloed heeft op de langsonvlakheid en daarmee op het rijcomfort voor de weggebruiker. Een ander effect van het gedrag van het jonge beton is de aanwezigheid (of afwezigheid) van niet-doorgescheurde voegen. Niet-doorgescheurde voegen zijn zeker van invloed op het lange termijn gedrag van OGBs onder verkeer omdat de ontwerp plaatlengte anders is dan de daadwerkelijk aanwezige plaatlengte.

Het doel van dit proefschrift is om de effecten van het gedrag van het jonge beton op het lange termijn gedrag van de OGB te evalueren. Daartoe is een systeembenadering gehanteerd waarbij het scheurvormingsproces van OGBs niet alleen afhankelijk is van de tijd maar ook van de plaats (met name is de interactie van een groep voegen in beschouwing genomen). De gemiddelde wijdte van de eerste serie scheuren en het percentage niet-gescheurde voegen worden beschouwd als de relevante resultaten van het gedrag van het jonge beton die van invloed zijn op het structurele en functionele gedrag van OGBs op lange termijn. In feite is niet alleen het structurele gedrag van de verharding onderdeel van de analyse maar ook het functionele gedrag omdat dit direct gerelateerd is aan de tevredenheid van de gebruikers van de verharding. Tenslotte is een economische evaluatie uitgevoerd waarbij zowel het structurele als functionele gedrag van OGBs met traditionele lange betonplaten en met innovatieve korte betonplaten in beschouwing is genomen.

Om realistische resultaten van de modellering van het gedrag van het jonge beton te verkrijgen, is een kalibratie procedure gedefinieerd waarbij rekening is gehouden met het beoogde gebruik van het model en met de noodzaak dat het model praktisch en bruikbaar moet zijn voor de beheerders van OGBs in diverse toepassingen, zoals wegen binnen en buiten de bebouwde kom en vliegvelden. In totaal zijn 10 proefvakken geselecteerd om de gemodelleerde gemiddelde wijdte van de eerste serie scheuren te vergelijken met de gemeten gemiddelde wijdte van die scheuren. Twee proefvakken met een traditionele OGB met lange betonplaten (1 in België en 1 in Chili) zijn gebruikt voor de kalibratie van het model en 8 proefvakken in Chili zijn gebruikt voor de post-kalibratie analyses. Drie van deze 8 proefvakken hebben een traditionele en de overige 5 hebben een OGB met korte betonplaten. Drie van deze 8 proefvakken zijn op wegen binnen de bebouwde kom, 2 op wegen buiten de bebouwde kom, 2 op een industriële vloer en 1 op een vliegveldplatform. Door de kalibratie procedure is de verhouding tussen de gemodelleerde en gemeten gemiddelde wijdte van de eerste serie scheuren verbeterd van (gemiddeld) 70% tot 100%.

De nadruk in de structurele analyse ligt op de ontwikkeling van de relatie tussen de lastoverdracht en de gemiddelde wijdte van de eerste serie scheuren. Daartoe zijn in Chili

veldmetingen uitgevoerd. Door deze relatie is het mogelijk om de directe oorzaak van lastoverdracht, ‘aggregate interlock’, te integreren in mechanistisch-empirische ontwerpmethoden. Dit is met name belangrijk voor de innovatieve OGBs met korte betonplaten waarbij de lastoverdracht wordt bepaald door de ‘aggregate interlock’. Als gevolg van de geringe scheurwijdten ($\leq$ 1,2 mm) is de lastoverdracht in de niet-verdeuvelde voegen adequaat ($\geq$ 70%). De toepassing van hoogwaardige grove steenslag in het beton resulteert in nog hogere waarden van de lastoverdracht. Gebruik van dergelijke steenslag kan zelfs in traditionele OGBs met lange niet-verdeuvelde betonplaten een adequate lastoverdracht opleveren.

De vermindering van de scheurwijdte onder de voegen in OGBs met korte betonplaten leidt tot een forse toename van de lastoverdracht. In termen van functioneel gedrag is dit bepalend voor de geringe trapjesvorming die waargenomen is bij OGBs met korte betonplaten en die niet goed voorspeld wordt met modellen die zijn ontwikkeld voor traditionele OGBs. Derhalve wordt aanbevolen om niet alleen een nieuw model voor de trapjesvorming in OGBs met korte betonplaten te ontwikkelen, maar ook om daarin de lastoverdracht mee te nemen. Verder kan de relatie tussen de lastoverdracht en de gemiddelde wijdte van de eerste serie scheuren, die oorspronkelijk is ontwikkeld voor constructieve doeleinden, ook bruikbaar zijn voor functionele doeleinden. De ontwikkeling van een model dat het verloop van de trapjesvorming in OGBs met korte betonplaten voorspelt, moet prioriteit krijgen omdat trapjesvorming de grootste bijdrage levert aan de langsonvlakheid van een OGB. Bovendien beperkt een effectieve ‘aggregate interlock’ de variatie van de kromming van de betonplaten hetgeen bijdraagt aan een constanter rijcomfort van OGBs met korte betonplaten, gemeten in termen van de International Roughness Index.

Het voorkómen van niet-gescheurde voegen is belangrijk voor zowel OGBs met traditionele, lange betonplaten als voor OGBs met korte betonplaten. In het laatste geval is het zelfs cruciaal omdat de voordelen van deze innovatie alleen optreden indien de betonplaten daadwerkelijk kort zijn. Een praktische, economische en effectieve methode om niet-doorgescheurde voegen te vermijden is de regulering van de diepte van de zaagsnede tijdens de uitvoering van de OGB.

De analyses in dit proefschrift, inclusief de effecten van het gedrag van het jonge beton op het lange termijn gedrag, tonen de structurele, functionele en economische voordelen van OGBs met korte betonplaten ten opzichte van traditionele OGBs met lange betonplaten. Deze voordelen zijn het gevolg van de reductie van de plaatlengte, de gunstiger verkeerslastconfiguratie, de vermindering van de kromming van de betonplaten (en de variatie daarvan) en de nauwere scheuren onder de voegen.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AHP	Analytic Hierarchy Process
APT	Accelerated Pavement Testing
AvCW1st	Average Crack Width of the 1st series of cracks
BRRC	Belgian Road Research Centre
BC	Bus Corridor
CBR	California Bearing Ratio
CF	Correction Factor
CRCP	Continuously Reinforced Concrete Pavement
EESC	Early-Entry Saw-Cutting
ESAL	Equivalent Single Axle Load of 80 kN
EverFE	Finite elements software for the structural analysis of JPCPs
FE	Finite Elements
FWD	Falling Weight Deflectometer
HDM-4	Highway Development and Management System (version 4)
IRI	International Roughness Index
JF	Joint Faulting
JPCP	Jointed Plain Concrete Pavement
JRCP	Jointed Reinforced Concrete Pavement
JS	Joint Spalling
LTE	Load Transfer Efficiency
LVDT	Linear Variable Differential Transformer
M-E	Mechanistic - Empirical
MEPDG	Mechanistic-Empirical Pavement Design Guide
MCDM	Multi-Criteria Decision-Making
NHL	National Highway Laboratory of Chile
PCP	Plain Concrete Pavement
RF	Relaxation Factor
RJD	Relative Joint Depth

RV	Residual Value of the Faultimeter
TC	Transverse Cracking
UnCrJ	Uncracked Joints
UnJs	Unsealed Joints
WP	Walking Profiler
WisDOT	Wisconsin Department of Transportation

1. INTRODUCTION

1.1. INTEGRAL ANALYSIS OF PAVEMENTS

According to the standard ISO 9000:2005, quality is the degree to which a set of inherent characteristics fulfils requirements, that is a need or expectation stated (ISO 9000, 2005). This definition is undeniably related with the clients' satisfaction that is the client's perception of the degree to which their requirements have been fulfilled. In the specific case of pavements: Who are the clients to serve? Haas & Hudson (1996) define the users and the owners as the largest groups of clients for pavements. In particular in case of public roads, the direct user clients (automobile drivers and passengers, motorcyclists and bicyclists, truck operators, etc.) and the owners or agents (federal, state/provincial and local/municipal transportation agencies) (Haas & Hudson, 1996). Hence, pavement solutions should incorporate factors related to their clients' satisfaction. In a classification of these factors, the ride quality was assigned a high priority for all the pavement users and the transportation agencies. However, the structural adequacy had the lowest grade in the pavement users' opinion (Haas & Hudson, 1996). In effect, Loizos & Plati (2008) states 'although structural capacity seems to be the major concern of many pavement engineers, road users primarily judge the quality of a road based on its roughness' or ride quality, i.e. the evaluation of the driver is primarily related with the functional pavement condition. Hence, even when the structural capacity of the pavement has the highest technical priority, it is important that the pavement solution also incorporates the relevant factors for the clients that, at the end, the pavements needs to serve.

In pavement engineering, functional performance is defined as a pavement's ability to provide a safe, smooth riding surface. These attributes are typically measured in terms of skid resistance or ride quality (Caltrans, 2003). According to Molenaar (2001) the functional pavement condition is especially of importance to the road user and is related, between others, to longitudinal evenness. Thenoux & Gaete (1995) expand the definition of functional pavement condition from the surface quality to the general condition of the pavement taking into account all the factors that negatively affect the serviceability, safety and users costs.

Hence, taking into account the pavement clients' satisfaction and the fact that paved roads represent the largest in-place asset value of transportation infrastructure in most countries, it is important to make an integral analysis of the pavements, that considers not only the structural adequacy but also the functional condition of every pavement alternative. In the specific case of the type of pavement studied in the present thesis, Jointed Plain Concrete Pavements (JPCPs), functional deteriorations such as joint faulting and pavement unevenness (roughness) are fundamental. Pavement roughness is especially significant because it is strongly related to the pavement performance providing good overall measure of the pavement condition and correlates well with subjective assessments (Loizos & Plati, 2002, 2008).

1.2. CONCRETE AS PAVING MATERIAL

Concrete is well known as a durable pavement material that, in general, does not require invasive interventions of maintenance or rehabilitation that can affect the users. This is particularly valuable in critical traffic hubs as intersections or roundabouts. In addition, concrete pavements can resist high traffic demands which make them an interesting alternative to roads with predominant trucks traffic or bus corridors. Furthermore, concrete pavements are a common solution for bus stops where the pavement is subjected to constant braking, accelerating and the action of pseudo-static loads. This is the case on bus stations as well, where also constant manoeuvring takes place. Similar loadings occur on some industrial applications such as warehouses that also include the static loads of the storage. Moreover, because of their characteristics, concrete pavements are the general solution for airport aprons. In effect, the concrete pavement can resist those kinds of traffic demands without deformations and it is resistant to aggressive agents as oils, greases, hydrocarbons and fuels.

However, concrete exhibits deformations at early-age because of thermal changes, drying and autogenous shrinkage. As the concrete is in contact with the supporting structural base, a restriction to the free movement is generated by this shrinkage that produces tension in the concrete and finally cracks. The different types of concrete pavements deal with the crack phenomenon in different ways. In Continuously Reinforced Concrete Pavement (CRCP) the cracking process is controlled by means of longitudinal steel reinforcement. In Jointed Reinforced Concrete Pavement (JRCP) the cracking process is controlled by means of a combination of contraction joints and reinforcing steel. The type of concrete pavement studied in the present thesis, the Jointed Plain Concrete Pavement (JPCP), controls the cracking using contraction joints, which are weakened cross-sections created by a saw-cut. The purpose is to locate the cracks specifically under the saw-cut.

The saw-cutting of the contraction joints is one of the most critical issues of the construction of a JPCP because the pavement needs to be cut before the cracks occur (to avoid random cracking) but not so early that it can produce ravelling (Fig. 1.2.1).

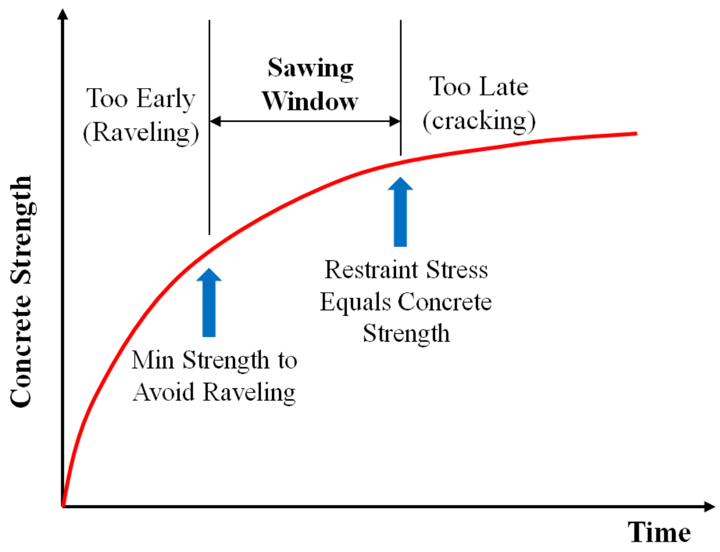

Fig 1.2.1. Concept of sawing window (Okamoto et al, 1994).

But even if the saw-cutting process is carried out correctly, the behaviour of the early-age concrete can have a positive or negative effect on the JPCP performance. In particular, the crack width under the joints and the presence of uncracked joints, both depending on the early-age concrete behaviour, will have a decisive influence on the in-service performance of JPCPs.

1.3. IN–SERVICE PERFORMANCE OF JPCPs

Although JPCPs present favourable characteristics that make them especially suitable for bus corridors, bus stations, airport aprons, industrial yards, urban and interurban pavements, the traditional JPCP can develop deteriorations as cracks in the slabs, joint spalling and joint faulting. Finally, all these deteriorations affect the value of the International Roughness Index, IRI (Eq. 1.3.1) that is the most significant factor associated with changes in drivers' perceptions of road roughness, as well as drivers' acceptability of a roadway's condition (Shafizadeh et al., 2002).

$$IRI = IRI_0 + 0.00265*(TFAULT) + 0.0291*(SPALL) + 0.15*10^{-6}*(TCRACK)^3 \qquad (1.3.1)$$

Where IRI =International Roughness Index (m/km); IRI_0 = initial roughness at construction (m/km); $TFAULT$ = transverse joint faulting (mm/km); $SPALL$ = spalled joints (%); $TCRACK$ = transverse cracks (Nº/km)

One of the results of the concrete behaviour at early-age is the crack width under the joints. The magnitude of the width of these cracks can have a positive or negative effect on the JPCP performance. This effect is bigger in non-dowelled JPCPs, where the transfer of loads between slabs depends on the aggregate interlock, and consequently the contact between the aggregates at the faces of the cracks is fundamental. Hence, wider cracks can produce low Load Transfer Efficiency (LTE), increasing the deterioration and the risk of structural failure. Wider cracks also contribute to functional deterioration like spalling and especially joint faulting that affects pavement unevenness (IRI –value), which means less comfort for the road user and higher rate of deterioration of the pavement. Thus, the knowledge of the crack width, and the most important variables that control its magnitude, is fundamental to determine the effect of the early-age concrete behaviour on the in-service performance of the JPCP and to take decisions in order to obtain narrower cracks for a better structural and functional JPCP performance.

The occurrence of joint faulting (resulting in a higher IRI) is more critical when joint seals are not functioning well and the concrete slabs are overlaying a granular base. In effect, when the seals exhibit failures and they are not well maintained, they allow the ingress of water and incompressible materials in the joints. The introduction of water can drag fines from the base (pumping), resulting in reduced support that can produce joint faulting or even cracks parallel and close to the joints.

Another result of the concrete behaviour at early-age is the presence (or absence) of Uncracked Joints (UnCrJ). The structural design of JPCPs assumes a slab length defined by joints that effectively crack. If a percentage of the joints remain uncracked then the design hypothesis is not necessary valid anymore. Consequently an acceleration of the deterioration in the performance of the JPCP can occur, and even premature failure of the pavement. In general, the structural design of JPCPs is controlled by the combination of traffic and environmental loads. In the traditional structural design, these last ones are basically considered by the curvature produced in the concrete slab. If the slab length is longer than the one assumed in the structural design, the effect of the slab curvature is more critical than the one originally calculated. More slab curvature can affect the functional condition of the pavement as well, for instance the driver comfort of the pavement users.

In JPCPs, nowadays innovations in joints configurations are available such as short joints spacing (short slabs), unsealed joints and early-entry saw-cutting of joints. These innovations can affect the pavement performance. In fact, shorter slabs not only should produce less curvature, but also they introduce a new configuration of traffic loads acting independently on every slab. There are different studies regarding the structural capacity of short slabs (Roesler et al., 2012; Covarrubias, 2012; Salsilli et al., 2013), but there is still a necessity for a functional analysis of the performance of this JPCP innovation. For instance, less slab curvature can influence the surface regularity perceived by the pavement users. Similarly, early-entry saw-cutting has become more accepted, especially in the USA, but there are some concerns regarding its influence on the long-term performance of JPCPs (Krstulovich et al., 2012).

The necessity of considering the JPCP performance is similar for unsealed joints. It is not enough to focus the studies on the joint performance, it is required to analyze the effect of that joint performance on the pavement performance, and the ride quality the users (the clients) experience. Accordingly, if the traditional practice of sealing the joints is a better solution for the pavement, they need to enhance the medium and long term performance of the JPCPs (lower levels of joint faulting and IRI), including the costs of sealing, resealing, repair and the effects upon the users, as delays and safety costs.

1.4. OBJECTIVE OF THE RESEARCH

The general objective of the thesis is to evaluate the effects of the early-age concrete behaviour on the in-service performance of JPCPs. The evaluation considers traditional JPCPs and innovative short slab JPCPs.

To fulfil the general objective the following specific objectives have been defined.

1. Model the early-age behaviour of JPCPs.

2. Determine relevant results for the link between the early-age behaviour and the in-service performance of JPCPs.

3. Analyse the structural performance of JPCPs including the relevant results of the early-age behaviour.

4. Analyse the functional performance of JPCPs including the relevant results of the early-age behaviour.

5. Make an economic analysis of the JPCPs including their structural and functional performance.

1.5. THE RESEARCH APPROACH

To realize the objective of the thesis, it is necessary to model the cracking process under the joints of JPCPs in order to determine the crack width that can be linked with a Load Transfer Efficiency (LTE) representative for the in-service performance of the JPCP. The Average Crack Width of the 1st series of cracks (AvCW1st) is a useful value to make this link (Fig. 1.5.1) because the 1st series of cracks are the wider ones (Houben, 2010; Roesler et al, 2012), therefore they control the performance.

Fig. 1.5.1. AvCW1st as the link between the early-age behavior and the in-service performance of JPCPs.

Erkens (2002) states that a model is, by definition, a simplification used to explain, calculate or predict something, for example a physical phenomenon. In the model the real situation is simplified by leaving out those aspects that are not important for whatever is being investigated. As a result, what is a "good" model depends on what it is used for and how accurate the prediction or calculation should be (Erkens, 2002). In the present research, a "good" model of the very complex process of JPCP cracking is the one able to predict correctly the AvCW1st. Methods of pavement design as AASHTO or MEPDG include a simplified formula for the calculation of the crack width under the joints. This is a formula that incorporates the effects of shrinkage and the thermal deformations by fixed mean values (AASHTO 1993; NCHRP 2003). In contrast, the modelling of the present research includes the calculation of the initiation and development of the cracking process in the JPCPs considering the pavement as a system with pre-defined weakened sections (the joint locations). In this system the value of the crack width is not only the result of the material changes but also of the location of the 1st series of cracks, the 2nd ones and so on until the cracking process is completed. The changes that the concrete experiences since early-age are modelled by the development in time of the concrete stiffness and tensile strength, the shrinkage and the thermal deformation rather than fixed mean values. In addition, the modelling includes specific construction conditions of the pavements rather than general cases. These construction conditions are defined in a factorial design that includes, for instance, the season and time of the day when the JPCP is built, the concrete grade, and the saw-cutting method.

In addition, as the model considers the development in time and space of the different series of cracks, it is possible to know the joints that effectively do crack and the joints that remain uncracked. This information determines the effective slab length of the JPCP that differs from the designed slab length when Uncracked Joints (UnCrJ) are present. In the case of short slabs, the reduction of concrete thickness (and costs) requires that the slabs are effectively shorter, because only then the slab curvature reduction and the postulated new traffic load configuration are valid. For instance, if in a JPCP with short slabs 1 of 2 joints remains uncracked, the effective slab length is the one of a traditional JPCP, but with 70 to 100 mm less thickness (Roesler et al., 2012).

Hence, in the present research UnCrJ and AvCW1st are the results from the early-age behaviour influencing the performance of JPCPs. However, priority is given to the AvCW1st because, between other factors (presented in later chapters), it is associated to the evaluation condition of the analysis. In effect, as short slabs are an innovation of traditional non-dowelled JPCPs, the new design configuration needs to be compared with the traditional one in order to evaluate if effectively it represents an improvement in the design in the sense that it positively affects the field performance (Montgomery, 2012). To do this, an objective criterion for comparison needs to be established. Considering the importance of the AvCW1st for the link between the early-age behaviour and the in-service performance of JPCPs, the objective criterion used is the comparison of the AvCW1st between traditional and short slabs JPCPs. Indeed, in non-dowelled JPCPs, the crack width (at joints) is directly related to the most influential load transfer mechanism in non-dowelled JPCPs, i.e. the aggregate interlock.

The Fig. 1.5.2 presents a scheme of the research approach of the thesis. In this Figure the AvCW1st is highlighted as the main link between the early-age behaviour and the in-service performance of JPCPs.

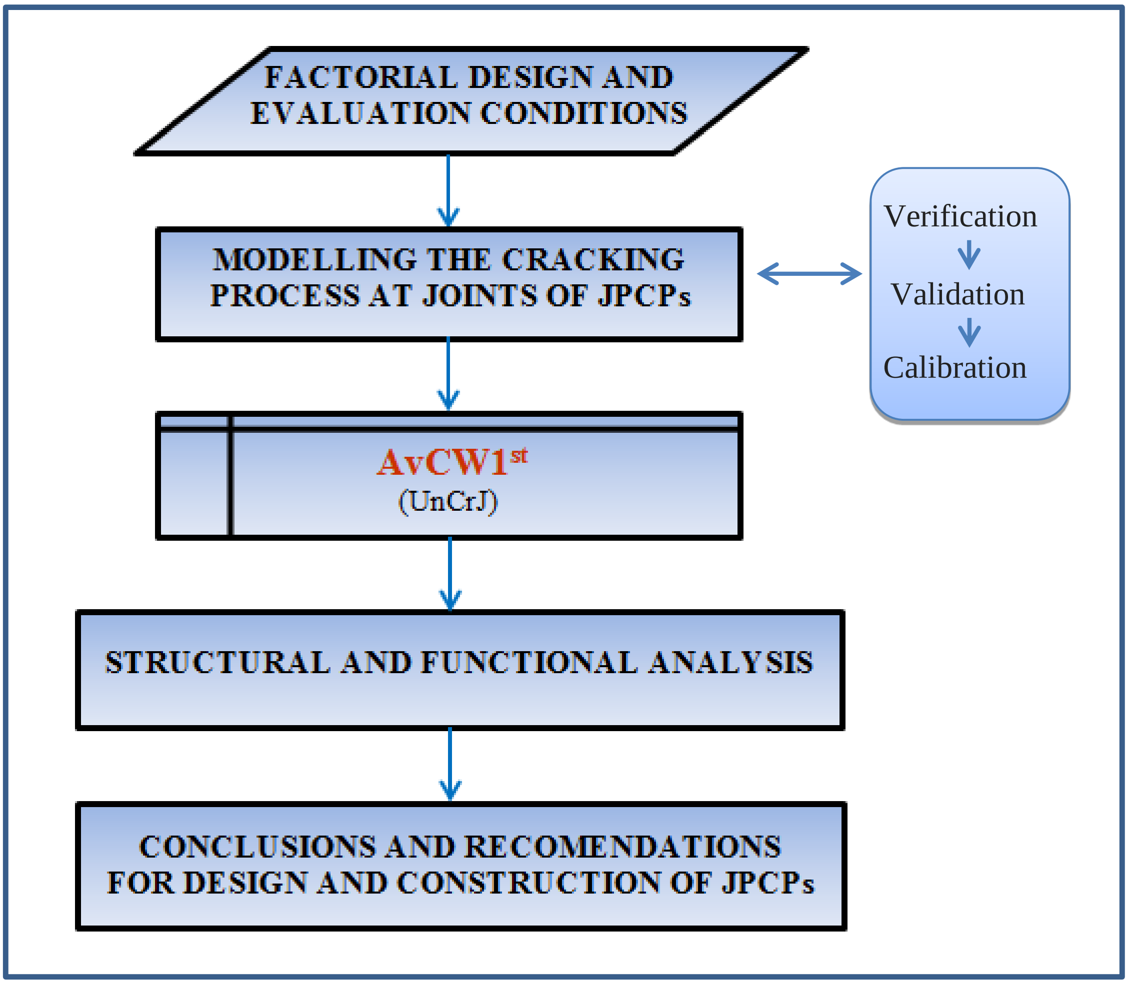

Fig. 1.5.2. Research approach of the thesis

In the Fig. 1.5.2 the verification, validation and calibration are progressive phases of the comparison, and adjustments, of the modelling results and real-world behaviour of JPCPs (focussed on the AvCW1st). The fundamentals, justification and complete definitions of these phases are given in Chapter 5 of this thesis.

As showed in Fig. 1.5.2, besides the determination of the AvCW1st (and the presence of UnCrJ) the emphasis of the research is on the analysis of the structural and functional performance of JPCPs.

The empirical support of the thesis can be summarized as follows:

- Early-age behaviour field data: centred on the determination of the AvCW1st (secondly the UnCrJ)
- Structural field data: focussed in the relationship between AvCW1st- LTE
- Functional field data: presence of joint faulting, amount of joint spalling (sealed and unsealed joints), and variations of IRI due to the slab curvature.

1.6. ORGANIZATION OF THIS THESIS

The thesis is composed of nine chapters.

Chapter 1 presents a brief introduction of the context, objectives and the research approach of the thesis. It describes the importance of an integral analysis of pavements in order to include the pavement clients' needs. In the chapter also the challenges of concrete as a paving material are discussed, especially how the early-age behavior of concrete can have a positive or negative effect on the in-service performance of JPCPs.

Chapter 2 gives the background of the research. It presents an analysis of the related literature identifying the research needs associated with the objectives of the thesis. For instance, the necessity of a model of the early-age concrete behaviour incorporating the interaction (in time and space) of the group of joints of the JPCP, or the necessity of establishing the relation between the LTE and the crack width (at joints) which is the direct cause of the provision of LTE by aggregate interlock in non-dowelled JPCPs. For that, in every section of the chapter the literature is presented followed by a brief analysis that exhibits the research needs that are treated in the following chapters of the thesis.

In Chapter 3 the modelling of the cracking process of JPCPs is described. It states the time-dependent concrete properties, the climatic conditions, the occurring tensile stresses in JPCPs and the development of the cracking process in JPCPs. In addition, a comparison of models is described, focussed on the most significant differences. Finally, the necessity of comparisons of the modelling results with the behaviour of real pavements is presented.

Chapter 4 describes the factorial design and the evaluation conditions of the research. The function of this factorial design is the definition of categories of variables, and their values, needed for the determination of the main outputs of the modelling of the concrete behaviour since early-age (AvCW1st and the presence of UnCrJ). In addition, associated to the factorial design, an objective criterion for comparison is defined in order to assess if short slabs effectively represent an improvement in the design that positively affects the pavement performance.

Chapter 5 presents the comparison between the modelling results with the real-world behaviour of JPCPs. The chapter starts with the intended uses of the model, the terminology associated to the verification, validation and calibration phases and the scope and consistency of the real-world data acquisition. The selection of the JPCPs sections according to the 4 main JPCPs applications (urban, interurban, industrial and airports) and the method of measurements are also described. Furthermore, an overview of the JPCPs sections is also presented. Finally, the progressive process of verification, validation and calibration of the model of cracking at joints in JPCPs is described.

The Chapter 6 describes the structural analysis of non-dowelled JPCPs, starting with the structural benefits of reducing the slab length in urban roadways, rural roads, industrial yards and floors, and airport aprons. The structural analysis is focussed on the relationship between LTE – AvCW1st. In fact, field measurements of this relationship are given in the chapter. Finally, the structural effects of the presence of uncracked joints are also presented.

Chapter 7 describes the functional analysis of non-dowelled JPCPs, starting with the effects of the unsealed joints on the in-service performance of JPCPs. After that, the modelling of the joint faulting is compared with trends of joint faulting observed in real-world short slabs. Next, an evaluation of the ride quality of JPCPs is made using the Analytic Hierarchy Process. Then, a specific study about the variations of the ride quality due to the slab curvature is presented, including field measurements of IRI performed in traditional and shot slabs JPCPs. Finally, the effect of the presence of uncracked joints on the functional performance is described.

Chapter 8 gives an economic comparison between traditional and short slabs JPCPs. This costs-benefits analysis is made from an integral perspective, including the priorities of the pavement clients. Accordingly, not only the structural performance but also the functional behaviour of the alternatives are included in the economic analysis. Finally, the effects of the uncracked joints in the economic comparison are addressed as well.

Finally, Chapter 9 presents the conclusions of the thesis and recommendations for the design and construction of JPCPs. In particular, recommendations about the slab length of JPCPs, saw-cutting method and use of seal at joints are given. In addition, recommendations for future research are given.

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2. ANALYSIS OF RELATED LITERATURE

2.1. INTRODUCTION

The present chapter presents a literature review identifying the available knowledge associated with the objectives of the present thesis, i.e. the modelling of the early-age behaviour of JPCPs and its effects in the structural and functional in-service performance of traditional and short slabs JPCPs. In every section, literature on a specific aspect of the project treated in this thesis is presented, followed by a brief analysis that describes the issues to be treated about that aspect in the next chapters of the thesis.

The analysis of the related literature starts with the innovations in joint configurations of JPCPs. After that, the literature related to modelling of the cracking process of JPCPs and the effects of this process on the structural and functional performance of JPCPs is analysed. Although these last topics are more directly related with the general objective of the thesis, the present research not only includes traditional JPCPs but especially short concrete slabs. The design features of the innovative short slabs include Unsealed Joints (UnJs) (Covarrubias, 2008; 2012) and, in general, their joints are made using Early-Entry Saw-Cutting (EESC). However, UnJs and EESC can also be applied to JPCPs with conventional slab length. In any case, the innovations need to be compared with the traditional systems in order to evaluate if they effectively represent an improvement in the design that positively affects the pavement performance (Montgomery, 2012).

2.2. INNOVATIONS IN JOINT CONFIGURATIONS OF JPCPS

2.2.1. Short slabs

In Chile, the Chilean Cement and Concrete Institute started promoting and building test sections of JPCPs with short slabs in 2004, placed directly over a granular base (Salsilli et al, 2013; 2015). In USA, Roesler et al (2012) report the construction of three 40 [m] test sections of short concrete slabs of 1.8 [m] length at the University of Illinois in order to perform an Accelerated Pavement Testing (APT) on them. In addition Covarrubias (2008, 2011, 2012) describes experiences of short concrete slabs in Chile, Guatemala and Peru.

Short slabs imply a change of paradigm regarding the traditional practice of limiting the number of joints. The reduction of joint spacings produces a new traffic load configuration for the slabs, as well as a reduction of slab curvature that allow the slabs to resist more traffic loads than traditional JPCPs with the same thickness (Roesler et al, 2012) or reduced thickness of the JPCP to resist the same traffic demands (Covarrubias et al, 2010). In fact, this results in thinner concrete pavements and lower initial construction costs (Roesler, 2013). The savings can reach 30% (Covarrubias, 2008; 2012) since the joints of short slabs are also undowelled and unsealed (Covarrubias, 2008; 2012). Other design features of short slabs are: slab length < 2.5 [m]; granular base with limited fines content (≤ 6% to 8% passing 75 μm); thin saw-cut at joints (2-3 mm thick); no dowel or tie bars (Roesler, 2013)[1].

[1] General design features of traditional JPCPs: slab length ≥ 3.5 m; sealed joints (initial saw-cut ≥ 3 mm + widening to ≥ 8 mm); granular base (≤ 10% to 15% passing 75 μm), application of dowels and tie bars.

The Fig. 2.2.1 shows the short slabs of the Route 60-Ch ‘Christ the Redeemer’ in Chile. This is the main transport route between Chile and Argentina and so carries quite heavy traffic (1500 trucks/day per direction). Short slabs of 2 [m] length required 170 [mm] concrete thickness instead of the 220 [mm] thickness of the traditional JPCP.

Fig. 2.2.1. Short slabs at the Route 60-Ch ‘Christ the Redeemer’, Chile.

Some aspects of the technology of short slabs are patented by a private Chilean company (Covarrubias, 2007, 2009a, 2009b). This situation and the continued interest for applied research and pavement innovations of the National Highway Laboratory of Chile have resulted in a concentration of test sections and projects of short slabs in Chile. As a result, the Chilean Highway Agency developed structural design guidelines for this innovation (Chilean Highway Agency, 2012). In addition, the Chilean Cement and Concrete Institute funded a research project to develop a design method for JPCPs with short concrete slabs with the objective to have it available for practitioners and public agencies (Salsilli et al, 2013). This Mechanistic - Empirical method was developed by Salsilli et al. (2013, 2015) using the principles of dimensional analysis (Ioannides, 1984).

The emphasis in the development of short slabs has been on the structural analysis of this innovation (Salsilli & Wahr, 2010; Covarrubias, 2009, 2011, 2012; Salgado, 2011; Roesler et al, 2012; Chilean Highway Agency, 2012; Salsilli et al, 2013, 2015). However, the largest groups of pavement clients, i.e. users and owners (or agents acting on their behalf, as transportation agencies) assign priority to the functional condition of the pavements, specifically the comfort of drivers (Haas & Hudson, 1996). Therefore, an integral analysis of this innovation is necessary, including the priorities of the pavements clients, i.e. the functional performance of JPCPs with short slabs.

2.2.2. Unsealed Joints

The traditional approach of sealing transverse contraction joints is estimated to account for between 2 and 7 percent of the initial construction costs of a JPCP (Hall, 2009). In fact, the sealing of joints has associated costs due to material, labour, construction, repair, traffic and lane closure. When the costs of keeping the joints sealed for 10 years is added, the JPCP with sealed joints ends up costing up to 45% more than the one with unsealed joints (Shober, 1986). The function of the seals is to avoid the contamination of the joints with coarse incompressible materials and to impede the infiltration of water that can drag fines from the base (pumping), which would lead to less support, and then joint faulting. The problem with coarse incompressible materials in the joint is the potential to produce excessive pressure at the edges of the joint, which can lead to spalling or even splitting cracks. But the joint seals are not working well enough, they commonly have adhesive and/or cohesive failures (Jung et al, 2011) and long-term joint faulting data shows a strong correlation with annual rainfall. The average service life of the joint seals is less than 10 years (Jung et al, 2011) (Fig. 2.2.2).

Because the joints were constructed wide enough to receive the seal, when they are not well maintained water and materials can enter to the joint (Fig. 2.2.2). The repair costs can be significant, especially in countries with important budget limitations where maintenance cannot be assigned priority. Because of this, the extra costs of the joint seals and their performance, there is increased interest in eliminating transverse joint sealants as a means of lowering concrete pavement construction and maintenance costs.

Fig. 2.2.2. Examples of joint seals deteriorations in a JPCP after 11 years in-service (Pradena & Diaz, 2016).

Already in 1995, 10 States of USA did not rely on the sealants and 1 State (Wisconsin) reported that it had dispensed with joint sealing entirely (Jung et al, 2011). In effect, for 50 years the Wisconsin Department of Transportation (WisDOT) has investigated joint filling/sealing in urban and rural areas, for various traffic levels and truck loadings, on open and dense graded bases, on sandy to silty-clay soils, with short and long joints spacing, with and without dowels bars, etc (Shober, 1997). In addition, there are other experiences with UnJs in USA (Hall, 2009), Austria, Spain and Belgium (Burke & Bugler, 2002). The technical bases of UnJs are: thin saw-cuts of the joints and limitation of the fines content of the underlying base/soil layer. In effect, in UnJs a saw-cut as narrow as possible (current technology allows cuts ≤ 3 mm), which impedes the penetration of coarse material in the joint saving the cost of sealing and re-sealing the joints. The thin saw-cutting can be performed with Early-Entry Saw-Cutting (EESC), conventional equipment but using a thin blade, or a combination of both methods. Unsealed joints become uniformly filled with fine incompressible material (Fig. 2.2.3) and when the pavement expands the stress is uniformly distributed across the entire pavement cross section. This uniform stress will be about 7 – 14 [MPa] maximum, well below the compressive strength of the concrete (Shober, 1997).

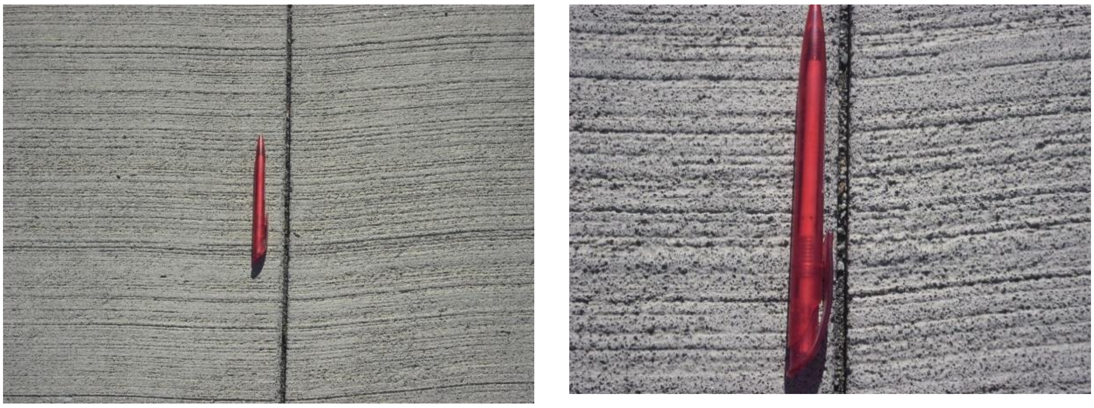


Fig. 2.2.3. Unsealed joints with thin saw-cutting and filled with fine incompressible material (Pradena & Houben, 2014)

In addition, in case of UnJs the fines content of the underlying base/soil layer of the concrete slabs should be limited (≤ 8% passing 75 μm). Hence, the water cannot drag up fines by pumping, which means the base support is retained and no joint faulting will occur for this concept.

As mentioned previously, the UnJs can be applied to traditional or short slabs JPCPs. In any case the evaluation of the UnJs needs to take into account the functional objective of the JPCP application, i.e. accessibility or mobility (FHWA, 2003). Indeed the effects of the UnJs will not be the same in a residential street (accessibility) than in a highway (mobility).

For the evaluation of UnJs it is necessary to include measurements of joint spalling and its potential effect on the functional pavement performance, similar to the investigations of Darter & Barenberg (1976), Al-Omari & Darter (1992), ERES (1995), Yu et al (1998) and Bustos et al (2000) in sealed joints, where they recognize the spalled joints as part of the direct causes of the deterioration of the JPCPs ride quality, i.e. the functional performance of the pavement.

2.2.3. Early-entry saw-cutting of joints

The saw-cutting of the contraction joints is one of the most critical issues of the construction of a JPCP because the pavement needs to be cut before spontaneous shrinkage cracks occur but not so early that it can produce ravelling (Fig. 2.2.4). To make the saw-cuts in time is fundamental, considering that there is only one chance to obtain a pavement without random cracking.

Conventional saw-cutting is made with water-cooled equipment, usually after the concrete final set. The general requirements for the minimum saw-cut depth are more than one-third or one-fourth of the slab thickness (ACPA, 1991; AASHTO, 1993; NCHRP, 2003; MINVU, 2008).

EESC was introduced to the paving industry in 1988 by a concrete pavement contractor (Concrete Construction, 1988). He was looking for a method to cut the joints shortly after the surface is finished. Such a saw-cut would eliminate the need to return the next day to cut the joints, which can be too late to prevent random cracks from forming (McGovern, 2002). The light equipment allows the saw-cutting to be done as soon as the slab can support the weight of the operator (McGovern, 2002). Hence, EESC is useful to avoid repair costs for random cracking because it relieves internal concrete stresses (Titi & Rasoulian, 2002).

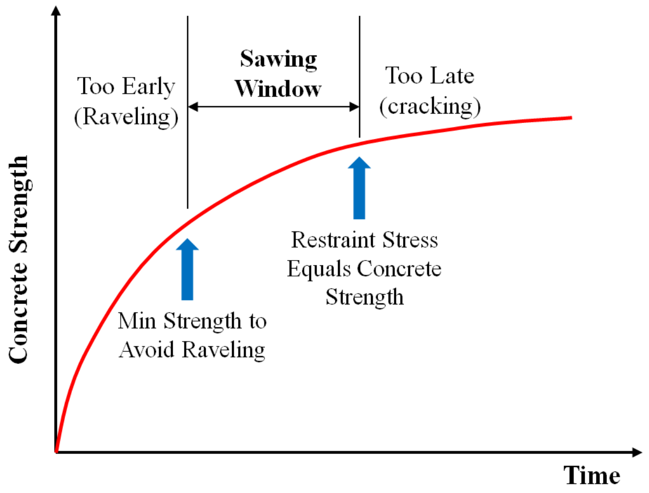


Fig 2.2.4. Concept of sawing window (Okamoto et al., 1994).

Most early-entry saws use a dry-cutting operation with specially designed blades that do not require water for cooling and cut upward as the saw advances (Chojnacki, 2001). It is postulated that the saw-cut can be shallower, i.e. 30 [mm], at early-age taking advantage of the significant changes in moisture and temperature conditions at the surface of the slab to help initiate the crack below the saw-cut (Zollinger et al, 1994).

Although EESC has become more accepted, especially in USA, there are some concerns regarding its influence on the JPCP performance. A report of the Illinois Center for Transportation highlighted the benefits of EESC, but before a general adoption of EESC is implemented, the authors recommended a Phase II study in order to evaluate the JPCP long-term performance in different environmental and climatic conditions. The Phase I study was based on very limited data collected from a single site constructed under very favourable conditions (Krstulovich et al, 20111). In the research of the Louisiana Transportation Research Center, the EESC was producing a very slow joint activation on a JPCP with slab thickness 250 [mm]. Then the joints were sawn deeper (65 mm instead of the original depth of 38 mm) in order to ensure the joint activation (Rasoulian et al, 2005).

The reports of the Illinois Center for Transportation and the Louisiana Transportation Research Center highlight the two main topics for the necessary evaluation of EESC, i.e. the analysis in less favourable conditions (time of construction for instance) and the presence of uncracked joints when only EESC is applied. This is especially valid in short slabs, because the state of the art is focussed on traditional JPCPs.

2.3. MODELLING THE CRACKING PROCESS OF JPCPS

There are several models that, rather than modelling the full cracking process of JPCPs, calculate one part of it, in general the joint opening. One of these models is the formula of the American Association of State Highway and Transportation Officials (AASHTO,

1993) for the joint opening ΔL, caused by temperature change and drying shrinkage as a function of the slab length L (mm).

$$\Delta L = C * L * (\alpha_t * \Delta T + \varepsilon) \quad \text{(mm)} \qquad (2.3.1)$$

Where, ΔL is influenced by the CTE α_t (°C^{-1}); the temperature range ΔT (°C); the drying shrinkage coefficient ε (mm/mm) and the slab-base friction C (-)

Later, the Mechanistic-Empirical Design Guide (MEPDG) used a refined version of the AASHTO formula for the prediction of the joint width jw, caused by temperature change and shrinkage, and expressed as a function of the slab length $JTSpace$ (ft) (NCHRP, 2003).

$$jw = Max(12000 * JTSpace * \beta * (\alpha_{PCC} * (T_{constr} - T_{mean}) + \varepsilon_{sh,mean}), 0) \quad \text{(0.001 in)} \qquad (2.3.2)$$

Where, the CTE is α_{PCC} (in/in/°F); the concrete temperature at set T_{constr} (°F); the mean monthly night-time temperature T_{mean} (°F), the mean shrinkage strain $\varepsilon_{sh,mean}$ (in/in) and the joint open/close coefficient β (-), similar to the coefficient C in Eq. 2.3.1.

As part of the improvements of the South African design method for concrete pavements an equation to predict the crack width under the joints of JPCPs was developed (du Plessis et al, 2007).

$$\Delta x = \begin{bmatrix} \dfrac{C_3}{h} * \left\{ \alpha_1 * \alpha_2 * \alpha_3 * \left(0.019 * \dfrac{w^{2.1}}{f^{0.28}} + 270 \right) + (900 - t) * (t - 0.08)^{0.18} \right\} \\ * (1 - h_u) + (T_0 - T_t) * \eta \end{bmatrix} * L \qquad (2.3.3)$$

Where:

C_3	=	Constant
h	=	Slab thickness (mm)
α_1	=	Coefficient of cement type
α_2	=	Factor of curing
α_3	=	Factor of aggregate type
w	=	Water content of the concrete
f	=	Cylinder compressive strength of the concrete (MPa)
h_u	=	Factor for relative humidity
t	=	Time (years)
T_0	=	Temperature at time of paving (°C)
T_t	=	Present temperature (°C)
η	=	Thermal coefficient of the concrete (°C^{-1})
L	=	Slab length (m)

Searching for a refinement of the model of AASHTO (1993), Roesler & Wang (2008) developed an algorithm for the prediction of the joint opening. For that, they modified the analytical model developed by Zhang & Li (2001) which, originally, only considered the drying shrinkage effect. Roesler & Wang (2008) modified the model in order to incorporate the temperature changes as well.

The described models have at least two characteristics in common:

- They do not consider the viscoelastic behaviour of the concrete since early-age
- They ignore the interaction (in time and space) and the effects of the behaviour of the group of slabs in the prediction of a particular joint opening.

Beom & Lee (2007) state that a cracked joint adjacent to Uncracked Joints (UnCrJ) would show an erratic large joint opening. This conclusion is based on investigations made on JPCPs in USA and Korea, and previous investigations related with the design and performance of joint seals (Lee & Stoffels, 2001; Lee & Stoffels, 2003; Lee, 2003). These studies confirm the earlier conclusions of Morian et al (1999) based on in situ investigations on JPCPs in USA, in the sense that the AASHTO method is not able to adequately represent the joint opening observed in the field. In effect, all the mentioned investigations applied this formula and all the studies conclude that this approach cannot predict adequately the joint opening. But the problem is not exclusively of the AASHTO formula, because the main cause of the larger joint openings observed in the field is the omission of the effects of the UnCrJ in the determination of the joint opening under study.

UnCrJ is not uncommon in JPCPs. Besides the referenced studies, the presence of UnCrJ in real-world JPCPs has been reported by other authors as Zollinger et al (1994) and Pradena & Houben (2016). In particular, Pradena & Houben (2016) identified in the field the presence of UnCrJ even when the traditional technical specifications for the saw-cutting depth had been fulfilled.

The Fig. 2.3.1 is a representation of the effects on the opening of a particular joint as a function of the behaviour of the adjacent joints. The Fig. 2.3.1 (a) shows the joint opening when all the joints are activated. In this case the effective slab length is equal to the designed slab length (L). A larger joint opening should be expected when the adjacent joints are not activated (Fig. 2.3.1 (b)). In this case the effective slab length is the double of the designed slab length, i.e 2L. An even larger joint opening should be expected when the 4 adjacent joints (2 at each side) are not activated (Fig. 2.3.1 (c)). In this case the effective slab length is 3 times the originally designed slab length, i.e 3L.

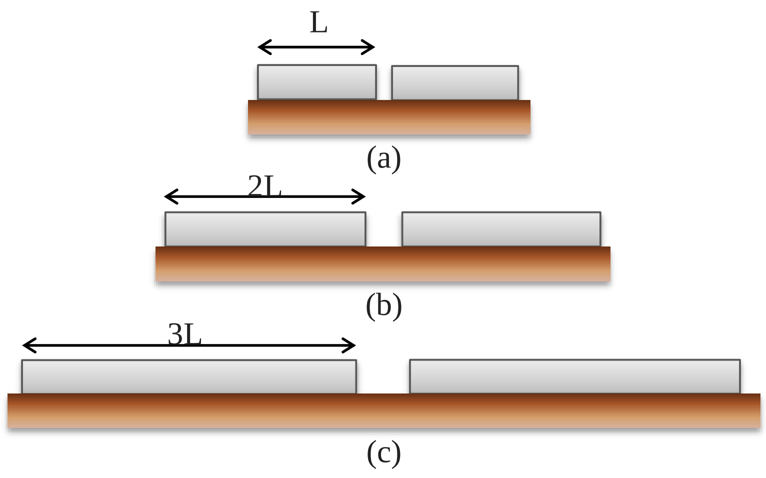


Fig. 2.3.1. Representation of the effects of the JPCP system on a particular joint opening

Moran and Stoffels (1998) and Morian et al (1999) suggest that the in-situ interaction between the joints causing larger joint openings than the calculated one, represents a reasonable explanation of the failure of joint seals. In the particular case of short slabs, Roesler et al (2012) suggests an interaction between the joints with the development of the cracking pattern of the JPCP. According to Roesler et al (2012) this interaction in time affects the Load Transfer Efficiency (LTE) of the non-dowelled short slabs. However this suggested interaction was not studied considering the relationship LTE - crack width, i.e. with the most influential load transfer mechanism in non-dowelled JPCPs, the aggregate interlock (Buch et al, 2000; Hanekom et al, 2003; IPRF, 2011).

The Fig. 2.3.2 represents the effect of the development in time of the cracking pattern of the JPCP over the crack width value under study, in this case the Crack Width of the 1st series of cracks (CW1st), i.e. the joints that are first activated. In Fig. 2.3.2 can be observed that not only the development in time of the UnCrJ has an effect on the CW1st, but also the development of the cracking pattern, i.e. the Crack Width of the 2nd series of cracks (CW2nd), the Crack Width of the 3rd series of cracks (CW3rd) and so on. After the cracking pattern is completed (Crack Width of the 'nth' series of cracks, CWnth), a process that can take several months, a stable value of the CW1st develops, which is the one useful for the link with the in-service JPCP.

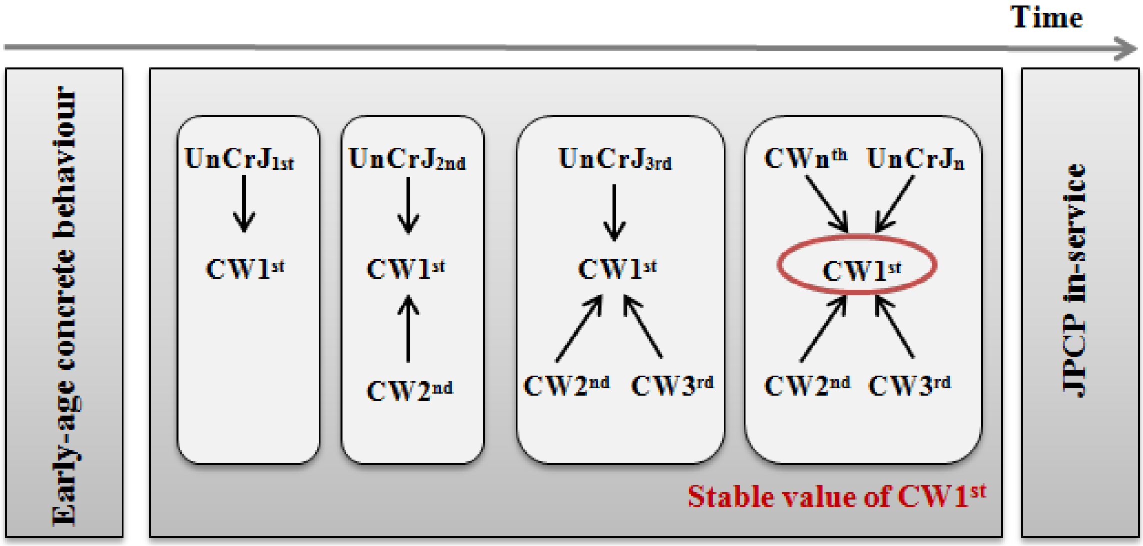


Fig. 2.3.2. Effects of the UnCrJ and the CW of the different series of cracks on the value of CW of the 1st series of cracks.

The investigations of Morian et al (1999), Lee & Stoffels (2001), Lee & Stoffels (2003), Lee (2003) and Beom & Lee (2007) focussed on the analysis of joint seals of in-service JPCPs. In fact, the procedure developed by Lee & Stoffels (2001) is not applicable to new JPCPs. Moreover, Lee & Stoffels (2003) explicitly acknowledge the necessity of having a model to predict the UnCrJ phenomenon and the progress of the transverse cracking since early-age in JPCPs. The system approach of Houben (2008a, 2008b) allows this. In effect, it is possible to model this interaction when the pavement is treated as a system where the modelled length is actually the length required by the cracking pattern itself (a group of slabs) instead of the length of 1 isolated slab. In the system approach of Houben (2008a, 2008b) the particular cracking pattern is developed in time and space according to the specific conditions of the JPCP under analysis. In effect, not only the development in time of the material properties affects the cracking process, but also the geometry of the pavement (thickness, slab length), the time of construction of the JPCP and the saw-cutting method applied (EESC or conventional), between others. Hence, this approach considers all the interactions in the JPCP system for the prediction of a crack width, for instance the CW1st of the Figure 2.3.2.

Another characteristic of the model of Houben (2008a, 2008b) is the consideration of the viscoelastic behaviour of the concrete in its early-age applying an assumed relaxation factor. In the literature there is a lack of experimental data and models based effectively on the relaxation since early-age instead of creep (Atrushi, 2003). In effect, in general researchers have made theoretical studies on self-induced stresses using creep properties for modelling. However, stress relaxation (and not creep) is involved directly in reduction of self-induced stresses in hardening concrete (Atrushi, 2003). Nevertheless, Morimoto and Koyanagi (1995) and Lokhorst (2001) have developed their models based on experimental data of stress relaxation. In the present thesis a comparison of the assumed relaxation factor of Houben (2008a) with these other two models is made.

Houben (2008a, 2008b) uses equations from the standard Eurocode 2 (EN 1992-1-1, 2005) (time-dependent concrete properties and deformations) and assumptions based on engineering judgment. Other authors have developed specific models for each of those assumptions. In the present thesis Houben's assumptions are compared to specific models proposed by other authors for the development of the coefficient of thermal expansion (Bjøntegaard & Sellevold, 2002), the calculation of the hydration temperature using hymostruc software (Van Breugel, 1991), the friction model (Zhan & Li, 2001) and the introduction of the maturity method in the calculations (Arrhenius, 1915).

The system approach of Houben has been applied by Xuan (2012), Mbaraga (2015) and Wu (2015). Xuan (2012) applied the model to analyse the cracking process of cement treated mix granulates with recycled concrete and masonry for use in pavement bases. Similar was the application of Mbaraga (2015) and Wu (2015), but they used it for cement stabilized bases with an additive.

Even when the Houben model gives realistic general trends (Houben, 2008b; 2008c), a formal validation of the model with field data of JPCPs is required. Specifically the validation of the results of the early-age concrete behavior researched in the present thesis (section 2.4).

2.4. EFFECTS OF THE EARLY-AGE CONCRETE BEHAVIOUR ON THE STRUCTURAL AND FUNCTIONAL PERFORMANCE OF JPCPs

2.4.1. Relevant results of the early-age concrete behaviour

From what was presented in the previous section, the relevant results of the early-age concrete behaviour on the in-service JPCP are the crack width (at joints) and the UnCrJ in JPCPs (Fig. 2.4.1). Therefore, they will be incorporated in the current study.

For the purposes of the present research, the validation of the Houben model focusses on the relevant results of the early-age concrete behaviour presented in Fig. 2.4.1. The UnCrJ can be identified in the field by performing visual inspections but the crack width needs to be measured. Related to the measurements of crack width there are several studies performed on JPCPs and continuously reinforced concrete pavements.

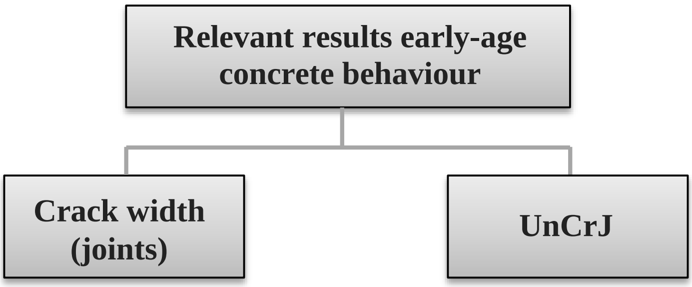


Fig. 2.4.1. Relevant results of the early-age concrete behaviour

In particular, Poblete et al (1988), NHL (1989), Kohler & Roesler (2005, 2006) present detailed crack width measurements and valuable information as the rotation of the crack faces during the day or crack width as a function of depth. Chou et al (2004) performed measurements of the crack width at the middle of the JPCP thickness (at the edge of the pavement) in airport JPCPs, even though these JPCPs are thicker than those in other applications. Chou et al (2004) based this decision on the work of Pitman (1996) who concluded, after a statistical analysis of pavement cores, that the crack widths of the upper half are statistically equal to the ones of the lower half of the cores. On the other hand, Lee & Stoffels (2001) performed measurements on the surface of the joints considering them representative for the variation of the crack width along the joint depth. Their decision to perform the measurements on the surface was based on the maximum difference of 0.15 [mm] reported by Poblete et al (1988) in Chilean undowelled JPCPs, which represents less than 5% of the range of joint movement in most cases. Lee & Stoffels (2001) considered this difference insignificant for the purposes of their research (analysis of the performance of joint seals). What these examples show, is that it is necessary to take into account the purposes of the particular study in order to define an optimal method to measure the crack width at joints of JPCPs (more details are given in Chapter 5).

2.4.2. Pavement clients and their priorities

The largest groups of pavement clients (users and owners or agencies acting on their behalf) assign priority to the functional condition of the pavement (Haas & Hudson, 1996). In addition, the agencies acting on behalf of the owners assign priority to the structural adequacy of the pavement as well (Haas & Hudson, 1996). The Figure 2.4.2 summarizes the pavement clients' priorities, in particular the ones of the public agencies, for instance highway agencies, departments of transportation and municipal transportation agencies.

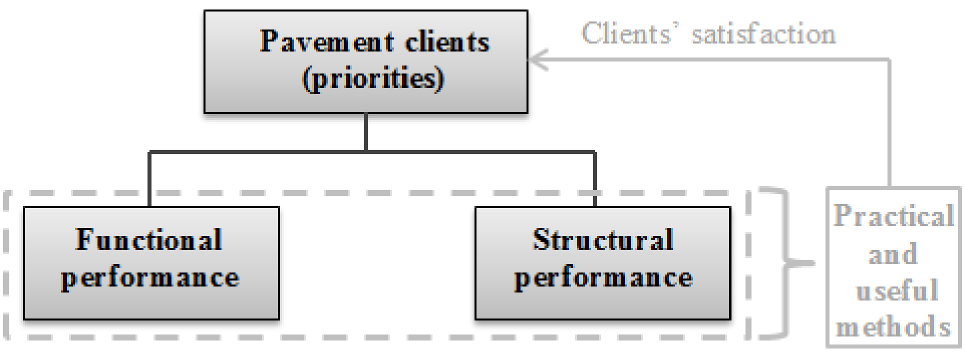


Fig. 2.4.2. Pavement clients' priorities and approach to improve their satisfaction

As mentioned in Chapter 1 the present thesis searches for the pavement clients' satisfaction. One of the largest groups of pavement clients are the pavements owners, or agencies acting on their behalf (as transportation agencies) (Haas & Hudson, 1996). In order to contribute to these pavement clients' satisfaction it is considered necessary to concentrate the structural and functional analysis on practical and useful methods for the pavement clients. For instance, Finite Elements (FE) is an important tool for structural pavement analysis but it cannot easily be implemented as part of a practical design method due to the complexity, computational requirements and time of execution. In the case of the functional evaluation it is necessary to implement a practical and useful formal decision-making framework to support the decisions that the owners or the agencies need to make (but neglecting the functional evaluation).

2.4.3. Structural performance of JPCPs

2.4.3.1. Crack width (at joints) - LTE

As stated before, the crack width is directly related to the LTE of the in-service JPCP. This influence is even more evident in non-dowelled JPCPs such as the short slabs that are the focus of the present research.

The Westergaard equation for slab edge stress due to an equivalent tire load includes different limiting assumptions that differ from real-world concrete slabs. In order to address these limitations, Ioannides (1984) outlined a methodology based on the application of the principles of dimensional analysis. This methodology has been incorporated into different M-E design procedures (Ioannides, 1990, 2006; Ioannides & Salsilli, 1989; Salsilli, 1991; NCHRP, 1992; Cabrera, 1998; Khazanovich et al. 2004; Bordelon et al, 2009, Salsilli et al, 2015; Salsilli, 2015). In particular, to overcome the Westergard assumption of an isolated slab and to include the LTE, Ioannides et al. (1985); Ioannides & Korovesis (1990); Salsilli (1991); Ioannides et al (1996); and Cabrera (1998) use the Eq. 2.4.1 as adjustment factor.

$$F_{LTE} = 0.996573 + 0.00439187 * LTE - 0.00005851 * (LTE)^2 - 0.003124 * \left(\frac{a}{l}\right) * LTE - 0.03682 * ln\left(LTE\right) \tag{2.4.1}$$

Where:

a = radius if circular tire contact area (mm)

l = radius of relative stiffness of the concrete slab (mm)

Other M-E methods for the design of traditional JPCPs not necessarily use the Eq. 2.4.1 but they also include the LTE to adjust their calculations to the reality of the load transfer between slabs in JPCPs. Examples of such methods can be found in the Netherlands (Houben, 2006), Sweden (Söderqvist, 2006), USA (Hiller, 2007), Chile (Chilean Highway Agency, 2012) between others. However, these methods or the ones using the Eq. 2.4.1 do not include a direct relation with the most influential load transfer mechanism in non-dowelled JPCPs, i.e. the aggregate interlock (and then the crack width under the joints). The exception is the MEPDG (NCHRP, 2003) that more than a practical pavement design method is a detailed procedure of structural pavement analysis. Still MEPDG does not consider the viscoelastic behaviour of the concrete since early-age in the development of the cracking process. In addition MEPDG ignores the interaction (in time and space) and the effects of the behaviour of the group of joints in the prediction of the crack width to be related with the LTE.

In the case of short slabs, studies have been focussed on the structural analysis of this non-dowelled JPCP (Salsilli & Wahr, 2010; Covarrubias, 2009, 2011, 2012; Salgado, 2011; Roesler et al, 2012; Chilean Highway Agency, 2012; Salsilli et al, 2015). In particular, Salsilli et al (2013, 2015) developed a practical M-E design method for short slabs using dimensional analysis (Ioannides, 1984). Salsilli et al (2013, 2015) adapted the adjustment factor for load transfer originally developed for traditional JPCPs (Eq. 2.4.1) to make it applicable to short slabs. In addition, the Chilean Highway Agency (2012) developed a method for the structural design of short slabs. In this method values of LTE are recommended to adjust the calculations (Chilean Highway Agency, 2012). However, also these structural design methods for non-dowelled short slabs do not include a direct relation with the most influential load transfer mechanism in non-dowelled JPCPs, i.e. the aggregate interlock (and then the crack width under the joints). The approach of Covarrubias (2008, 2011, 2012) is similar to MEPDG, i.e. it does not consider the viscoelastic behaviour of the early-age concrete and it ignores the interaction (in time and space) and the effects of the behaviour of the group of joints in the prediction of the crack width to be related with the LTE.

For short concrete slabs, Salsilli et al (2015) acknowledge the necessity to make specific studies of the LTE in this innovation. Although Roesler et al (2012) performed measurements of LTE in non-dowelled short slabs, they did not study the relationship LTE - crack width, i.e. including the most influential load transfer mechanism in non-dowelled JPCPs, the aggregate interlock (Buch et al, 2000; Hanekom et al, 2003; IPRF, 2011).

Davids & Mahoney (1999) validated the model of the FE software EverFE that relates the crack width (at joints) with the LTE for non-dowelled JPCPs. In the validation they used the laboratory data of Colley & Humphrey (1967) which represents typical aggregates from the USA. Jensen (2001) also studied the relationship LTE-crack width specially focused on the influence of coarse aggregates typical for USA. Later, Hanekom et al (2003) compared the relationships LTE-crack width given by Jensen (2001) and the FE software EverFE (Davids & Mahoney, 1999) with the ones obtained with typical South African aggregates (Hanekom et al, 2003). From this comparison they found significant differences in the LTE by aggregate interlock provided by the American aggregates and the South African ones. Hence, it is necessary to take into account the differences in the aggregates to evaluate the relation crack width-LTE in non-dowelled JPCPs.

2.4.3.2. Uncracked Joints

The UnCrJ not only affects the crack width and through that the LTE, but it also determines the effective slab length of the in-service JPCP. This effective slab length will be different from the designed slab length in the structural JPCP design if UnCrJ are present. The presence of UnCrJ is particularly problematic in the case of short slabs where the postulated benefits, i.e. thinner slabs, are incorporated in the design. The Fig. 2.4.3 represents a section of short slabs where 50% of the joints remain uncracked. In this case the effective slab length is the one of a traditional JPCP, but with 70 to 100 mm less thickness (Roesler et al., 2012) because it was designed as a shorter 2 m slab.

As mentioned before, previous researchers have been studied the UnCrJ phenomenon (Lee & Stoffels, 2001; Lee & Stoffels, 2003; Lee, 2003; Beom & Lee, 2007). However, these investigations have been focused on the design and performance of joint seals but not on the effects of the UnCrJ on the structural and functional performance of JPCPs.

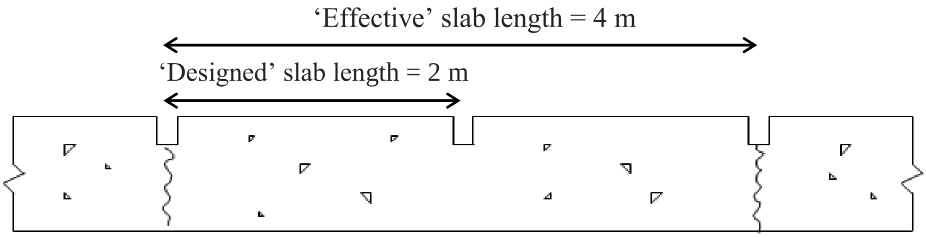


Fig. 2.4.3. Example of section of short slabs with uncracked joint.

2.4.4. Functional performance of JPCPs

2.4.4.1. Necessity of an integral evaluation

As has been mentioned, the largest groups of pavement clients, i.e. users and agencies acting on behalf of the owners, assign priority to the functional pavement performance. Accordingly, an integral evaluation of pavements needs to include this aspect.

In pavement engineering, functional performance is defined as a pavement's ability to provide a safe and smooth riding surface. These attributes are typically measured in terms of skid resistance and ride quality (Caltrans, 2003). According to Molenaar (2001) the functional pavement condition is especially of importance to the road user and is related, among others, to longitudinal evenness. Thenoux & Gaete (1995) expand the definition of functional pavement performance from the surface quality to the general condition of the pavement, taking into account all the factors that negatively affect the serviceability, safety and user costs. In particular, pavement roughness is strongly related to pavement performance providing a good, overall measure of pavement condition that correlates well with subjective assessments of the users (Loizos & Plati, 2008; Papageorgioua & Mouratidis, 2014).

The short slabs are an innovation of the traditional JPCPs. Hence, the new design configuration needs to be compared with the traditional one in order to evaluate if effectively it represents an improvement in the design that positively affects the field performance (Montgomery, 2012). In effect, usually deterioration models are used for the functional evaluation of pavements. As the available deterioration models were originally developed for traditional JPCPs it is necessary to study their applicability to short slabs. Nevertheless, it is also necessary to consider complementary methods for the functional evaluation of the innovative short slabs because, even if the deteriorations models are applicable to short slabs, they have not been calibrated yet.

2.4.4.2. Applicability of deterioration models

The major contributor to the JPCP roughness is JF (Bustos et al, 2000; Jung et al, 2011). There are different deteriorations models to predict JF in traditional JPCPs (Wu et al, 1993; Simpson et al, 1994; ERES, 1995; Owusu-Antwi et al, 1997; Yu et al, 1998; Titus-Glover et al, 1999; Hoerner et al, 2000). However, it is necessary to take into account some factors for the potential applicability of these models to the case of short slabs. A model applicable to short slabs should fulfil the following requirements:

- Have a mechanistic component, i.e. to incorporate relevant variables that influence the development of joint faulting.
- Directly incorporate (and be sensitive to) relevant variables for short slabs (for instance, slab length)
- Allow the evaluation of non-dowelled JPCPs
- Has demonstrated to yield suitable predictions of joint faulting in traditional JPCPs (especially at the geographical locations of the short slabs projects) in order to have a well-established base of comparison.
- The variables required need to be available in projects of short slabs with joint faulting information (in order to compare the model results with the real-world behaviour).

The last factor is necessary because the short slabs with information available of JF are limited to a few projects reported by Roesler et al (2012) and Salsilli et al (2015).

2.4.4.3. MCDM method to evaluate the functional performance

As the short slabs are a recent development where the studies have been focussed on the structural behaviour, there is not enough data for the calibration of the deterioration models, provided that the ones developed for traditional JPCPs are applicable. Considering the importance that pavement clients assign to the functional performance of the pavements, it is necessary to have a formal decision-making framework to support the decisions that the owners or the agencies need to make (but without an actual functional evaluation). The Multi-Criteria Decision-Making (MCDM) method provides a transparent decision-making frame to take decisions in an explicit, traceable and well-sustained way. This improves the transparency of the decisions, instead of overlooking the functional pavement performance in the evaluation of short slabs.

The MCDM methods can be used together with deterioration models (Cafiso et al, 2002; Brownlee et al, 2007) and their application can be based on empirical evidence reducing the subjectivity (Saaty, 1990; Bodin & Gass, 2004; Brownlee et al, 2007; Alexander, 2012). In effect, decision-making has become a mathematical science today (Figuera et al, 2005).

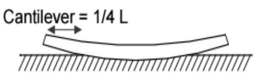
In the particular case of pavement-related decisions, MCDM methods have been used extensively (Azis, 1990; Masami, 1995; Kim & Bernardin, 2002; Cafiso et al, 2002; van Leest et al, 2006; Brownlee et al, 2007; Selih et al, 2008; Sharma et al, 2008; Selih et al, 2008; Farhan & Fwa, 2009; Sun & Gu, 2011; Kabir et al, 2014). In particular, Kabir et al (2014) made a classification of the MCDM methods used in infrastructure management between 1980 and 2012. In the particular cases of roads and pavements they found the Analytic Hierarchy Process (AHP) method (Saaty, 1980) the most used by the scientific community. And according to Cafiso et al (2002) the AHP method lends itself for application in road-related decisions.

2.4.4.4. Effects of the slab curvature on the ride quality

The slabs of JPCPs can be affected by curling and warping that are deviations of concrete slabs from their original shape. Curling is caused by differences in temperature between the top and bottom slab surfaces, and the cause of warping is the moisture difference between these two slab surfaces (Hatt, 1925; Hveem, 1951). Especially when the slab curvature is high, traditional JPCPs may not be the most economical pavement system to construct because of the required slab thickness (ARA, 2007; Hiller, 2007; Hiller & Roesler, 2005). The slab curvature is not only unfavourable from the structural point of view but also from the functional one. In effect different investigations have evaluated the effect of slab curvature on the ride quality of JPCPs (Karamihas et al., 2001; Byrum, 2005; Siddique & Hossain, 2005; Chang et al, 2008; Johnson et al., 2010; Karamihas & Senn, 2012). All studies showed that slab curvature has a decided negative effect on ride quality. However, all these investigations have evaluated traditional JPCPs, but not short slabs.

Byrum (2005) establishes that for the same magnitude of slab curvature longer slab lengths result in more joint uplift and steeper slopes at joints. Thus, if the slabs are shorter the joint uplift is less and the slopes at joints are less steep. The Fig. 2.4.4 represents a comparative example of the portion of the slab length in cantilever for a traditional slab and for a short slab. The differences between traditional and short slabs JPCPs represented in Fig. 2.4.4 will produce an effect in the ride quality. In fact, Covarrubias (2011) reports values of the International Roughness Index, IRI $\leq$ 2.3 [m/km] in 4 projects of short slabs after 9.000.000 ESALs (average). This is an excellent IRI considering that the average initial IRI value of these projects was 2.0 [m/km].

Length traditional slab = 4.0 m
Cantilever = 1.0 m



Length short slab = 2.0 m
Cantilever = 0.5 m

Fig. 2.4.4. Differences on slab curvature between traditional and short slabs JPCPs.

However the slab concavity not only depends on the permanent component (built-in for instance) but also on the transient one, depending basically on the changes of the slab temperature (Lowrie & Nowlen, 1960; Poblete et al, 1988). Because of this transient component the users can perceive a different ride quality at different times of the day, for instance in the morning, when most of the users can use the pavement to commute to their daily activities and a general upward concavity is present (Poblete et al, 1988). These potential differences in the ride quality also present a challenge for the transportation agencies when they need to decide about the reception of new JPCPs or rehabilitation works. The effect of the transient curling depends on local climate conditions. When comparing the variation of the ride quality provided by traditional and short slab JPCPs the transient effect should also be taken into account. In effect, a high concavity of the slabs has been reported at different regions due to the local climate conditions (Hveem, 1951; Larrain, 1985; Poblete et al, 1988; Eisenmann & Leykauf, 1990; Bustos et al, 1998; Yu et al, 1998; Hansen et al, 2000; Vandenbossche 2003a, 2003b; Rao & Roesler, 2005; Rojas-

Torrico, 2008). In particular Poblete et al (1988) and Poblete et al (1990) summarize the results of a comprehensive study in Chile regarding to curling and warping at different climatic conditions (21 test sections located between the latitudes 33°2′S and 40°10′S). The work of Poblete et al (1988) has been part of the baseline investigations for different research studies (ERES, 2001; Chen et al, 2002; Chou et al, 2004; Hiller & Roesler, 2005; Rao & Roesler, 2005; Kohler & Roesler, 2005; Wells et al. 2006; Vandenbossche & Wells, 2006; Kim, 2006; Bordelon et al, 2009; Lederle & Hiller, 2011; Lederle et al, 2011; Roesler et al, 2012) and it is the base of the present study as well, not only for the valuable and recognized information of the study itself but also because the investigations of Poblete et al (1988) and Poblete et al (1990) were developed in Chile where nowadays there is a concentration of projects of the innovative short slabs JPCPs (more details are given in Chapter 7).

2.4.4.5. Crack width (at joints) and UnCrJ on the functional performance

According to Ioannides et al. (1990) the prevention of JF of non-dowelled JPCPs relies on effective load transfer through aggregate interlock. According to this, the crack width at joints should have an important role in the modelling of the joint faulting of the (non-dowelled) short slabs. SHRP 2 (2011) and NCHRP (2012) also acknowledge the importance of the load transfer in the production of JF. In addition, Darter & Barenberg (1976), Al-Omari & Darter (1992), ERES (1995), Yu et al (1997) and Bustos et al (2000) also recognize the transversal cracking as a factor of deterioration of the JPCPs ride quality. The slab transversal cracking is the product of the analysis of the structural performance of JPCPs (section 2.4.2) that in the present thesis is focussed on the relation between the crack width (at joints) and the presence of UnCrJ. Finally, the presence of UnCrJ jeopardize the potential benefits over the ride quality of short slabs (due to the less slab curvature). In effect, when joints remain uncracked, the effective slab length is not short anymore.

2.5. CONCLUSIONS

The focus of the present chapter has been the analysis of the related literature, identifying the necessities associated with the objectives of the present research.

In particular, as the studies of short slabs have been focussed on the structural performance, while clients tend to value functional performance, an integral analysis including the functional performance is needed. As short slabs include unsealed joints, a functional evaluation of the unsealed joints is required as part of this study (this topic is treated in Chapter 7). In addition, although there are some studies regarding the UnCrJ phenomenon, they have been focused on the analysis of joint seals. In this way, it is necessary to analyse the potential effects of UnCrJ on the functional (and structural) performance of short slab JPCPs.

The models available for predicting joint opening in JPCPs ignore the interaction (in time and space) of the group of joints in a particular joint opening. This has caused significant differences between the model predictions and the values of joint opening observed in real-world JPCPs. To assess this part of JPCP performance, an analysis that takes into account the interaction of the group of joints will be used. In chapters 3,4 and 5 this is addressed by using the system approach of Houben, which models the interaction between the joints of JPCPs. This model allows for a cracking pattern not only depending on the development in

time of the material properties, but also of the geometry of the pavement (thickness, slab length), the time of construction of the JPCP, the saw-cutting method applied (EESC or conventional), between other variables. In this way it is possible to predict a particular crack width (for instance the one of the 1st series of cracks) or the presence of uncracked joints in the JPCP.
In addition, joint activation is crucial for short slabs because the shorter design leads to thinner structures. If uncracked joints are present, the actual slab length is larger than the design slab length. Since the saw-cutting method affects the joint activation, its effectiveness must be evaluated. The system approach of Houben allows evaluating the effectiveness of the saw-cut method.

Another aspect of structural performance that is crucial especially for non-dowelled short slabs, is the most influential load transfer mechanism in non-dowelled JPCPs, i.e. the aggregate interlock (and then the crack width at joints). Also, the viscoelastic behaviour of early-age concrete and its effects on the development of the crack pattern needs to be taken into account.

To take all these inter-related influences into account, first it is necessary to study the applicability of the existing functional evaluation methods developed originally for traditional JPCPs, incorporating evaluation methods such as the Analytic Hierarchy Process as well.

As the present thesis searches for the pavement clients' satisfaction, the structural and functional analyses are focussed on practical and useful methods for the pavement clients and the models and assumptions are validated based on elaborate field investigations. In a similar way, the model of the early-age concrete behaviour is also validated in the field focused in the relevant results for the link with the structural and functional in-service performance of JPCPs, i.e. the AvCW1st and the UnCrJ.

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3. MODELLING THE CRACKING PROCESS OF JPCPs

3.1. INTRODUCTION

As mentioned in Chapter 2, the starting point used in this thesis to calculate the relevant results of the early-age concrete behaviour of JPCPs (i.e. AvCW1st and UnCrJ) is the model developed by Houben (2008a, 2008b). Fig. 3.1.1 presents the independent variables considered in the factorial design of Houben's model. Fig. 3.1.1 shows that the modelling is done according to Eurocode 2 (EN 1992-1-1, 2005) and assumptions based on engineering judgment (Houben, 2008a). The present chapter introduces the time-dependent concrete properties of Eurocode 2 (EN 1992-1-1, 2005), the assumptions and the cracking process according to Houben (2008a, 2008b). In addition a comparison of these assumptions with others models is carried out with the objective to identify if significant differences can be produced in the relevant results sought for the present research.

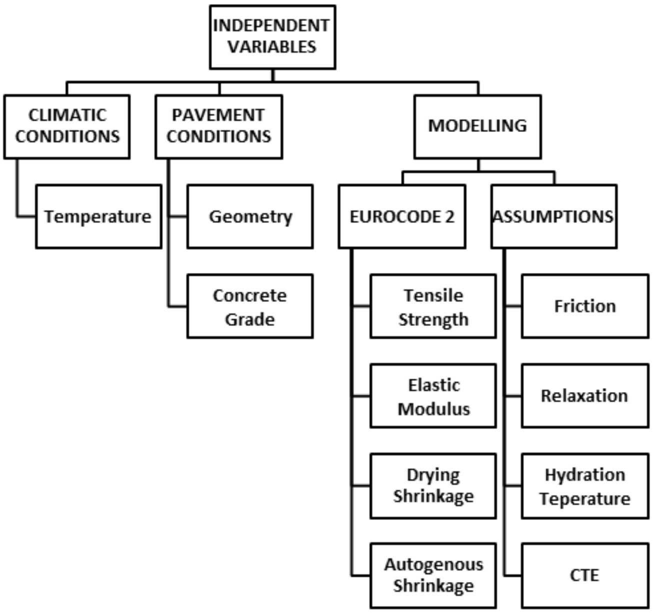

Fig. 3.1.1. Factorial of the Houben model

The factorial of Fig. 3.1.1 describes the modelling of the transverse and longitudinal cracking process of JPCPs. However, the present thesis puts emphasis on the transversal cracking because the crack width of those joints is the one directly related to the Load Transfer Efficiency (LTE) associated to the main direction of the traffic. Accordingly, in the present chapter the model for the transverse cracking is described. The model of the longitudinal cracking is basically the same as the one presented in this chapter (sections 3.2 to 3.4), with small differences in the development of cracks (section 3.5). These differences are presented in Houben (2011,2014).

3.2. TIME-DEPENDENT CONCRETE PROPERTIES

With the exception of the coefficient of thermal expansion, Houben (2008a, 2008b) uses the time-dependent concrete properties from the standard Eurocode 2 (EN 1992-1-1, 2005). In the present section a summary of it is presented. More details can be found in Eurocode 2 (EN 1992-1-1, 2005) and Houben (2008a, 2008b).

3.2.1. Basic strength properties

In the Eurocode 2 (EN 1992-1-1, 2005) the compressive strength of concrete is expressed as the characteristic compressive strength (probability of exceeding 95%) after 28 days of a cylinder or a cube (Eq. 3.2.1 to 3.2.4).

cylinder:	f_{ck}	= 28 MPa (concrete grade C28/35)	(3.2.1)
cube:	$f_{ck,cube}$	= 35 MPa (concrete grade C28/35)	(3.2.2)
cylinder:	f_{ck}	= 35 MPa (concrete grade C35/45)	(3.2.3)
cube:	$f_{ck,cube}$	= 45 MPa (concrete grade C35/45)	(3.2.4)

The average compressive strength after 28 days is 8 MPa higher than the characteristic compressive strength (Eq. 3.2.5 to 3.2.8).

cylinder:	f_{cm}	= 36 MPa (concrete grade C28/35)	(3.2.5)
cube:	$f_{cm,cube}$	= 43 MPa (concrete grade C28/35)	(3.2.6)
cylinder:	f_{cm}	= 43 MPa (concrete grade C35/45)	(3.2.7)
cube:	$f_{cm,cube}$	= 53 MPa (concrete grade C35/45)	(3.2.8)

3.2.2. Drying shrinkage

The drying starts t_s hours after construction, i.e. after the curing period. For t_s a value of 48 hours (2 days) is taken. The drying shrinkage is also dependent on a 'shape factor' h_0 of the structure (Eq. 3.2.9).

$$h_0 = 2 * \frac{A_c}{u} \quad \text{(mm)} \qquad (3.2.9)$$

Where:

A_c = cross section of the structure; for a plain concrete pavement structure $A_c = b*h$

u = the structure's circumference subjected to drying; for a pavement structure $u=b+2*h$

The thickness of a JPCP is around 250 [mm] and the width of the pavement is usually between 5 [m] and 15 [m]. For these thicknesses the value of h_0 varies from 455 [mm] to 484 [mm]. Calculations are done with h_0 = 475 [mm] as used by Houben (2008a, 2008b, 2010).

The time dependency of the drying out of concrete is calculated by means of the Eq. 3.2.10.

$$\beta_{ds}\left(t,t_s\right)=\frac{\left(\frac{t}{24}-\frac{t_s}{24}\right)}{\left(\frac{t}{24}-\frac{t_s}{24}+0.04*h_0^{1.5}\right)} \qquad (3.2.10)$$

Where the time t and the start t_s of the drying out are expressed in hours.

The drying shrinkage $\varepsilon_{cd}(t)$ is calculated by means of the Eq. 3.2.11.

$$\varepsilon_{cd}(t)=\beta_{ds}(t)*\varepsilon_{cd,\infty}=\beta_{ds}(t)*k_h*\varepsilon_{cd,0} \qquad (3.2.11)$$

Where:

$\varepsilon_{cd,0}$	=	0.276‰ (for concrete grade C28/35 in outside air, with a relative air humidity of 80%)
$\varepsilon_{cd,0}$	=	0.252‰ (for concrete grade C35/45 in outside air, with a relative air humidity of 80%)
k_h	=	coefficient depending of the ‘shape factor’ h_0. According to Eurocode 2 (EN 1992-1-1, 2005) k_h = 0.706 for h_0 = 475 [mm] (Houben, 2008a, 2008b).

3.2.3. Autogenous shrinkage

The time dependency of the autogenous shrinkage of concrete is calculated by means of Eq. 3.2.12.

$$\beta_{as}(t)=1-e^{-0.2*\sqrt{\frac{t}{24}}} \qquad (3.2.12)$$

Where the time t is again expressed in hours.

The autogenous shrinkage of concrete at t = ∞ is described by Eq. 3.2.13.

$$\varepsilon_{ca}(\infty)=2.5*(f_{ck}-10)*10^{-6} \qquad (3.2.13)$$

For concrete grade C28/35, with f_{ck} = 28 MPa, $\varepsilon_{ca(\infty)}$ = 0.000045.
For concrete grade C35/45, with f_{ck} = 35 MPa, $\varepsilon_{ca(\infty)}$ = 0.0000625.

The time-dependent autogenous shrinkage $\varepsilon_{ca}(t)$ is calculated by Eq. 3.2.14.

$$\varepsilon_{ca}(t)=\beta_{as}\left(t\right)*\varepsilon_{ca}(\infty) \qquad (3.2.14)$$

3.2.4. Total physical and chemical shrinkage

The total physical and chemical $\varepsilon_{ch}(t)$ of the JPCP is equal to the sum of the drying shrinkage and the autogenous shrinkage (Eq. 3.2.15).

$$\varepsilon_{ch}(t)=\varepsilon_{cd}(t)+\varepsilon_{ca}(t) \qquad (3.2.15)$$

3.2.5. Thermal deformation

In general the standards and recommendations include a constant value for the Coefficient of Thermal Expansion (*CTE*) of hardened concrete. For instance, in the Eurocode 2 (EN 1992-1-1, 2005) the *CTE*-value $10*10^{-6}$ $°C^{-1}$ is advised. However, the *CTE* is not necessarily constant in fresh concrete. Houben (2008a) modelled the development of the CTE as function of the modulus of elasticity $E_{cm}(t)$ as follows:

$$CTE(t) = 3.095*10^{-10}*E_{cm}(t) \qquad (°C^{-1}) \qquad (3.2.16)$$

The thermal deformation $\varepsilon_T(t)$ is expressed by means of Eq. 3.2.17.

$$\varepsilon_T(t) = -CTE(t)*\Delta T(t) \qquad (3.2.17)$$

Where:
ΔT = difference in temperature (°C) between time t and time of construction of the JPCP.

3.2.6. Total deformation

The total deformation $\varepsilon(t)$ of the JPCP is equal to the sum of the physical and chemical shrinkage and the thermal deformation (Eq. 3.2.18).

$$\varepsilon(t) = \varepsilon_{ch}(t) + \varepsilon_T(t) \qquad (3.2.18)$$

As the cracking process of JPCPs is a very complex phenomenom with numerous variable involved, simplifications have been made considering the intended uses of the model, i.e. realistic trends of AvCW1st for the link of the early-age behaviour and the in-service JPCP performance. One of the simplifications is to calculate the tensile stresses associated to the cracking of JPCPs considering only contants deformations over the cross section of the slab, i.e. uniform shrinkage and temperature changes. Although there are temperature and shrinkage gradients present in the slab they have not been considereded in the modeling taking into account that the actual temperature gradient at the bottom of the joint (so at a depth of about 1/3 of the concrete height) is small. In the case of the shrinkage gradient it is assumed that the drying out associated to the 1st series of crakcks is very limited.

3.2.7. Average compressive strength

There is a coefficient s in the equations for the strength that is dependent on the type of cement applied in the JPCP. The cement class N, i.e. normal hardening cement, is used in the modelling (Houben, 2008a). In this way the coefficient s=0.25 (EN 1992-1-1, 2005).

The average compressive strength $f_{cm}(t)$ of concrete is described by means of the Eq. 3.2.19.

$$f_{cm}(t) = \beta_{cc}(t)*f_{cm} = e^{s*\left(1-\sqrt{\frac{672}{t}}\right)}*f_{cm} \qquad (MPa) \qquad (3.2.19)$$

Where:
$\beta_{cc}(t)$ = coefficient depending on the age of the concrete *t*

t	=	time (number of hours) after construction
f_{cm}	=	average compressive strength at 28 days (Equations 3.2.5 and 3.2.7)

3.2.8. Average tensile strength

The average tensile strength f_{ctm} of a cylinder after 28 days is related to the characteristic compressive strength f_{ck} of a cylinder after 28 days (Eq. 3.2.20).

$$f_{ctm} = 0.3 * f_{ck}^{2/3} \qquad \text{(MPa)} \qquad (3.2.20)$$

The average tensile strength $f_{ctm}(t)$ of concrete is described by means of the Eq. 3.2.21.

$$f_{ctm}(t) = \left(\beta_{cc}(t)\right)^{\alpha} * f_{ctm} = \left(e^{s*\left(1-\sqrt{\frac{672}{t}}\right)} \right)^{\alpha} * f_{ctm} \qquad \text{(MPa)} \qquad (3.2.21)$$

Where:
$\alpha = 1$ for $t < 672$ hours (28 days)
$\alpha = 2/3$ for $t \geq 672$ hours (28 days)
t = time in hours

3.2.9. Modulus of elasticity

The average modulus of elasticity E_{cm} after 28 days is related to the average compressive strength f_{cm} of a cylinder after 28 days (Eq. 3.2.22).

$$E_{cm} = 22000 * \left(0.1 * f_{cm}\right)^{0.3} \qquad \text{(MPa)} \qquad (3.2.22)$$

The development of the average modulus of elasticity $E_{cm}(t)$ of concrete is described by means of the Eq. 3.2.23.

$$E_{cm}(t) = \left(\frac{f_{cm}(t)}{f_{cm}} \right)^{0.3} * E_{cm} \qquad \text{(MPa)} \qquad (3.2.23)$$

3.3. CLIMATIC CONDITIONS

The modelling was developed originally by Houben (2008a, 2008b) to obtain trends in agreement with the reality more than exact values. Accordingly, simplifications of the complex phenomenon of cracking in JPCPs have been made. In effect, Houben (2008a, 2008b) modelled the climatic conditions that determine the cracking process of JPCPs basically by the changes of the temperature in the JPCP that produce the thermal deformations (Eq. 3.2.17). The temperature in the concrete slabs is modelled by means of sinusoidal functions that represent the seasonal and daily variation of the temperature in the JPCP.

The following seasonal variation of the temperature of the JPCP is taken into account (referred to the north hemisphere).

- the daily temperature is on average $T_{aveyear}$ °C, and that temperature is present at May 1and at November 1
- the amplitude of the average daily temperature is $T_{ampyear}$ °C
- thus the average daily temperature is minimum ($T_{aveyear}$-$T_{ampyear}$) °C (at February 1) and maximum ($T_{aveyear}$+$T_{ampyear}$) °C (at August 1)

The average temperature T_1 at the day of construction of the JPCP is described by means of Eq. 3.3.1.

$$T_1 = T_{aveyear} + T_{ampyear} * \sin(t_1) \qquad (°C) \qquad (3.3.1)$$

Where:
t_1 = time of construction (number of days after May 1)

Similar to the seasonal variation of the temperature, the daily variation of the temperature of the JPCP is also described by a sinusoidal function in which:

- the daily temperature is average at 10:00 AM and 10:00 PM, maximum at 4:00 PM and minimum at 4:00 AM
- the amplitude of the daily temperature variation is T_{ampday} °C

The temperature T_2 at the hour of construction of the JPCP is described by means of the Eq. 3.3.2.

$$T_2 = T_1 + T_{ampday} * \sin[15*(-10+t_2)] \qquad (°C) \qquad (3.3.2)$$

Where:
t_2 = clock hour (from 0 to 24 hours) of construction at day of construction

So the climate-dependent temperature T_3 of the plain concrete pavement is (Eq. 3.3.3):

$$T_3 = T_2 + T_{ampyear} * \sin\left[\frac{t}{24} + t_1\right] - T_{ampyear} * \sin(t_1) + T_{ampday} * \sin(15*t) \qquad (°C) \qquad (3.3.3)$$

Where:
t = time (number of hours) after construction

Hydration heat occurs after the construction of the JPCP, resulting in a temporary temperature increase T_4. This hydration temperature is described by means of Eq. 3.3.4.

$$T_4(t) = t^2 * e^{(-0.27*t)} \qquad (°C) \qquad (3.3.4)$$

Where the time t is again expressed in hours.

This assumed formulation leads to a maximum temperature increase due to the hydration heat of about 7.5°C after about 7.5 hours.

The actual temperature T_5 of the JPCP is equal to the sum of the climate temperature and the hydration temperature (Eq. 3.3.5).

$$T_5 = T_3 + T_4 \qquad (°C) \qquad (3.3.5)$$

3.4. OCCURRING TENSILE STRESSES IN JPCPs

According to Houben (2008a, 2008b), in the JPCP the occurring tensile stresses as a product of the restricted deformation, follow from Hooke's law, but they are affected by the viscoelastic behaviour of the concrete (relaxation factor) (Eq. 3.4.1).

$$\sigma(t) = g*R*E(t)*\varepsilon(t) \qquad (MPa) \qquad (3.4.1)$$

Where:

$E_{(t)}$	=	time-dependent modulus of elasticity of the concrete (MPa)
$\varepsilon(t)$	=	total time-dependent JPCP tensile strain due to shrinkage and thermal effects (-)
R	=	relaxation factor (-)
g	=	enlargement factor (-)

The greatest tensile stresses occur in the joints of the JPCPs (weakened cross-section). The enlargement factor g represents this condition (Eq. 3.4.2).

$$g = \frac{h}{h - jd} \qquad (3.4.2)$$

Where *h* (mm) is the slab concrete thickness and *jd* (mm) is the saw-cut depth.

The Eq. 3.4.3 presents the relaxation factor assumed by Houben (2008a, 2008b).

$$R(t) = e^{-0.0003*t} \qquad (3.4.3)$$

Where:
t = time (number of hours) after construction

3.5. DEVELOPMENT OF THE CRACKING PROCESS IN JPCPs

3.5.1. Basis of the modelling of the cracking process of JPCPs

In order to model the cracking process at joints in JPCPs, firstly Houben (2008a, 2010) developed the basis of the model for non-weakened plain concrete pavements, i.e. without joints. This is the case of the JPCP before the saw-cutting of the joints or the case of (lean) concrete bases for instance. In the present section a summary of the development of the primary cracks is presented. More details can be found in the work of Houben (2008a, 2010).

The mutual distance between the primary cracks is determined by the so-called breathing length, L_{a1} (Fig. 3.5.1). This is that part of a long structure that exhibits horizontal movements due to temperature changes (or another influencing factor). This phenomenon also occurs for instance on long-welded railway track (Houben, 2008a, 2010). The breathing length is expressed by means of the Eq. 3.5.1.

$$L_{a1}(t) = \frac{E_{cm}(t) * \varepsilon(t)}{\gamma * f} \qquad \text{(m)} \qquad (3.5.1)$$

Where:

$E_{cm}(t)$	=	modulus of elasticity (MPa) of the plain concrete pavement at the moment of the cracks
$\varepsilon(t)$	=	maximum total obstructed deformation (tensile strain) of the plain concrete pavement at the moment of the primary cracks
γ	=	volume weight of the plain concrete pavement (kN/m^3)
f	=	friction between the plain concrete pavement and the underlying layer

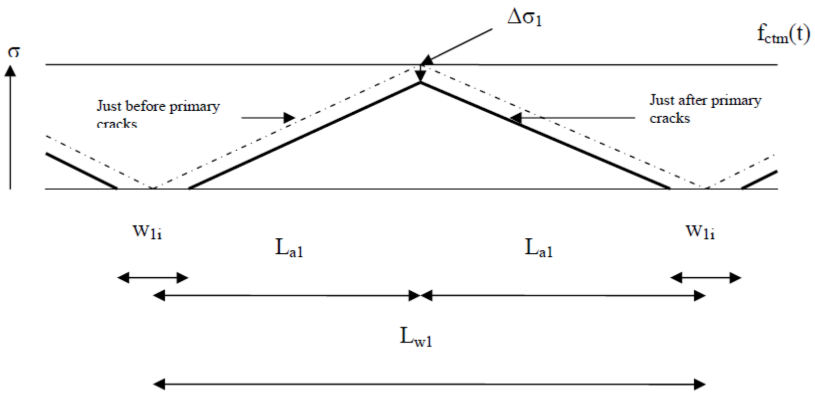

Fig. 3.5.1. Tensile stresses in plain concrete pavement at the time of the primary cracks (Houben, 2008a).

The spacing L_{w1} between 2 primary cracks is equal to 2 times the breathing length.

The initial width of a primary crack w_{1i} can be calculated with the Eq. 3.5.2.

$$w_{1i} = \frac{1000000 * E_{cm}(t) * \varepsilon(t)^2}{\gamma * f} \qquad \text{(mm)} \qquad (3.5.2)$$

Where t = time of occurrence of primary crack

Because of the initial crack width w_{1i}, a reduction $\Delta\sigma_1$ of the maximum tensile stress (midway between 2 primary cracks) occurs (Eq. 3.5.3).

$$\Delta\sigma_1 = \frac{w_{1i} * \sigma(t)}{2 * L_{a1}} \qquad \text{(MPa)} \qquad (3.5.3)$$

Where:

t	=	time of occurrence of primary crack
σ(t)	=	the maximum tensile stress in the plain concrete pavement at the time of occurrence of the primary crack

After the occurrence of the primary cracks the maximum tensile stresses $\sigma_1(t)$ in the layer (midway between 2 subsequent primary cracks) keep on developing (Fig. 3.5.2) due to the ongoing chemical process and the temperature effect (Eq. 3.6.4).

$$\sigma_1(\mathrm{t}) = \sigma(\mathrm{t}) - \Delta\sigma_1 \qquad \text{(MPa)} \qquad (3.5.4)$$

Where:
$\sigma(t)$ = maximum tensile stress without primary cracks
$\Delta\sigma_1$ = reduction of the maximum tensile stress at the time of occurrence of the primary cracks (Eq. 3.5.3)

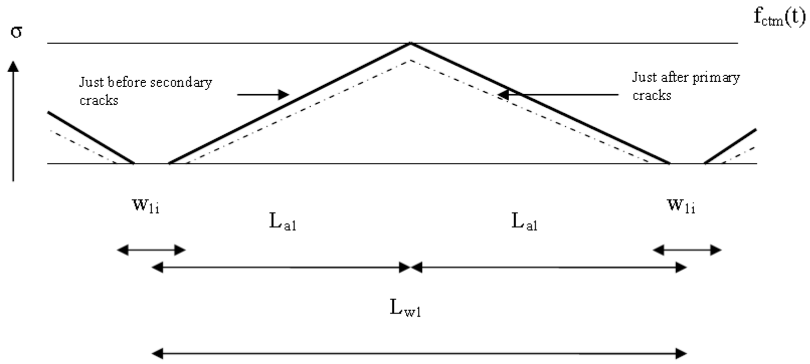

Fig. 3.5.2. Build-up of tensile stresses in the non-jointed plain concrete pavement between the primary and secondary cracks

The maximum tensile strain $\varepsilon_1(t)$ in the layer, midway between 2 primary cracks, follows from Hooke's law (Eq. 3.5.5).

$$\varepsilon_1(\mathrm{t}) = \frac{\sigma_1(\mathrm{t})}{E_{cm}(t)} \qquad (3.5.5)$$

Where:

$\sigma_1(t)$	=	maximum tensile stress (MPa) in the layer, between 2 primary cracks (Eq. 3.5.4)
$E_{cm}(t)$	=	modulus of elasticity (MPa) of the plain concrete pavement

The change of the width of the primary cracks can be expressed by the Eq. 3.5.6.

$$\Delta w_1(\mathrm{t}) = \frac{1000000 * \mathrm{E_{cm}}(\mathrm{t}) * \varepsilon_1(\mathrm{t})^2}{\gamma * \mathrm{f}} \qquad \text{(mm)} \qquad (3.5.6)$$

The development of the width $w_1(t)$ of the primary cracks after the occurrence of the primary cracks is expressed by the Eq. 3.5.7.

$$\mathrm{w_1}(\mathrm{t}) = \mathrm{w}_{1i} + \Delta \mathrm{w_1}(\mathrm{t}) \qquad \text{(mm)} \qquad (3.5.7)$$

Further details of the development of the cracking process in non-weakened plain concrete pavements can be found in the work of Houben (2008a, 2010).

3.5.2. Cracking process in JPCPs as a function of the location of the 1st series of cracks

In the modelling developed by Houben (2008b) the calculation of the initiation and development of the cracking process in JPCPs considers the pavement as a system with pre-defined weakened sections (joints location). In this system the value of the crack width is not only the result of the material changes but also of the location of the 1st series of cracks, the 2nd ones and so on until the cracking process is completed. In particular, it depends of the location of the 1st series of cracks, how the calculation of the cracking process exactly develops. In the present section the case when the 1st series of cracks occur at the location of every 3rd joint is presented as an example. More details can be found in Appendix A and in the reference of the work of Houben (2008b).

The Fig. 3.5.1 shows that when the 1st series of transverse cracks occur every 3rd joint (so at the joints nr. 1, nr. 4, nr. 7, etc.), for reasons of symmetry the possible 2nd series of cracks then occur together in the 2 joints lying in between (nrs. 2 and 3, nrs. 5 and 6, nrs. 8 and 9, etc.).

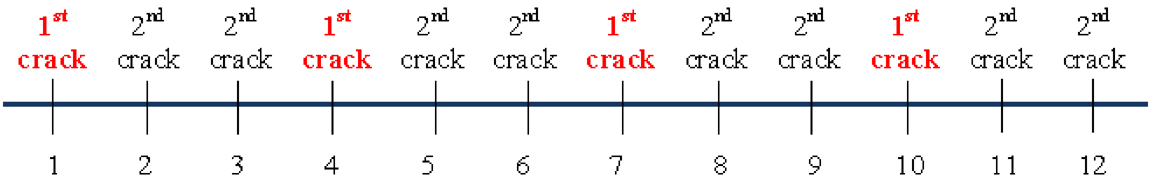

Fig. 3.5.1. Development of the cracking process when the 1st series of cracks occur every 3rd joint.

Taking into account the geometry presented in Fig. 3.5.1, Houben (2008b) developed the following equations for the development of the cracking process of the JPCP.

The stress reduction at the location of the 2nd series of cracks due to the occurrence of the 1st series of cracks becomes (Eq. 3.5.8):

$$\Delta\sigma_{12zs} = 1.333 * 0.5 * \sigma(t) * \left(1 + \frac{w_{12i}}{(1000 * L_{a12})}\right) \quad \text{(MPa)} \qquad (3.5.8)$$

The breathing length L_{a12} is now equal to 1.5 times the slab length.

The tensile stress at the location of the 2nd series of cracks due to the occurrence of the 1st series of cracks becomes (Eq. 3.5.9):

$$\sigma_{2zs}(t) = \left(2/3\right) * \sigma_{zs}(t) - \Delta\sigma_{12zs} + \left(1/3\right) * \sigma_{zs(t=ocurrence1^{st}\,cracks)} \quad \text{(MPa)} \qquad (3.5.9)$$

The Eq. 3.5.10 describes the maximum tensile strain midway between the 1st series of cracks, so not at the location of a 2nd series of cracks but halfway between them.

$$\varepsilon_2(t) = \left(\frac{3}{2}\right) * \left(\frac{\sigma_2(t)}{g * E_{cm}(t)}\right) \tag{3.5.10}$$

In the case that the 2nd series of cracks indeed occur, then the initial crack width of every of these cracks can be represented by the Eq. 3.5.11.

$$w_{2^{nd}\, i} = \frac{0.5 * 1000000 * E_{cm}(t) * \varepsilon_2(t)^2}{\gamma * f} \quad \text{(mm)} \tag{3.5.11}$$

After the possible occurrence of the 2nd series of cracks (all the joints are then cracked through) it should be checked whether or not a transversal crack occurs in the middle of the individual slabs. This calculation goes according to the procedures explained in Houben (2008a, 2010) for non-weakened plain concrete pavements.

3.6. COMPARISON OF MODELS

3.6.1. Intended uses of the model of cracking in JPCPs

As expressed in the introduction, the assumptions made by Houben (2008a) are compared with more sophisticated models with the objective to evaluate if significant differences could be produced in the relevant results required by the present research. With the same objective, the maturity method is included in the calculations of the tensile strength and the elastic modulus.

The cracking process of JPCPs is a complex phenomenon with numerous variables involved. Hence, it is necessary to take into account the intended uses of the model according to the particular objective of the research (Fig. 3.6.1). In effect, Erkens (2002) states that a "good" model depends on what it is used for and how accurate the prediction or calculation should be. The Fig. 3.6.1 represents the intended uses of the model for the present research. In this context a "good" model of the cracking process of JPCPs is the one able to predict the crack width at joints useful for the incorporation of the early-age concrete behavior in the design of non-doweled JPCPs. As the 1st series of cracks are the wider ones, they control the design and then they are useful for the link with the representative LTE of the JPCP in-service. In particular, the Average Crack Width of the 1st series of cracks (AvCW1st) is the value used in the present research to make the link between the early-age concrete behaviour and the in-service JPCP (Fig. 3.6.1).

Fig. 3.6.1. Intended uses of the model in the present research.

3.6.2. Most significant differences: the case of the relaxation factor

Erkens (2002) states that a model is, by definition, a simplification used to explain, calculate or predict something, for example a physical phenomenon. In the model the real situation is simplified by leaving out those aspects that are less important for whatever is being investigated. From this perspective, considering the intended uses of the model (Fig. 3.6.1) and the fact that the Houben model already gives realistic general trends (Houben, 2008b; 2008c), the analyses in the present thesis are focussed on the independent variable with the most significant differences.

Comparisons with models proposed by other authors have been made for the development of the CTE (Bjøntegaard & Sellevold, 2002), the relaxation factor (Morimoto & Koyanagi,1995; Lokhorst, 2001), the calculation of the hydration temperature using hymostruc software (Van Breugel, 1991), the friction model (Zhan & Li, 2001) and the introduction of the maturity method in the calculations (Arrhenius, 1915). With the exception of the relaxation factor, these comparisons do not produce significant changes in the development of the time-dependent concrete properties (Fig. 3.6.2), the tensile stresses or the AvCW1st. Further details can be found in Pradena & Houben (2012, 2015).

For instance, the Fig 3.6.2 shows the development of the tensile strength according to Houben and using the maturity method. This method introduces the concrete temperature during the time interval Δt for the calculations which produces an hourly cyclic variation (Fig. 3.6.2). Still, the behaviour is very similar to the Houben model based in the standard Eurocode 2 (EN 1992-1-1, 2005).

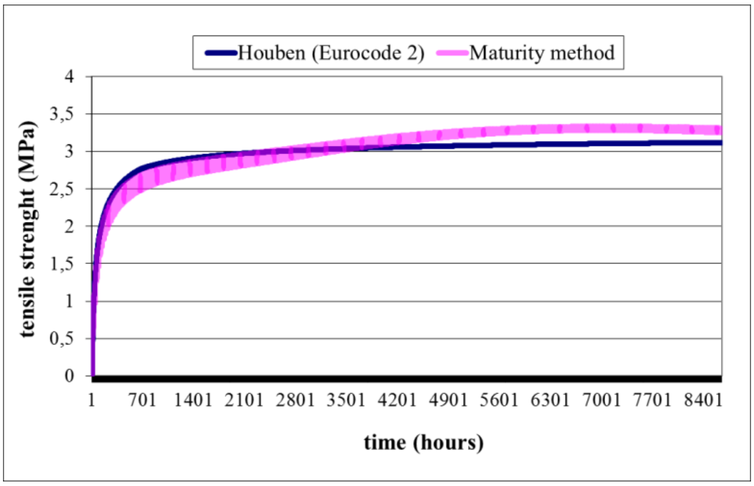

Fig. 3.6.2. Tensile strength according to Houben and the maturity method.

The case of the relaxation factor is different. In general researchers have made theoretical studies on self-induced stresses using creep properties for modelling. However, stress relaxation (and not creep) is involved directly in reduction of self-induced stresses in hardening concrete (Atrushi, 2003). There is a lack of experimental data on stress relaxation since early-ages (Atrushi, 2003). Nevertheless, Morimoto and Koyanagi (1995) and Lokhorst (2001) have developed their models considering experimental data. In effect, Morimoto & Koyanagi (1995) performed laboratory tests and they found an empirical equation for the relaxation as a function of time (t) (Eq. 3.6.1). And Lokhorst (2001) proposes a simplification of Van Breugel's expression of the relaxation (Van Breugel, 1991), as a function of the degree of hydration α(t) and the degree of hydration at loading α(τ) (Eq. 3.6.2).

$$R = \frac{0.32 + 0.85 * t}{0.32 + t} \tag{3.6.1}$$

$$R = \exp\left[-\left(\frac{\alpha(t)}{\alpha(\tau)} - 1 \right) \right] \tag{3.6.2}$$

Where:

α(t) = degree of hydration at age t
α(τ) = degree of hydration at the moment of loading τ

The Fig. 3.6.3 shows the significant differences between the relaxation factors expressed by Eq. 3.6.1, Eq. 3.6.2 and Eq. 3.4.3 (Houben assumption) when they are compared in a period of time useful for the objectives of the present research. The effect on the JPCP behaviour of these differences requires further study. This specific analysis is made in Chapter 5 where the model results are compared with field inspections and measurements.

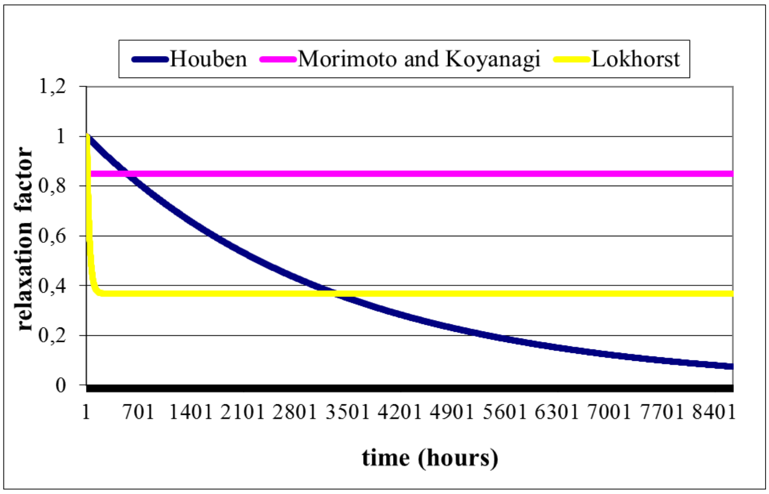

Fig. 3.6.2. Relaxation factors according to Houben (2008a), Morimoto & Koyanagi (1995) and Lokhorst (2001).

3.7. CONCLUSIONS

Houben developed a rational and detailed model to predict the cracking process in JPCPs as a system where the results are not only the product of the material changes but also of the location of the 1st series of cracks, the 2nd ones and so on until the cracking process is completed. This system approach is different to what is available in the state of the art for the challenging prediction of the cracking process in JPCPs. In effect, it is a complex phenomenon with numerous variables involved where Houben made simplifications and assumptions based on engineering judgement. Even though his model gives realistic general trends, in the present section a deeper study of the Houben assumptions has been made taking into account the intended uses of the model and concentrating it on the most significant differences between models. In this way, with the exception of the relaxation factor, no significant differences result when the Houben assumptions are compared with more sophisticated models.

The case of the relaxation factor is special due to the lack of experimental data and models based effectively on the relaxation since early-age instead of creep. In addition, between the few models available (based on the relaxation itself) significant differences exist when they are compared in a period of time useful for the objectives of the present research.

Because what has been mentioned in the previous paragraphs, in the present research the Houben model is applied with the exception of the relaxation factor that requires further study. This specific analysis is made in Chapter 5 where the model results are compared with the behaviour of real-world JPCPs.

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4. FACTORIAL DESIGN AND EVALUATION CONDITION

4.1. FACTORIAL DESIGN

Houben (2010a; 2010b) made a sensitivity analysis using the traditional research method to estimate the effect of One-Factor-At-a-Time (OFAT) on the cracking process of Jointed Plain Concrete Pavements (JPCPs), varying one factor, *ceteris paribus*. This traditional method can reduce the set of variables to be considered in further studies, but factorial designs are most efficient when the study involves two or more factors (Montgomery, 2012). In effect, factorial designs would enable to study the combined effect of the factors on a response (Antony, 2003).

Factorial design is a useful method for streamlining research, allowing the determination of the effects of multiple variables on a response, because factorial design tests all possible conditions. In fact, in a factorial design all the possible combinations of the levels of the factors are investigated, i.e. every level of one factor with all levels of another factor in order to determine the effects of the independent variables on the dependents ones. In this way, a factorial design allows the interpretation of the main effects of the independent variables, but also the interaction effects.

Factorial designs have been used in broad areas of research, for instance:

- Stabilizers in nanoparticles (Vandervoort & Ludwig, 2002)
- Sensitivity analysis of welding processes (Kim et al, 2003)
- Development of an anticancer drug (Derakhshandeh et al, 2007)
- Structural factors of JPCPs (FHWA, 2005)
- Fatigue behaviour of hot mix asphalt (Stubbs, 2011; Tsai et al, 2004)
- Effects of a national road safety strategy (Ayamah, 2011)
- Cracking resistance (Mamlouk & Mobasher, 2004; Saadeh, 2009) and crack propagation in asphalt pavements (Soncim & Fernandez, 2014)
- Durability of wet night visible pavement markings (Gibbons & Williams, 2012)
- Evaluation of input parameters of the Mechanistic-Empirical Pavement Design Guide**,** MEPDG (Cooper et al, 2012)
- Efficiency of curing compounds (De Solminihac, 1987)
- Pavement design alternatives (Crovetti & Owusu-Ababio,1999)
- Effects of different factors on the resilient modulus (Khan et al, 2012)
- Prioritize pavement maintenance and rehabilitation (Thenoux & Halles, 2002; Flintsch et al, 1998)
- Factors which affect rutting (Manik et al, 2007)
- Pavement monitoring (Øverby & Paige-Green, 2010).

Factorial design works better with a small number of variables with few states, when the interactions between the variables are important, and where every variable contributes significantly. Accordingly, a pre-selection of variables is made based on the description of the model and the conclusions of the Houben studies (Houben 2010a; Houben 2010b). This method also has been used to reduce the set of variables that need to be considered in a global sensitivity analysis of MEPDG (Ceylan et al, 2010; Schwartz et al, 2011) or to provide a practical guidance in PCC material property selection for rigid pavement design in view of the numerous inputs required by MEPDG (Ceylan et al, 2010).

Considering the objective, the scope and the research approach of the thesis, the Average Crack Width of the 1st series of cracks (AvCW1st) and the presence of Uncracked Joints (UnCrJ) are the main results sought after the modelling of the concrete behaviour since early-age. In order to determine these values for the conditions of interest for the research, a factorial design is established. The function of this factorial design is the definition of categories of variables, and their values, that represent the scenarios of interest to the determination of the main outputs of the modelling of the concrete behaviour since early-age (AvCW1st and presence of UnCrJ), which are the link with the in-service performance of JPCPs.

4.2. EVALUATION CONDITION

In the non-dowelled short slabs JPCPs the load transfer relies on aggregate interlock which is directly related with the crack width (at joints). In this way, the determination of the AvCW1st is fundamental for the in-service performance of short slabs JPCPs.

According to Montgomery (2012) experimental design methods can play a major role in engineering design activities, where new products are developed and existing ones improved to obtain better field performance and lower product costs. Some applications include the determination of key product design parameters that impact performance, and the evaluation and comparison of design configurations. As short slabs JPCPs are an innovation of traditional non-dowelled JPCPs, the new design configuration needs to be compared with the traditional one in order to evaluate if effectively it represents an improvement in the design that positively affects the field performance (Montgomery, 2012). Accordingly, an objective criterion for comparison needs to be established as evaluation condition.

Considering the importance of the AvCW1st for the link between the early-age behaviour and the in-service performance of non-dowelled JPCPs, the objective criterion used in the evaluation condition is the comparison of the AvCW1st between traditional and short slabs JPCPs (see section 4.5.1 as well).

4.3. DEPENDENT VARIABLES OF THE FACTORIAL DESIGN

According to the objective of the thesis and the research approach, the dependent variables of the factorial design are as follows:

- AvCW1st: Average Crack Width of the 1st series of cracks
- UnCrJ: Uncracked Joints

The AvCW1st 1 year after the pavement construction is especially important for the objectives of the present research. In effect, after 1 year the AvCW1st is more stable (Houben, 2010b) and then it is a representative value for the link between the early-age behaviour and the in-service performance of non-dowelled JPCPs. Accordingly, the calculations and analyses presented in this chapter are focussed on the AvCW1st 1 year after of the pavement construction.

4.4. INDEPENDENT VARIABLES OF THE FACTORIAL DESIGN

To the definition of the factorial design it is necessary to make a selection of factors and their levels. A pre-selection has been made considering the work of Houben (Houben, 2010a; Houben, 2010b) and the description of the model (Chapter 3). Accordingly, the modelling of AvCW1st and presence of UnCrJ depends basically on the season of construction, time of construction during the day, geometry of the JPCP, saw-cutting method and depth (Relative Joint Depth, RJD), concrete grade and friction with the base. The geometry of the JPCP is basically represented by its slab length and thickness. And even more than the thickness itself, by the RJD. As the slab length is part of the evaluation condition (see sections 4.2 and 4.6.1), the Figure 4.4.1 represents the most influential independent variables for the determination of AvCW1st and the presence of UnCrJ.

As it has been mentioned in the previous chapters, the intended use of the modelling of the early-age concrete behaviour in the present thesis is to establish the effects of that early-age behaviour on the in-service performance of JPCPs. From that perspective, there are 2 seasons of construction of interest that decisively affects the dependent variables of the factorial design. Indeed, summer and winter are the most unfavourable conditions for the AvCW1st and UnCrJ respectively. In effect, construction in summer produces the widest cracks (Houben, 2010b) and construction in winter can lead to UnCrJ even when the traditional specifications of RJD have been fulfilled (Pradena & Houben, 2016a).

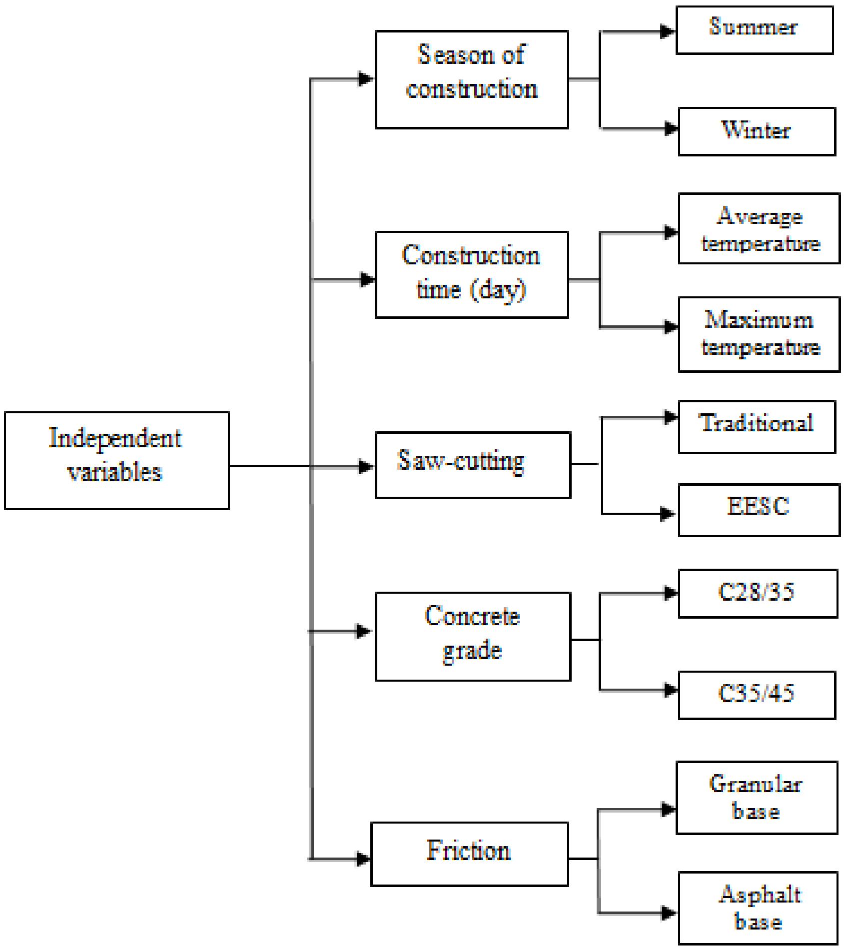


Fig. 4.4.1. Independent variables of the factorial design

4.4.1. Season of construction

The season of construction not only determines the temperature of construction of the JPCP but also the development of its temperature afterwards, that according to Houben is an influence factor for the development of the cracking process in JPCPs (Houben, 2010a; Houben, 2010b). The base model of the present research was originally developed and applied for the transversal cracking process of JPCPs under Dutch climatic conditions (Houben, 2010b). These climatic conditions are in general representative for a moderate sea climate, very similar to the conditions of the JPCPs where the field measurements were done, i.e. in Belgium and some locations in Chile. Hence, these are the general climatic conditions included in the factorial design. The seasonal pavement temperature is assumed to be average at the beginning of May and November, and it is estimated to be maximum in midsummer (beginning of August) and minimum in midwinter at the beginning of February (the months refer to the northern hemisphere) (Houben, 2010b).

4.4.2. Time of construction during the day

The daily pavement temperature is assumed to be average at 10:00 am and 10:00 pm, maximum at 4:00 pm and minimum at 4:00 am (Houben, 2010b). For the study of the cracking process in JPCPs there are two important times of construction during the day, the average conditions and the maximum temperature (Houben, 2010b).

4.4.3. Saw-cutting

In general in traditional JPCPs conventional saw-cuts of one-third of the concrete thickness are applied. Even more, it is possible to make deeper saw-cuts with conventional equipment but using a thin blade (Pradena & Houben, 2015a). On the other hand, Early-Entry Saw-Cutting (EESC) produces a shallow cut up to 30 [mm] depth and in short slabs the thickness of the JPCPs can be reduced with 70 to 100 mm compared to the traditional AASHTO design (Roesler, 2012). The factorial design needs to take into account the saw-cut depths of the EESC and the conventional saw-cutting method when they are applied to traditional and short slabs JPCPs.

4.4.4. Friction

In the modelling of the cracking process in JPCPs, the friction between the concrete pavement and the underlying base is a combination of the coefficient of friction and the dead weight of the concrete slab (Houben, 2010a). The base layers considered in the factorial design are granular and asphalt concrete bases because they are the common types of base applied in JPCPs. In addition, the granular base is critical for joint deterioration and slab failures that affect the in-service pavement performance. In effect, the water can drag fines from the base producing pumping, so less support and finally joint faulting (that is the major contributor of the JPCP's roughness) and cracks in the slab.

4.4.5. Concrete grade

In the factorial design the two most widely used concrete grades are included, i.e. C28/35 and C35/45, for which most of the required (mechanical) properties are available in the standard Eurocode 2 (Houben, 2010a).

4.5. VALUES ASSOCIATED TO THE EVALUATION CONDITION AND THE FACTORIAL VARIABLES

4.5.1. Evaluation condition

Because the reduction of the crack width (at joints) is produced by the slab length reduction (AASHTO, 1993; NCHRP, 2003; Pradena & Houben, 2016b), the values of the evaluation condition are the slab lengths of traditional and short slabs JPCPs.

The basic principle of the short slabs JPCP is the reduction of the slab length, in general 50% of reduction regarding to the slab length of traditional JPCP (Covarrubias, 2011, 2012; Covarrubias et al, 2010). In this way, if the slab length of the traditional JPCP is L, the length of the short slabs is 50% less, i.e. 0.5L. The traditional slab lengths L considered in the analysis are 4.0 [m], 4.5 [m] and 5.0 [m]. In addition, further specific calculations are made in Chapter 5 with other slab lengths.

4.5.2. Season of construction

As mentioned in section 4.4.1, the general climatic conditions considered by Houben (2010b) for the Netherlands are the ones included in the factorial design. For Dutch climatic conditions, Houben (2010b) considered the amplitude of the average daily pavement temperature ($T_{ampyear}$) as 10^{o}C where $T_{ampyear}$ is described by means of a sine-function (as it was presented in Chapter 3). The average daily temperature of the concrete pavement is estimated to be maximum about 25^{o}C midsummer (August 1) and minimum about 5^{o}C midwinter (February 1) (Houben, 2010b).

4.5.3. Time of construction during the day

For Dutch climatic conditions, Houben (2010b) considered the daily pavement temperature amplitude of the concrete pavement (T_{ampday}) as 5^{o}C where T_{ampday} is also described by means of a sine-function that reaches its maximum at 4:00 pm, its minimum at 4:00 am, and its average at 10:00 am and 10:00 pm. For the calculations, the maximum temperature 4:00 pm and the average temperature at 10:00 am are considered not only because they are the ones of interest for the cracking process prediction (section 4.4.2) but also because they are realistic in the practice of construction of JPCPs. In particular, the scenarios of evaluation considered in the simulations are construction of the JPCP at summer 4:00 pm and winter 10:00 am. In order to take into account variations of the climatic conditions further specific calculations are made in Chapter 5 varying the T_{ampday} because significant variations of the climatic conditions are basically reflected in the daily temperature variation of the pavement (Houben, 2010 b).

4.5.4. Saw-cutting (RJD)

The combinations of the saw-cutting method and the thickness of the traditional and short slabs JPCPs determine the different values of Relative Joint Depth (RJD) to be used in the modelling. The factorial design includes the RJD 25%, 30% and 35%. In addition, further specific calculations are made in Chapter 5 with other RJD values.

4.5.5. Friction

Between the limited studies that exist about the friction between the concrete slab and the base layer (Stott, 1961; Wesevich et al, 1987; Wimsatt et al, 1987; AASHTO, 1993; Rozycki & Rasmussen, 1998), there are different definitions of friction and not all of them include the asphalt concrete base. However, the research of Zhang & Li (2001), considered for the comparison with the Houben assumption of friction, yields similar frictional restraint characteristics for concrete on asphalt concrete and granular bases (Zhang & Li, 2001). Accordingly, in the factorial design the sections over granular base and asphalt concrete are considered with the same friction value of 1.0 according to the value adopted by Houben (2010a). This friction value is also in the order of magnitude applied in other investigations regarding to JPCPs (Darter & Barenberg, 1977; AASHTO, 1993; Smith et al, 1990; Lee & Stoffels, 2003; Ruiz et al, 2006; Beom & Lee, 2007). In order to take into account possible variations of the friction value, a sensitivity analysis is performed in Chapter 5.

4.5.6. Concrete grade

The research approach of the present thesis includes the changes that the concrete experiences since early-age rather than fixed mean values. In particular, the concrete changes are modelled by the development in time of the modulus of elasticity, the tensile strength of the concrete, the shrinkage and the thermal deformation. The development in time of these properties depends on the concrete grade (C28/35 and C35/45) as they have been described in Chapter 3.

4.6. CONFIGURATION OF THE FACTORIAL MATRIX AND SIMULATIONS

The Table 4.6.1 presents the factorial matrix with the combinations of independent variables. The 3 simulations per cell correspond to the 3 values of the slab lengths of the evaluation condition.

Table 4.6.1. Factorial matrix with number of simulations per cell.

Saw-cutting (RJD %)	Friction	Concrete grade	Season and time of construction	
			Summer 4 pm	Winter 10 am
25	1.0	C28/35	3	3
		C35/45	3	3
30	1.0	C28/35	3	3
		C35/45	3	3
35	1.0	C28/35	3	3
		C35/45	3	3

The simulations are made with the Relaxation Factor (RF) proposed by Pradena & Houben (2015b) for JPCPs (Eq. 4.6.1). This proposed RF has been the base for the RF applied by Xuan (2012), Mbaraga (2015) and Wu (2015) in their investigations. Xuan (2012) analysed the cracking process of cement treated mix granulates with recycled concrete and masonry for use in pavement bases. And Mbaraga (2015) and Wu (2015) studied the cracking process of cement stabilized bases with an additive. More details about the RF are given in Chapter 5 including comparisons of the modelling results with the behaviour of real-world JPCPs to adjust the RF.

$$R = 0.8265 * e^{-8*10^{-5}*t} \tag{4.6.1}$$

In addition, the simulations of AvCW1st for short slabs are made considering a reduction of 50% of the crack width regarding to the one of a traditional JPCP with double slab length. This reduction is based on the work developed by AASHTO (1993), NCHRP (2003) and Pradena & Houben (2016b). More details are given in Chapter 5 where a field corroboration of the AvCW1st in test sections of traditional and short slab JPCPs is included. Indeed, the present research is focussed on the comparison between traditional and short slabs JPCPs when the effects of the early-age concrete behaviour are incorporated in the integral design of JPCPs. Actually, the configuration of the simulations and the results presented in this chapter have been defined from that perspective.

The cracking process of JPCPs is a complex phenomenon with numerous variables and outputs involved. Houben (2008, 2010b) presents detailed calculations, results and analyses of the cracking process of traditional JPCPs. In the present chapter the simulations and the results presented are focussed on the dependent variables previously established (AvCW1st and UnCrJ) to fulfil the particular purposes of the present investigation.

4.7. RESULTS OF THE SIMULATIONS

4.7.1. Results of the AvCW1st after 1 year of the JPCPs construction

The first dependent variable of the factorial design, and the priority in the present research, is the AvCW1st. The Table 4.6.2 presents the resulting AvCW1st (mm) 1 year after the pavement construction for the case of slab length L = 5 [m] of traditional JPCPs (i.e. short slabs of length 0.5L = 2.5 m). The AvCW1st of short slabs are presented in red colour adjacent to the ones of traditional JPCPs. The resulting trends presented in the Table 4.6.2 are similar for L = 4 [m] and L = 4.5 [m] (which are included in detail in Appendix B).

Table 4.6.2. AvCW1st (mm) for traditional JPCPs (L = 5 m) and short slabs JPCPs (0.5L = 2.5 m).

Saw-cutting (RJD %)	Friction	Concrete grade	Season and time of construction	
			Summer 4 pm	Winter 10 am
25	1.0	C28/35	2.6/1.3	1.5/0.8
		C35/45	3.7/1.9	1.9/1.0
30	1.0	C28/35	2.2/1.1	1.5/0.8
		C35/45	3.4/1.7	1.9/1.0
35	1.0	C28/35	2.0/1.0	1.5/0.8
		C35/45	2.2/1.1	1.7/0.9

4.7.1.1. Evaluation condition

As mentioned before, the objective criterion of the evaluation condition is the comparison of the AvCW1st between traditional and short slabs JPCPs. The 50% reduction of the AvCW1st in short slabs JPCPs can produce a radical difference in the provision of Load Transfer Efficiency (LTE) by aggregate interlock, i.e. without the necessity to include dowel bars. For instance, according to the experimental validation of the nonlinear aggregate interlock model of the finite element program EverFE, the crack width must be ≤ 1.1 mm for LTE ≥ 70% considered adequate for JPCPs (Davids & Mahonney, 1999). Considering this, in all the cases presented in Table 4.6.2, the traditional JPCPs are not able to provide adequate LTE by aggregate interlock. On the contrary, the short slabs can provide this LTE by aggregate interlock in most cases; in particular in all cases of winter construction (Table 4.6.2).

4.7.1.2. Effects of the independent variables on the AvCW1st

The influence of the independent variables on the AvCW1st is as follows:

- **Season and time of construction**: Clearly bigger AvCW1st are produced when the JPCP is constructed in summer due to after construction concrete shrinkage goes together with temperature decrease. Coincidentally summer is a typical season of construction of JPCPs, as the ones presented in Chapter 5 used for calibrating the model.
- **Saw-cutting (RJD):** In general is valid that the deeper the saw-cut is (i.e. bigger RJD), the smaller the AvCW1st because when the RJD is bigger the cracking process develops faster. Hence, the saw-cutting method applied can influence the AvCW1st and then the provision of adequate LTE by aggregate interlock.
- **Concrete grade:** The higher the concrete grade is, the bigger the AvCW1st. In effect, the higher shrinkage dominates over the higher tensile strength.

4.7.2. Results of the UnCrJ

The second dependent variable of the factorial design is the UnCrJ. The Table 4.6.3 presents the resulting UnCrJ (%) for the case of slab length L = 5 m of traditional JPCPs (i.e. short slabs of length 0.5L = 2.5 m). The UnCrJ of short slabs are presented in red colour adjacent to the ones of traditional JPCPs. The resulting trends are similar for L = 4 m and L = 4.5 m which are presented in Appendix B.

As short slabs JPCPs have more joints than the traditional JPCPs, the presence of UnCrJ is higher in short slabs as it is presented in Table 4.6.3.

The influence of the independent variables on the UnCrJ is as follows:

- **Season and time of construction**: Clearly the higher percentage of UnCrJ is produced when the JPCP is constructed in winter. The high presence of UnCrJ can be explained by the increasing temperature in the period shortly after construction and the RF applied (Houben, 2010b). The presence of UnCrJ must be avoided because they change the design hypothesis of JPCPs.

- **Saw-cutting (RJD):** In general is valid that the deeper the saw-cut is (i.e. bigger RJD), the lower the percentage of UnCrJ. Indeed, bigger RJD means weaker cross sections of the JPCP at the locations of the joints. Hence, the saw-cutting method applied can influence the presence UnCrJ (as it is presented in Chapter 5).

- **Concrete grade:** In general is valid that the higher the concrete grade is, the higher percentage of UnCrJ. In effect, the higher shrinkage dominates over the higher tensile strength.

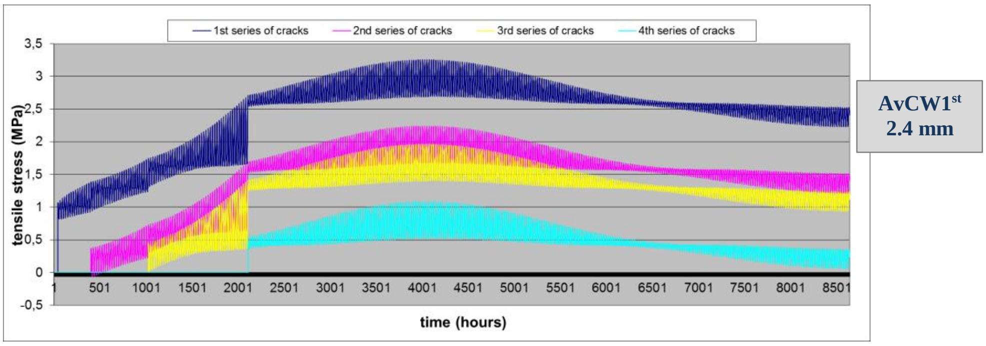
Table 4.6.3. UnCrJ (%) for traditional JPCPs (L= 5 m) and short slabs JPCPs (0.5L= 2.5 m).

Saw-cutting (RJD %)	Friction	Concrete grade	Season and time of construction	
			Summer 4 pm	Winter 10 am
25	**1.0**	**C28/35**	0/48	83/91
		C35/45	0/48	85/92
30	**1.0**	**C28/35**	0/0	73/86
		C35/45	0/48	85/92
35	**1.0**	**C28/35**	0/0	73/86
		C35/45	0/0	83/91

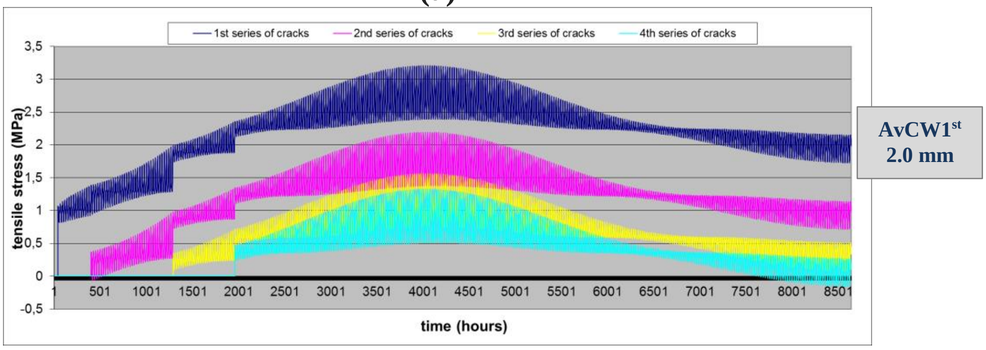
4.7.3. Necessity of a calibration process

As presented in sections 4.7.1 and 4.7.2 the model is able to predict important trends in traditional and short slabs JPCPs. However, a calibration of the model is required. For instance, the Table 4.6.2 presents some high values of AvCW1st (> 3.0 mm) that requires verification in the field. These high values are related with the relaxation function applied.

The Fig. 4.7.1 presents the trends of the crack width development until 1 year after construction of the JPCP for different slab lengths. In the present section a typical example is presented, but the behaviour is similar for the other cases of the factorial design included in Appendix D. In particular the Fig. 4.7.1 shows the case of JPCPs constructed in summer (4 pm), RJD 35%, concrete grade C28/35 and slab lengths 4 [m] and 5 [m]. Considering that in both cases of Fig. 4.7.1 there is no presence of UnCrJ, wider cracks for longer slabs (or at least similar crack widths) are expected. However, the Fig. 4.7.1 shows a smaller AvCW1st after 1 year for the longer slabs. This is the result of the relaxation function applied. Hence, although the model is able to predict important trends in traditional and short slabs JPCPs, a calibration process focussed on the RF is required. In effect, as presented in Chapter 3, the case of the RF is special due to the lack of experimental data and the significant differences produced when relaxation functions found in the literature are used to model the cracking process of JPCPs.



(a)



(b)

Fig. 4.7.1. Crack width development for JPCPs built in summer (4 pm), RJD 35%, concrete grade C28/35, slab lengths 4 [m] (a) and 5 [m] (b)

Another important behaviour observed in the Fig. 4.7.1 is the very similar initial development of the crack width of the 1st series of cracks despite the differences in slab length. Indeed, the crack widths are very similar until 1000 hours after the construction of the JPCPs (i.e. the first 40 days); the differences are produced later in time.

Again, the trends presented in this section are valid for all the cases of the factorial design which are presented in detail in Appendix B.

4.8. CONCLUSIONS

In the present chapter a factorial design has been established with the definition of the associated variables, the categories of the independent variables, their values and the number of simulations of the general scenarios of interest for the determination of the dependent variables AvCW1st and UnCrJ. Further specific scenarios of interest are included in the analyses of Chapter 5.

An evaluation condition associated to the factorial design has been established as well. Considering the importance of the AvCW1st for the link between the early-age behaviour and the in-service performance of non-dowelled JPCPs, the objective criterion used in the evaluation condition is the comparison of the AvCW1st between traditional and short slabs JPCPs. The modelling results presented in this chapter show that the reduction of AvCW1st in short slab JPCPs produces a radical difference in the provision of LTE by aggregate interlock. Indeed, while the traditional JPCPs cannot provide this LTE, the short slabs are able to provide it in most of the cases presented.

Considering the mentioned importance of the AvCW1st for the present research, the construction of JPCPs in summer requires further analyses (Chapter 5) because in this season of construction the most unfavourable condition for the AvCW1st, and the provision of adequate LTE by aggregate interlock is produced. Furthermore, summer is a typical season of construction of JPCPs.

As short slabs JPCPs have more joints than traditional JPCPs, the presence of UnCrJ is higher in short slabs. The presence of UnCrJ jeopardizes the benefits of short slabs because their postulated benefits depend on the fact that the slabs are effectively shorter. And when the joints remain uncracked, the effective slab length is not short anymore.

The saw-cutting method applied (RJD) has an influence on the value of the AvCW1st and on the presence of UnCrJ. Considering this, the RJD is one of the most influential factors possible to control in order to obtain the desired results of AvCW1st and UnCrJ.

Although the model is able to predict important trends in traditional and short slabs JPCPs, an adjustment of the model to the real-world behaviour of JPCPs is required. This calibration process is presented in Chapter 5.

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5. MODEL CALIBRATION AND POST-CALIBRATION ANALYSES

5.1. INTRODUCTION

Client satisfaction is a goal toward which any service or product provider, including pavement engineers, should strive (Haas & Hudson, 1996). In quality terms this is basically the application of the principle of the costumer focus of the standard ISO 9000 (ISO, 2005). Another principle of this standard is the process approach, which includes in its definition ‘the identification and interactions of these processes’ to produce the desired outcome (ISO, 2008). In the Fig. 5.1.1 the specific objectives of this thesis are presented as processes. In effect, the first specific aim can be seen as the process of modelling the concrete behaviour since early-age in JPCPs. Similarly, the third and fourth objectives can be seen as the structural and functional processes of the in-service performance of the JPCP. In this way, the interaction of the processes is represented by the second specific objective “Determine relevant results for the link between the early-age behaviour and the in-service performance of JPCPs”.

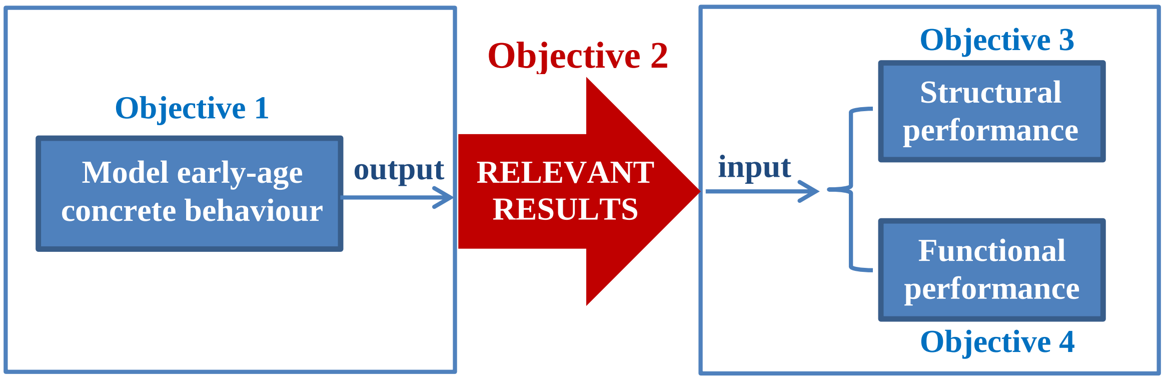

Fig. 5.1.1. Objectives of the research as a system and interaction of processes

Client satisfaction is the client's perception of the degree to which their requirements have been fulfilled (ISO 9000, 2005). In the classification of these requirements made by Haas & Hudson (1996) the ride quality was classified as highly important for the largest groups of pavement’s clients, i.e. the users and the owners (or agencies acting in their behalf). In addition, the structural adequacy was classified as high priority for the owners or agencies. Then considering the pavements client’s requirements, the relevant results of the early-age behaviour of JPCPs will be the ones that have influence on the functional and structural in-service performance of the JPCPs. As mentioned in previous chapters, these relevant results are the Average Crack Width of the 1st series of cracks (AvCW1st) and the presence of Uncracked Joints (UnCrJ). However, in the present investigation, priority is given to the AvCW1st because it is the direct cause of the provision of LTE by aggregate interlock of non-dowelled JPCPs (as short slabs) and, as it was presented in Chapter 4, the AvCW1st is associated to the evaluation condition which is actually the comparison of the AvCW1st between traditional and short slabs JPCPs (in order to compare the LTE by aggregate interlock provided by these two alternatives).

The Fig. 5.1.2 presents these relevant results of the early-age behaviour of JPCPs that have an influence on the functional and structural in-service performance of the JPCPs. In the Fig. 5.1.2 the priority that the AvCW1st has in the present investigation is highlighted.

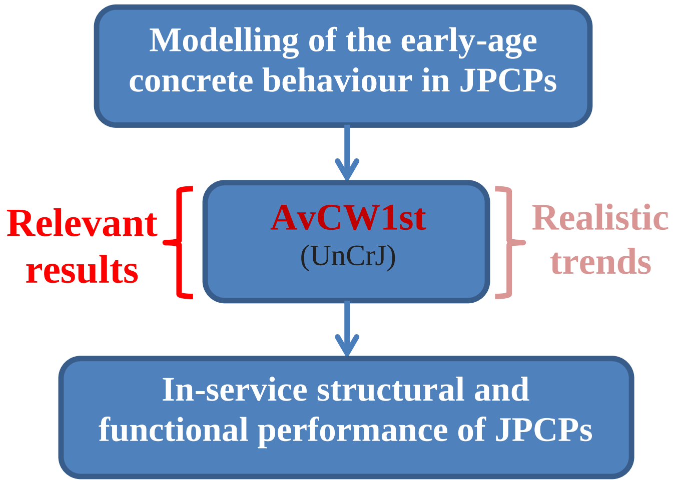

Fig. 5.1.2. Relevant results for the link between the early-age behaviour and the in-service performance of JPCPs

The modelling of the cracking process in JPCPs is a complex subject with numerous variables involved. Even the detailed Mechanistic-Empirical Pavement Design Guide (MEPDG) uses a simplified formula for the estimation of the crack width under the joints (NCHRP, 2003). As mentioned in the previous chapters, in this thesis a system approach is applied to predict the cracking process of JPCPs, where the results are not only time-dependent but also space-dependent (see chapters 2 and 3). Still, simplifications have been made to find practical and useful trends of AvCW1st with the following objectives:

- Make the link between the early-age behaviour and the in-service performance of JPCPs (Fig. 5.1.2)
- Compare the LTE by aggregate interlock provided by non-dowelled traditional and short slabs JPCPs.

In order to obtain realistic trends from the modelling (Fig. 5.1.2), a calibration procedure is required. This procedure must consider that a “good” model depends on what it is used for and how accurate the prediction or calculation should be (Erkens, 2002). In addition, as mentioned during the course of this thesis, the present investigation searches for pavement clients’ satisfaction. Accordingly, practical and useful methods for pavement clients are needed. Because of that, in the present thesis a practical procedure for the calibration of the model of the early-age concrete behaviour in JPCPs is adopted. With such a calibration procedure, the model can be adjusted to the particular characteristics of JPCPs of specific regions and/or JPCPs applications (urban, interurban, airports, etc.) in a practical way. This can be very useful for pavement clients as public agencies, for instance to define the relation between the LTE by aggregate interlock with the direct cause of it, which is the crack width (at joint) resulting from the early-age concrete behaviour.

5.2. CALIBRATION PROCEDURE

5.2.1. Terminology

The terms ‘calibration’ and ‘validation’ are used somewhat interchangeable. Sometimes the two terms are used to refer to different phases of the process of making sure that the model represents real world conditions. Rahka et al (1996) established an ascending order of real world/experimental data required for the verification, validation and calibration phases. In this way, model verification is the process of determining that a model implementation accurately represents the developer's conceptual description of the model and the solution to the model (AIAA, 1998). Thus this part of the process is focussed on the mathematical and logical procedure and does not require a comparison with data of the real world conditions. Instead, model validation requires a comparison of the model results with real world or experimental conditions. In effect, model validation is the process of confirming that the predictions of a model represent measured physical phenomena (Trucano et al, 2006). The main idea behind validation is to know whether a model can be used for prediction purposes. Validation compares simulated system output with real system observations using data that were not used in the model development (Rykiel, 1996). Calibration instead, is the improvement of the agreement of a model calculation with respect to a chosen benchmark through the adjustment of parameters implemented in the model (Trucano et al, 2006). Hence, the requirements for real-world/experimental data are ascending between the verification, validation and calibration phases[2]. It is important to note from the definition of calibration that the model calculation needs to be compared with a chosen benchmark. In the present investigation the main benchmark is the AvCW1st for the reasons expressed in the previous section and chapters of the thesis.

In pavement engineering, this progressive process with increasing requirements for real-world/experimental data is used by the calibration process of the Highway Development and Management System (HDM-4). Indeed, the Fig. 5.2.1 presents the 3 calibration levels of HDM-4 with increasing requirements of field data, time and resources (Bennett & Patterson, 2000).

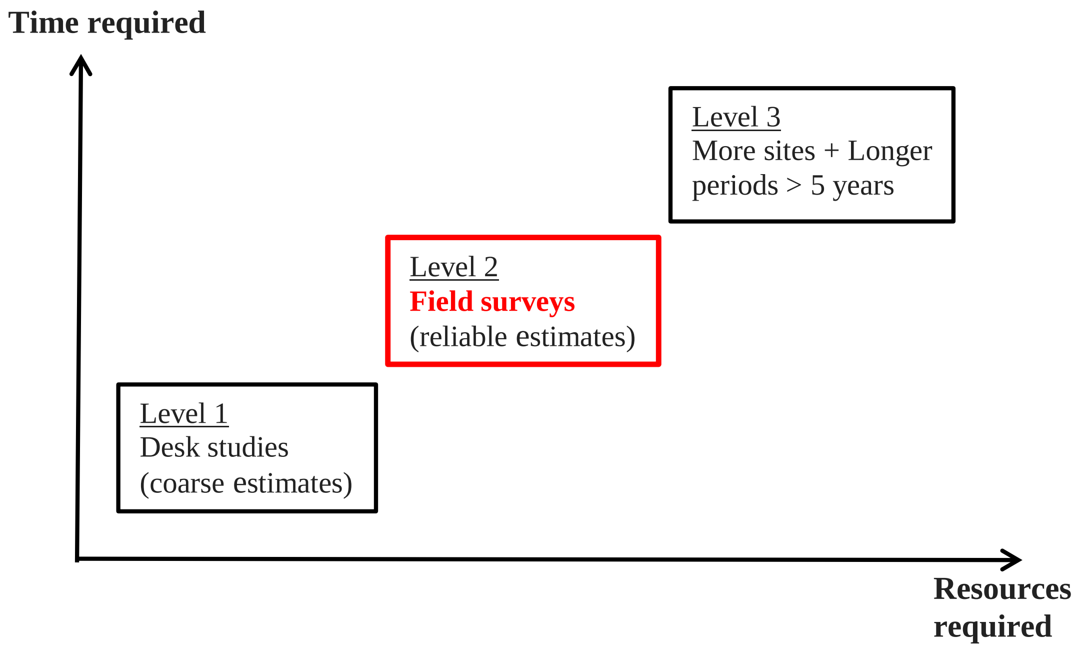

Fig 5.2.1. Calibration levels and their requirements
(Adapted from Bennett & Patterson, 2000)

[2] In order to avoid confusion between the terms 'calibration procedure' and 'calibration phase', from now on the term 'calibration phase' is replaced for 'adaptation phase'.

The Level 1 is focussed on the most sensitive parameters comparing the modelling results with desk studies or minimal field surveys. The Level 2 requires measurement of additional input parameters and field surveys to calibrate key predictive relationships. The Level 3 undertakes major field surveys and controlled experiments to enhance the existing predictive relationship or to develop a new and locally specific relationship for substitution in the source code of the model. In the terms presented in Fig 5.2.1 the work of Houben (2008b, 2008c, 2010b), which is the base of the modelling of the present investigation, was concentrated in the level 1. In the present thesis a review of the level 1 is made but the work is centred in level 2 (highlighted in Figure 5.2.1), i.e. measurement of input parameters and field surveys to calibrate key predictive relationships.

5.2.2. Progressive phases of the calibration procedure

The progressive calibration procedure of the present research starts with the verification phase, next the validation phase and finally the adaptation phase. The Fig. 5.2.2 presents a scheme of these phases with a brief explanation of them. Further explanations are given in sub-sections 5.2.2.1 to 5.2.2.3.

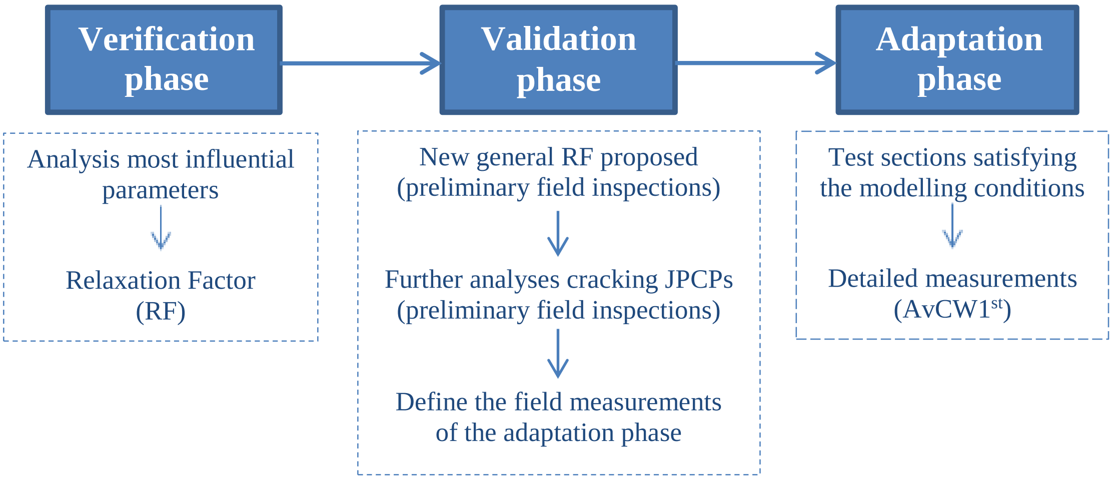

Fig. 5.2.2. Progressive calibration procedure

5.2.2.1. Verification phase

In the guide to calibration and adaptation of HDM-4 the recommendation is, before to start the calibration process itself, to make an analysis of the most influential parameters and then concentrate the next phases of the calibration procedure on the selected parameters or key predictive relationships (Bennett & Patterson, 2000). The analysis of Chapter 3 showed the case of the Relaxation Factor (RF), where there is a lack of experimental data and models based effectively on the relaxation instead of creep (Atrushi, 2003). In addition, between the few models available (based on the relaxation itself) significant differences were detected when they were compared in a period of time useful for the objectives of the present research. Accordingly, the validation and adaptation phases are concentrated on the RF which is also in agreement with Rykiel (1996) who states 'calibration procedures can be used to estimate parameter values that are otherwise unknown' or the definition of the NASA (2016) that states 'it adjusts unmeasured or poorly characterized experimental parameters'.

In the present chapter the RFs of Morimoto & Koyanagi (1995), Lokhorst (2001) and Houben (2008a, 2008b) (presented in Chapter 3) are applied to the modelling of the cracking process of JPCPs.

5.2.2.2. Validation phase

The validation phase starts when the results of the modelling with the different RFs are compared with the general behaviour of real-world JPCPs. This comparison is made according to experience but also considering the order of magnitude of the AvCW1st observed in preliminary field inspections of JPCPs in Chile and Belgium. The output of such analysis is the proposal of a general equation of RF. At this level, only the general shape of the equation is proposed, including calibration constants with default values 1 for adjustments in the adaptation phase.

The proposed general equation of relaxation is proposed based only on the transversal cracking at joints of JPCPs because the crack width under those joints is directly related with the LTE of the predominant traffic direction. In addition, in the field those crack widths are more easy to inspect than the ones under longitudinal joints. This is not only due to the obvious less quantity of longitudinal contraction joints but also because these joints do not appear at a pavement edge. Furthermore, many JPCPs do not even have these joints. Indeed, depending on the system of construction, the JPCPs can have longitudinal construction joints instead of contraction ones.

However, with the proposed general equation of RF further analyses of cracking in concrete pavements are performed in order to identify the behaviour of the longitudinal cracking in JPCPs, the available time to saw-cut the joints, or the width possible to build in one gang without risk of longitudinal cracks. The results obtained are compared with the behaviour observed in the general practice of JPCP construction, the previous work of Houben (2008a, 2008b, 2010a, 2011, 2014) and preliminary field inspections of JPCPs in Chile and Belgium. One of the objectives of these analyses is to determine the priorities (and practical possibilities) to perform the field measurements of the adaptation phase.

5.2.2.3. Adaptation phase

According to the progressive calibration procedure, the focus of the field measurements of the adaptation phase is defined in the previous phase of validation. These field measurements of the calibration phase must be performed in test sections that satisfy the modelling conditions. For instance, test sections with enough length for the cracking process to develop and with continuity of the pavement structure at the beginning and the end of the test section. In effect, the model to predict the cracking process at joints in JPCPs was developed originally for pavements with these conditions. In addition, the requirements of the real-world data are higher for the adaptation phase. In particular, in the test sections used to adapt the model, a Linear Variable Differential Transformer, LVDT with accuracy 0.001 mm is used to determine the AvCW1st (section 5.4.2).

In the adaptation phase the calibration constants (default value = 1) need to be adjusted to improve the agreement between the model predictions and the real-world behaviour of JPCPs. The objective indicator of this agreement (AGREE) is given by the ratio between the mean real-world value and the mean modelled value (Bennett & Patterson, 2000) as it is shown in Eq. 5.2.1.

$$AGREE = \frac{Mean\ real-world\ AvCW1^{st}}{Mean\ \text{modelled}\ AvCW1^{st}} \tag{5.2.1}$$

As mentioned during the course of this thesis, the present investigation searches for pavement clients' satisfaction. Accordingly, a calibration procedure practical and useful for pavement clients (as transportation agencies) is required. In agreement with this, the adjustment of the calibration constants associated to the RF curve is a practical and useful calibration procedure that can certainly be implemented for public agencies related with the different JPCP applications (urban, interurban, airports, etc.). Such procedure can be applied, for instance, to define the relation AvCW^{1st} – LTE for pavement design manuals of geographical regions and/or certain JPCP applications (urban, interurban, airports, etc.).

5.3. REAL-WORLD DATA

5.3.1. Necessity of obtaining real-world data directly

As JPCP with short slabs is a recent development in which the relevant results of the early-age concrete behaviour have not been properly investigated, it is necessary to obtain real-world data directly. As the development of short slabs JPCPs has been concentrated in Chile, most of the real-world data are obtained in Chile. Additionally, a test section of a traditional JPCP is located in Belgium.

5.3.2. Main JPCP applications: interurban, urban, industrial, airports

The scope of the real-world data includes the 4 main areas of application of JPCPs (urban roadways, industrial areas, rural roads and airport aprons) to analyse if the potential benefits of short slabs can be valid in different JPCP applications with their particular characteristics (see chapters 6 and 7). In any case, to make the link between the early-age behaviour and the in-service performance of JPCPs it is necessary first to know the relevant results from the early-age concrete behaviour in these 4 main JPCPs applications. Theoretically the modelling of the early-age concrete behaviour to determine these relevant results should be basically the same for the 4 main JPCP applications. However, the model to predict the cracking process at joints in JPCPs was developed originally for pavements that have available the necessary length, with continuity of the pavement structure at the beginning and the end of the JPCP, to the development of the cracking pattern. This is a typical case in interurban JPCPs, but not necessarily in some short urban streets, industrial floors and yards with irregular configurations, discontinuities as offices or warehouses dimensions; and airport aprons where the construction length can be restricted by the necessity to continue the airport operations (as is the case presented in this thesis). Hence, as part of the post-calibration analyses, the AvCW1st predicted by the calibrated model is compared with real-world data of test sections of the 4 main JPCP applications, particularly the ones with restricted length.

The Table 5.3.1 presents the test sections of JPCPs included in the analyses of the present chapter.

Table 5.3.1. Test sections of JPCPs used in the present study

ID test section	Country	Province	Application	Year of constr.	Length section (m)	Slab thickness (mm)	Saw-cut[1] depth (mm)	Slab length (m)	Year of investigation	Base	Concrete grade
I1	Belgium	Limburg	Interurban	2006	100	220	65	5.50	2011-2012	Granular[2]	C35/45
U1	Chile	Concepcion	Urban	2012	100	260	80	4.50	2012	Granular	C35/45
U2a	Chile	Bio-Bio	Urban	2012	70	120	40	3.50	2012	Granular	C28/35
U2b	Chile	Bio-Bio	Urban	2012	70	120	40	1.75	2012	Granular	C28/35
U3	Chile	Concepcion	Urban	2012	70	150	50	3.50	2012	Granular	C28/35
I2a	Chile	Tierra del Fuego	Interurban	2012	100	140	50	2.00	2012	Granular	C35/45[3]
I2b	Chile	Tierra del Fuego	Interurban	2012	100	140	50	2.00	2012	Granular	C35/45[3]
Ind-a	Chile	Concepcion	Ind. Floor	2012	24	120	30	2.00	2012	Granular	C28/35
Ind-b	Chile	Concepcion	Ind. Floor	2012	24	120	30	2.00	2012	Granular	C28/35
A1	Chile	Santiago	Airport Apron	2012	30	460	115	4.00	2012	Granular	C28/35

(1) Average saw-cut depth measured in the field
(2) Assumed (a sensitivity analysis of the friction value is included in section 5.4.3.5)
(3) Includes synthetic fibres

All the test sections presented in Table 5.3.1 were built in summer, which is a typical season of construction of JPCPs. And, as mentioned in Chapter 4, in this season of construction the most unfavourable condition for the AvCW1st (and provision of LTE by aggregate interlock) is produced. In addition, one of the main reasons why the majority of the test sections of the Table 5.3.1 are located in Chile is due to the concentration of the innovative short slabs JPCPs in Chile.

5.3.3. Obtaining the real-world data

Related to the measurements of crack width there are several studies performed on JPCPs and continuously reinforced concrete pavements. In particular, Poblete et al (1988), NHL (1989), Kohler & Roesler (2005, 2006) have presented detailed crack width measurements and valuable information as the rotation of the crack faces during the day or crack width as a function of depth. Chou et al (2004) performed measurements of the crack width at the middle of the JPCP thickness (at the edge of the pavement) in airport JPCPs, even though these JPCPs are thicker than those in other applications. Chou et al (2004) based this decision on the work of Pitman (1996) who concluded, after a statistical analysis of pavement cores, that the crack widths of the upper half are statistically equal to the ones of the lower half of the cores. On the other hand, Lee & Stoffels (2001) performed measurements on the surface of the joints considering them representative for the variation of the crack width along the joint depth. Their decision to perform the measurements on the surface was based on the maximum difference of 0.15 mm reported by Poblete et al (1988) in Chilean undowelled JPCPs, which represents less than 5% of the range of crack width in most cases. Lee & Stoffels (2001) considered this difference insignificant for the purposes of their research (analysis of the performance of joint seals).

The previous examples show that it is necessary to take into account the purposes of the particular study in order to define an optimal method to measure the crack width at joints of JPCPs. Accordingly, for the objectives of the present investigation ($AvCW1^{st}$ for the link with the in-service JPCP performance) it is considered appropriate to perform the measurements of the crack width at the middle of the JPCP thickness (at the edge of the pavement) using a fissuremeter (Fig. 5.3.1 left).

Fig 5.3.1. Measurement of the crack width with fissurometer (left) and pavement temperature measured with infrared thermometer (right).

The average pavement temperature is estimated from the information of the meteorological station closest to the JPCP, supported by discrete measurements of the pavement temperature made with infrared thermometers (Fig. 5.3.1 right). With this information, the variables $T_{ampyear}$ and T_{ampday} used in the modelling (chapters 3 and 4) are determined with the objective to compare the modelling results with the real-world JPCP behaviour.

As mentioned previously, the requirements of real-world data are higher for the adaptation phase. In this phase in particular, the crack width measurements are performed with fissurometer, Linear Variable Differential Transformer (LVDT) and an invar bar (due to its low coefficient of thermal expansion, $1.2*10^{-6}$/°C). The measurements with the fissuremeter yield the absolute value of the crack width, and the LVDT (Fig. 5.3.2) yields the variation of the crack width at different temperatures. These values are compared to the calculation of the AvCW1st. For every measurement of the variation of the crack width, the LVDT is set up with the bar (Fig. 5.3.2 left), and then the LVDT is positioned on two studs previously fixed on the edge of the pavement (Fig. 5.3.2 right).

Fig. 5.3.2. Measurements with LVDT and invar bar.

The LVDT-invar bar system has different practical advantages because the measurements do not require embedded sensors placed previously in the pavement during the construction process. Even more, it is not necessary to disrupt the pavement surface and the method is cost-effective because of its accuracy (0.001 mm) and the necessity of just 1 LVDT to perform the field measurements. The particular LVDT used in the present research is the model Sylvac 305-1301 (Fig. 5.3.2).

Finally, it is possible to identify the presence of UnCrJ in the field performing a visual inspection of the edge of the pavement.

5.4. VERIFICATION, VALIDATION AND ADAPTATION PHASES

5.4.1. Verification phase

As presented in Chapter 3, the case of the Relaxation Factor (RF) is special due to the lack of experimental data and the fact that, between the few models available (based on the relaxation itself), significant differences exist when they are compared over a period of time useful for the objectives of the present research. In the verification phase the RFs of Morimoto & Koyanagi (1995), Lokhorst (2001) and Houben (2008a, 2008b) (presented in Chapter 3) are applied to the modelling of the cracking process in JPCPs. The Table 5.4.1 shows the modelling results after 1 year of the transversal cracking of JPCPs built in summer[3] (4 pm) with concrete grade C28/35, friction 1, RJD 30% and slab length 4.5 m.

According to experience, the expression proposed by Houben (2008a, 2008b) is the only one that provides realistic results. In fact, the results obtained applying the expression of Lokhorst (2001) are highly unreal for JPCPs, both with respect to the percentage of cracked joints and the width of the few cracked joints. The expression of Morimoto &

[3] This is the same scenario indicated in Chapter 4, i.e. at the beginning of August (referred to the northern hemisphere).

Koyanagi (1995) not only yields 100% of the joints cracked but also the slabs are cracked, i.e. once all joints have cracked new cracks are produced in the middle of the slabs, which is not observed in the field.

Table 5.4.1. Modelling results (transversal cracking) with different relaxation factors.

Dependent variables	Houben	Morimoto & Koyanagi	Lokhorst
Joints cracked (%)	50	100	6
Maximum crack width (mm)	1.5	4.3	20.4
AvCW1st (mm)	0.8	3.2	19.0

5.4.2. Validation phase

5.4.2.1. Proposal of a general equation of the RF

Although the RF proposed by Houben (2008a, 2008b) is the only one of the Table 5.4.1 that provides realistic results, the AvCW1st is smaller than the ones observed in preliminary field inspections of JPCPs in Chile and Belgium. Considering these results, the fact that there is a lack of experimental data on stress relaxation at early-ages (Atrushi, 2003), and the importance of stress relaxation in the reduction of self-induced stresses in hardening concrete, a first general proposal of a RF as a function of time is made (Eq. 5.4.1), based on a theoretical and practical analysis of the transversal cracking in JPCPs (Pradena & Houben, 2012a, 2012b).

$$R = a * 0.8265 * e^{-b*8*10^{-5}*t} \tag{5.4.1}$$

Where:

t = time (hours after construction)

a,b = calibration constants (default value 1)

The Fig. 5.4.1 shows different expressions of the RF including the proposed Eq. 5.4.1 of Pradena & Houben (2012a, 2012b).

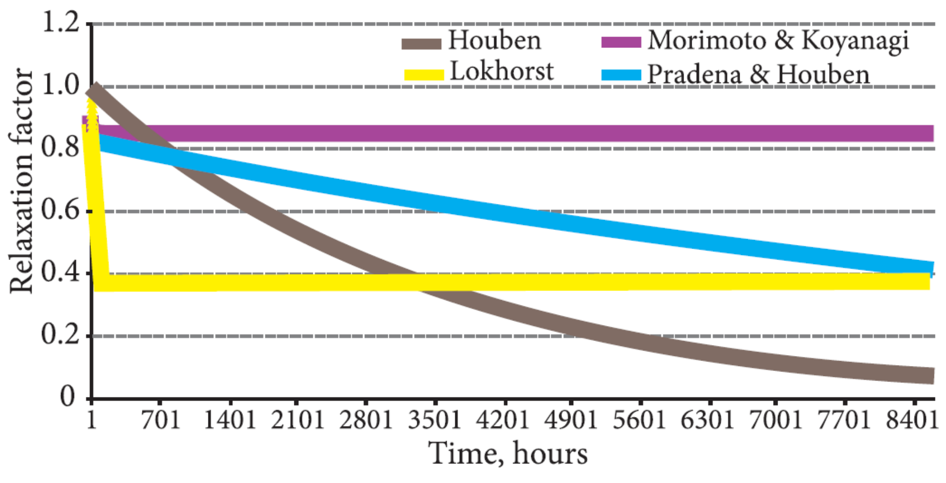

Fig 5.4.1. Different curves of relaxation factors.

The Table 5.4.2 shows the modelling results with the proposed new relaxation equation for the same simulation conditions utilized for the results of the Table 5.4.1. The results of the Table 5.4.2, especially the AvCW1st, are in agreement with the order of magnitude of the AvCW1st observed in preliminary field inspections of JPCPs in Chile and Belgium.

Although the Eq. 5.4.1 is in a general state, i.e. with the constants 'a' and 'b' still not calibrated, it has been the base for the RF applied by Xuan (2012), Mbaraga (2015) and Wu (2015) in their investigations. Xuan (2012) analysed the cracking process of cement treated mix granulates with recycled concrete and masonry for use in pavement bases. And Mbaraga (2015) and Wu (2015) studied the cracking process of cement stabilized bases with an additive.

Table 5.4.2. Modelling results (transversal cracking) with the proposed RF.

Dependent variables	Proposal
Joints cracked (%)	100
Maximum crack width (mm)	3.3
AvCW1st after 1 year (mm)	2.1

In the case of JPCPs, further analyses of the cracking process of concrete pavements can be made with the general RF proposed (Eq. 5.4.1) in order to evaluate where the field measurements for the adaptation phase must be focussed.

5.4.2.2. Concentration of the field work

The general equation of RF was proposed based only on the cracking at transversal joints of JPCPs because the crack widths under those joints are directly related with the LTE of the predominant traffic direction. In addition, as mentioned previously, in the field those crack widths are more easy to inspect than the ones under longitudinal joints.

Later, applying the proposed RF equation, Pradena & Houben (2015) made further analyses of cracking processes in JPCPs (transversal and longitudinal) and PCPs[4] (transversal and longitudinal). The results obtained by Pradena & Houben (2015) confirmed the general good behaviour of the proposed RF and the necessity to concentrate the calibration phase on the transverse cracking of JPCPs. For instance, the crack width under the longitudinal joint of JPCPs (if a crack is actually produced) never exceeds 0.3 mm, which is far below the limit of 1.1 mm for adequate LTE (Davids & Mahonney, 1999). This is in agreement with the results obtained by Houben (2011) for different conditions and with the magnitude observed in preliminary field inspections in a Chilean JPCP (Pradena & Houben, 2015). Hence, the further analyses of Pradena & Houben (2015) confirmed that the focus of the field measurements of the adaptation phase must be at the transverse joints of JPCPs.

Hence, the field measurements need to be focussed on the transversal cracking process of JPCPs, specifically on the AvCW1st. With this approach, it is possible to find particular calibration constants (associated to the RF curve) that adjust the model for the conditions

[4] PCPs: Plain Concrete Pavements, i.e. "non-jointed" or "non-weakened". This represent the case of JPCPs before saw-cutting the joints. The analysis of the cracking process in that state (PCPs) is useful to define the available time to saw-cut the joints or the width of the JPCP that can be built in one gang without risk of longitudinal cracks.

under study, for instance, characteristics of the JPCPs of a particular region. This is a practical calibration procedure taking into account the intended uses of the model (Fig. 5.4.2), the mentioned difficulties associated to model the complex cracking process of JPCPs, the general good correlation of the model with the real-world trends, and the fact that the present thesis searches for practical and useful methods for the pavement clients (as transportation agencies for example). Indeed, as mentioned during the course of this thesis, the present research searches for the pavement clients' satisfaction and this practical calibration approach can be very useful not only for the different public agencies related with different JPCP applications (urban, interurban, airports, etc.), but also for others organisations related with the structural and functional design of JPCPs (normative institutions, concrete associations, consultants, research groups, universities, etc.)

Fig. 5.4.2. Intended use of the model of the early-age concrete behavior in the present research.

5.4.3. Adaptation phase

5.4.3.1. The method of the windows

The determination of the AvCW1st and the presence of UnCrJ require access to the edge of the pavement, ideally at different times. In the practice of pavement construction this is not always possible because the edge is covered soon by the construction of the adjacent lane of the airport apron, a shoulder or a curb of the urban roadway, a perimeter fencing of industrial warehouses, etc. Hence, when it is necessary the method of the windows or the snapshots can be applied. This method consists of comparing sections of structures with similar characteristics but different ages. The method has been used in the calibration of pavement deterioration models (Videla et al, 1996; Videla et al, 1997; de Solminhiac et al, 2003a, 2003b; Thube, 2011) or the analysis of pavement friction in time (Echaveguren et al, 2009a, 2009b), between others applications. A window is a section that exhibits uniformity in terms of its most representative variables (according to the phenomenon under study). Following the logic that the performance models of these individual sections will be very similar, sections which exhibit similar characteristics can be assimilated for these purposes into a single structure (Videla et al, 1996; Videla et al, 1997). With this method, the measurements of deterioration can be done in a short period of time. Because of this and taking into account the intended uses of the model (Fig. 5.4.2), the mentioned difficulties to access the edge of the concrete slabs, and the fact that the present thesis searches for practical and useful methods for pavement clients, the method of the windows is applied in the adaptation phase.

In particular, in the curve of the crack width development, there are 2 fundamental windows to consider. The first one is immediately after the construction, when all the changes in the early-age concrete are developing, and then they determine the future concrete behaviour. Additionally, as explained previously, when the JPCP is still under construction the pavement edge is accessible to perform the field measurements. The second fundamental window is when the crack width behaviour is more stable (≥ 1 year) and it can be related with a representative LTE of the in-service pavement performance. The Fig. 5.4.3 shows these 2 fundamental windows in the crack width development.

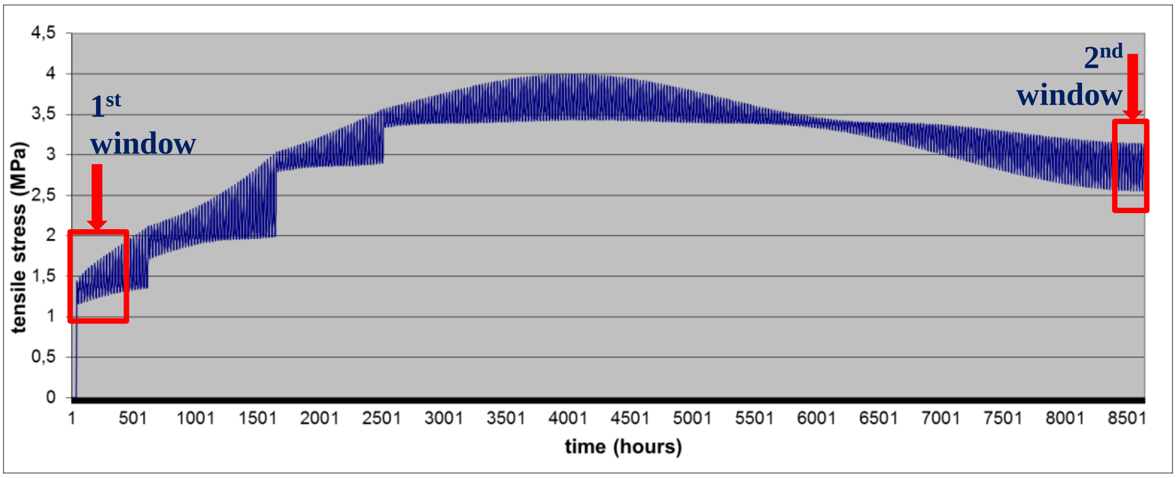

Fig. 5.4.3. Fundamental windows in the crack width development of the 1st series of cracks (construction at summer 4 pm, $T_{ampyear}$ = 10°C and T_{ampday} = 5°C, with the characteristics of section 'I1' of Table 5.3.1)

5.4.3.2. Test sections for the adaptation phase

The field measurements for the adaptation phase must be done in test sections that satisfy the conditions for which the model was originally developed, i.e. traditional JPCPs with enough length to develop the cracking pattern, with continuity of the pavement structure at the beginning and the end of the test section (Houben, 2008a). Considering the work of Houben (2008b, 2008c, 2010b) and Pradena & Houben (2012a, 2012b, 2015), 100 [m] of pavement is considered enough length to develop the cracking pattern of JPCPs.

Taking into account the two most fundamental windows in the curve of the crack width development, tests sections immediately after the JPCP construction and 1 year (or more) after the pavement construction are required for the adaptation phase. The test sections of the Table 5.3.1 that fulfil these requirements are the sections 'I1' and 'U1', i.e. the interurban JPCP oonN74 in the province of Limburg (Belgium) and the urban JPCP of the province of Concepcion (Chile). This last one, although it is an urban JPCP, is part of a bus corridor with continuity of the pavement structure at the beginning and the end of the test section (Fig. 5.4.4 right). The Table 5.4.3 presents the characteristics of these test sections.

Table 5.4.3. Characteristics of the test sections for the adaptation of the model.

ID test Section	Length section (m)	RJD (%)	Slab length (m)	Base	Concrete grade
I1	100	30	5.50	Granular	C35/45
U1	100	30	4.50	Granular	C35/45

In Table 5.4.3 the saw-cutting depth and the slab thickness have been replaced by the Relative Joint Depth (RJD) which is actually the ratio of them.

All the parameters of the Table 5.4.3 are the same for both sections, with the only exception of the slab length. According to Houben (2008b, 2010b) the slab length does not have a significant influence on the AvCW1st 1 year after the JPCP construction. Although the simulations presented in Chapter 4 (and Appendix B) show some differences, they only occur since 1000 hours after the construction of the JPCPs (i.e. after the first 40 days). In effect, the initial behaviour (< 1000 hrs) of the crack width of the 1st series of cracks is very similar despite the differences in slab length (additionally presented in Figures 5.4.5 and 5.4.6). As the crack width measurements in the Chilean test section are performed until 500 hours after the JPCP construction, the behaviour of the crack width of the 1st series of cracks can be considered equivalent to the one of the Belgian section.

As explained in Chapter 4, the friction value (between the concrete slab and the granular base) has been taken as 1.0 according to the value adopted by Houben (2008b, 2008c, 2010b). Additionally, a sensitivity analysis of the friction value is included in section 5.4.3.5.

The Figures 5.3.1 and 5.3.2 correspond to the Belgian test section. In the Fig. 5.4.4 the Chilean test section is presented, in particular the paving between the formwork (Fig. 5.4.4 left) and the saw-cutting of transverse joints (Fig. 5.4.4 right). In this last picture it is possible to observe the continuity of the pavement structure.

Fig. 5.4.4 Paving with formwork (left) and saw-cutting the transverse joints (right) of the Chilean JPCP.

5.4.3.3. Measurements for the adaptation phase

The crack width measurements are performed with fissurometer (Fig. 5.3.1 left) and LVDT (Fig. 5.3.2) as explained in section 5.3.3. In that section the procedure to obtain the average pavement temperature was also described. The closest meteorological stations to the JPCPs are the station of the city of ‘Maaseik’ for the case of the Belgian JPCP and the station ‘Carriel Sur’ in Talcahuano city for the case of the Chilean JPCP. The measurements in the Chilean JPCP were performed at the end of January 2012 (summer in the southern hemisphere) since it was possible to have access to the edge of the pavement (after the formwork removal). In the case of the in-service Belgian JPCP the measurements were performed at October 2011 and July 2012.

5.4.3.4. Results

Comparison of the real-world AvCW1st and the modelled AvCW1st

The modelling has been made with the characteristics of the test sections 'I1' and 'U1' presented in Table 5.4.3, construction in summer (4 pm), $T_{ampyear}$ = 10°C and T_{ampday} = 5°C (determined according to the procedure of section 5.4.3.3). The calculated development of the 1st series of cracks during the first year is shown in Fig. 5.4.5 for slab length 5.5 [m], i.e. the case of test section 'I1'. In the graph the AvCW1st at 60 [hrs], 500 [hrs] and after 1 [year] are indicated.

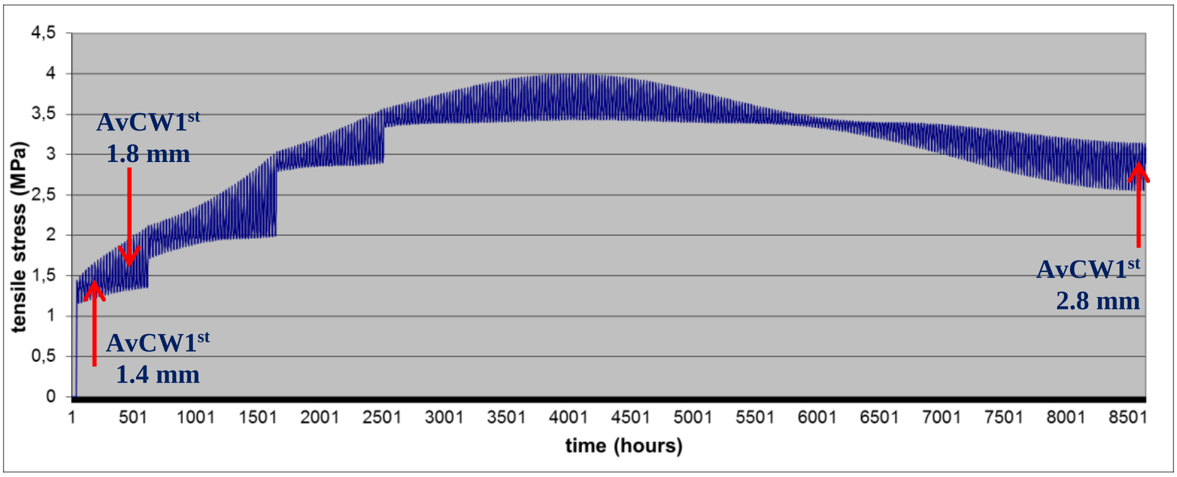

Fig. 5.4.5. Modelled crack width development (1st series of cracks) for slab length 5.5 [m] (the red arrows show where the AvCW1st is obtained)

The Table 5.4.4 presents the results of the AvCW1st from the field and the modelling of the test sections 'I1' and 'U1'. The Table 5.4.4 also presents the AGREE value, which is the objective indicator of the level of agreement between the modelled AvCW1st and real-world AvCW1st.

Table 5.4.4. Real-world AvCW1st, modelled AvCW1st (pre-calibration) and AGREE value

ID test section	Time	AvCW1st (mm)		AGREE
		Field	Model	
U1	60 hrs	1.1	1.4	0.8
U1	500 hrs	1.2	1.8	0.7
I1	> 1 year	1.6	2.8	0.6

The differences between the real-world AvCW1st and the modelled AvCW1st observed in Table 5.4.4 require adjustments of the calibration constants 'a' and 'b' of the model in order to improve the agreement with the real-world behaviour, because the actual measure of this agreement is 0.7 (average AGREE), i.e. 70% of agreement.

In addition, the model gives 0% UnCrJ which is coincident with the real-world cracking pattern observed in the Belgian JPCP, i.e. ≥ 1 year, when the crack pattern is fully developed and then it is the crack pattern of the in-service performance of the JPCP.

As mentioned in section 5.4.3.2, similar values of the AvCW1st after 60 [hrs] and 500 [hrs] for the slab length 4.5 [m] (Table 5.4.4) and for the slab length 5.5 [m] (Fig. 5.4.5) are obtained.

Adjustment of the calibration constants

After an iteration process with different values of the calibration constants 'a' and 'b', the results converged (improved AGREE for different timing) with the values 'a = 1.1' and 'b = 2.3'. Actually, the Fig. 5.4.6 shows the crack width development for slab length 5.5 [m] with the calibrated model. In the graph the AvCW1st at 60 [hrs], 500 [hrs] and after 1 [year] are indicated.

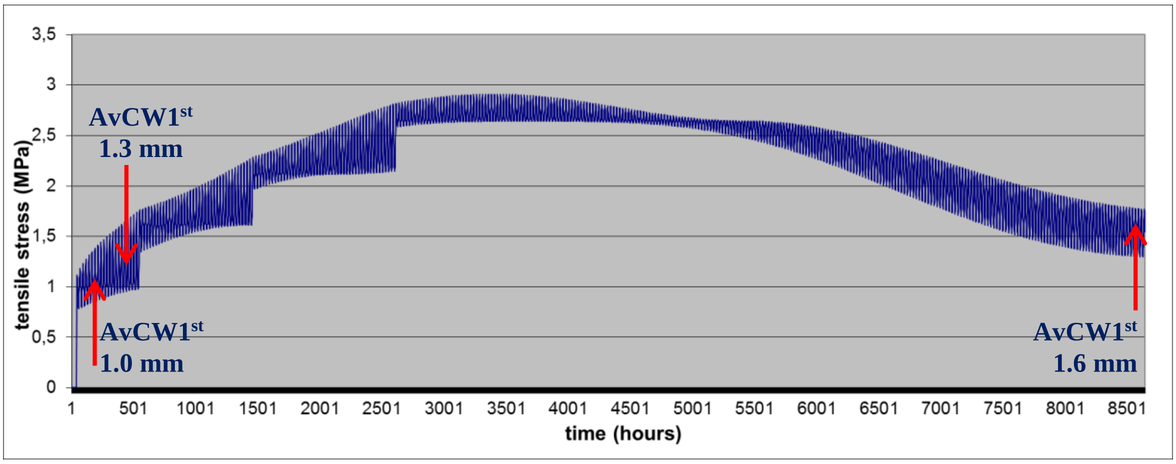

Fig. 5.4.6. Post–calibration crack width development (1st series of cracks) for slab length 5.5 [m] (the red arrows show where the AvCW1st is obtained)

The Table 5.4.5 presents the results of the real–world AvCW1st and the modelled AvCW1st already calibrated.

Table 5.4.5. Real-world AvCW1st, modelled AvCW1st (calibrated) and AGREE value

ID test section	Time	AvCW1st (mm)		AGREE
		Field	Model	
U1	60 hrs	1.1	1.0	1.1
U1	500 hrs	1.2	1.3	0.9
I1	> 1 year	1.6	1.6	1.0

The agreement certainly has been improved, from 0.7 AGREE (average) in the pre-calibrated state to 1.0 AGREE (average) post-calibration. The Table 5.4.6 presents the pre-calibration and post-calibration individual AGREE values for the different times considered in the calibration.

The calibrated model also gives 0% UnCrJ which is coincident with the real-world cracking pattern observed in the Belgian JPCP, i.e. ≥ 1 year, when the crack pattern has fully developed. Hence, although the calibration is based in the AvCW1st, the model gives a correct representation of the fully developed cracking pattern, which is the crack pattern of the in-service performance of the JPCPs.

Table 5.4.6. Pre-calibration and post-calibration AGREE values (and improvement of AGREE**)** for the times considered in the calibration

ID test section	Time	AGREE (pre-calibration)	AGREE (post-calibration)	Improvement AGREE
U1	60 hrs	0.8	1.1	0.3
U1	500 hrs	0.7	0.9	0.2
I1	> 1 year	0.6	1.0	0.4

As mentioned in section 5.4.3.2, the calibrated model also gives similar values of the AvCW1st after 60 [hrs] and 500 [hrs] for the slab length 4.5 [m] (Table 5.4.5) and for the slab length 5.5 [m] (Fig. 5.4.6).

Further analyses are performed in section 5.5 with the model already calibrated. Actually the title of the section 5.5 is 'Post-calibration analyses'.

5.4.3.5. Sensitivity analysis of the friction value

As mentioned in Chapter 4 the friction value 1.0 assumed by Houben (2008a, 2008b) is similar to the friction value applied in different investigations regarding to JPCPs, such as the FHWA model (Smith et al, 1990; Ruiz et al, 2006), the AASHTO model (AASHTO, 1993) and the investigations about UnCrJ of Lee & Stoffels (2003) and Beom & Lee (2007). In the present section the friction value assumed by Houben (2008a, 2008b) is incremented by 50% and 100% according to the friction values applied by Huang (2004) and Lim & Tayabji (2005) respectively. The results obtained are presented in Fig. 5.4.7. For comparison purposes the friction value 1.0 has been added to the results of AvCW1st (Fig. 5.4.7 left) and percentage of joints cracked (Fig. 5.4.7 right).

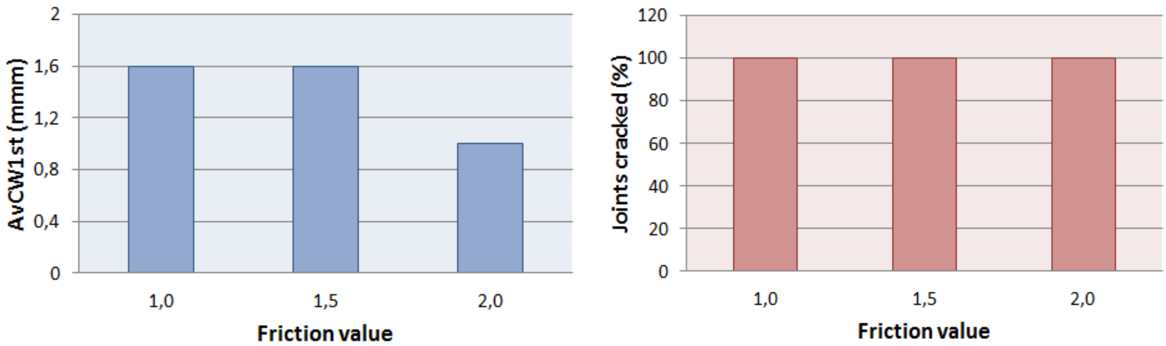

Fig. 5.4.7. AvCW1st (mm) and Joints Cracked (%)
for increments of 50% and 100% of the friction value.

The values of AvCW1st and UnCrJ are still the same when the friction value is increased by 50%. However, a difference is obtained in the AvCW1st (but not in the UnCrJ) when the friction value is incremented by 100%. This confirms that the complex modelling of the cracking process of JPCPs is a topic, although big progress has been made (see chapters 2, 3 and 4), that is far from complete and further research is needed (for instance, a more accurate definition of the friction value). In fact, although the system approach applied in this research makes a difference with the existing models[5], still simplifications of the complex cracking process of JPCPs have been made (as it has been declared) and the model is perfectible. Actually the Fig. 5.4.8 shows how in the present research a calibration level has been established according to the intended uses of the model (Fig. 5.4.2) leaving open the option for continuous improvements (ISO, 2008), i.e. changes in the model (as friction for instance) and new calibration processes (new verification, validation and adaptation phases).

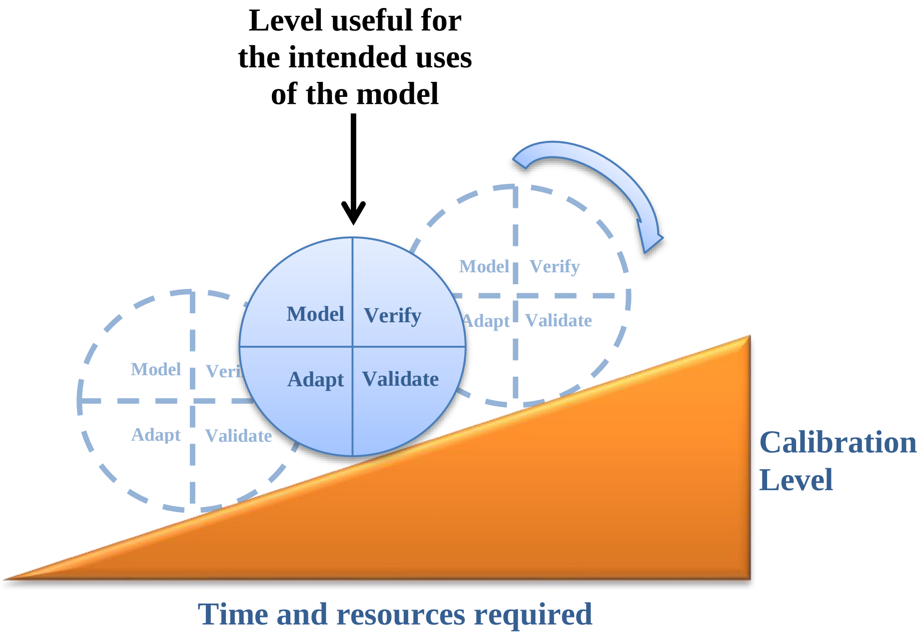

Fig 5.4.8. Continuous improvement principle applied to the calibration process. (Adapted from ISO 9001, 2008)

Nevertheless, as it has been mentioned during the course of this thesis, the pavement clients' satisfaction is also priority in the present research. Accordingly, practical and useful methods for pavement clients are required. Consequently, a practical and useful calibration procedure has been adopted. Hence, nowadays it is possible to adjust the calibration constants of the model to the real-world AvCW1st. This allows adapting the model to the particular characteristics of the pavements under study, for instance the JPCPs of specific geographical regions and/or JPCP applications. This practical calibration procedure certainly is useful for the intended uses of the model in the present investigation and for pavement clients (as transportation agencies). Furthermore, this practical procedure can also be useful for others organisations related with the design of JPCPs.

In particular, the calibration constants 'a=1.1' and 'b=2.3' have been adjusted using this practical calibration procedure (with friction value 1.0) and this calibrated model is the one used in the post-calibration analyses presented in section 5.5, where all the test sections have granular bases.

[5] In the system approach the cracking process of JPCPs is not only time-dependent but also space-dependent (see chapters 2 and 3) and it includes the viscoelastic concrete behaviour as well. Furthermore, the system approach also considered the saw-cutting method applied (specific analyses in section 5.5.2).

5.5. POST-CALIBRATION ANALYSES

5.5.1. Introduction

With the calibrated model, the following further analyses are performed:

- Modelling under different conditions and comparison with real-world AvCW1st of the 4 main JPCP applications, i.e. urban, interurban, industrial and airports
- Studying the influence of the Relative Joint Depth, RJD

In effect, in section 5.5.2 the (calibrated) model results for different conditions are compared with real-world AvCW1st of the 4 main JPCP applications. These comparisons include traditional and short slabs JPCPs, with continuity of the pavement structure (and enough length for the cracking pattern to develop) but also test sections with restricted length. In order to make the comparisons, the parameters $T_{ampyear}$ and T_{ampday}, used in the model (chapters 3 and 4) are determined according to the procedure indicated in section 5.3.3. In addition, as all the test sections have a granular base, the friction value applied in the modelling is 1.0 according to what was presented in sections 5.4.3.2 and 5.4.3.5.

Moreover, in section 5.5.3 the effectiveness of the RJD is evaluated taking advantage that the system approach applied in the present research incorporates the RJD as one of the variables affecting the cracking process of JPCPs. Actually, the importance of this evaluation is based on the fact that RJD is one of the most influential variables possible to control externally in a comparative simpler and economical way.

5.5.2. AvCW1st in the 4 main JPCPs applications

5.5.2.1. Urban JPCPs

The Chilean urban traditional JPCP used in the adaptation phase is part of a bus corridor with continuity of the pavement structure at the beginning and the end of the (100 m) test section and enough length for the cracking pattern to develop, i.e. the conditions for which the model was originally developed. However, in cities it is possible to find JPCPs where the situation can be different. In fact, the test sections ‘U2a’ and ‘U2b’ of the Table 5.3.1 are short streets known as ‘cul-de-sac’ or dead-end roads. The characteristics of these test sections are presented in the Table 5.5.1. Regarding to the Table 5.3.1 the saw-cutting depth and the slab thickness have been replaced by the Relative Joint Depth (RJD) which is actually the ratio of them.

Table 5.5.1. Characteristics of the urban test sections ‘U2a’ and ‘U2b’

ID test Section	Length section (m)	RJD (%)	Slab length (m)	Base	Concrete grade
U2a	70	30	3.50	Granular	C28/35
U2b	70	30	1.75	Granular	C28/35

The Table 5.5.2 presents the results of the AvCW1st obtained from the field and the modelling (built at summer 4 pm, $T_{ampyear}$ = 10^{o}C and T_{ampday} = 6^{o}C). It can be observed that the model is able to predict the real-world AvCW1st.

Table 5.5.2. Comparison of real-world AvCW1st and modelled AvCW1st

ID test section	Time (hrs)	AvCW1st (mm)	
		Field	Model
U2a	60	0.46	0.48
U2b	60	0.23	0.24

Similar to Chapter 4 the AvCW1st of the short slab section (U2b) was calculated with the model considering a reduction of 50% of the crack width (0.24 mm) compared to the one of the traditional JPCP (U2a) with double slab length (0.48 mm for slab length 3.50 m). As in Chapter 4, this reduction was based on the work done by AASHTO (1993) and NCHRP (2003) and it is actually confirmed by the AvCW1st obtained from the field for the traditional JPCP (0.46 mm) and the short slabs JPCP (0.23 mm) when the slab length is reduced to 50%, i.e. 3.50 [m] for the traditional JPCP and 1.75 [m] for the short slabs JPCP. Indeed, it is a field corroboration because these test sections correspond to 2 adjacent streets of a residential area, they have the same characteristics (Table 5.5.1), with the exception of the slab length, and they were built under the same climactic conditions and time of the day (Pradena & Houben, 2016b).

Moreover, the test section U3 of the Table 5.3.1 also has restricted length in terms of the absence of continuity in the pavement structure at the beginning and/or the end of the test section. This test section is shown in the Fig. 5.5.1. Furthermore, the Table 5.5.3 presents the characteristics of the test section.

Fig. 5.5.1. Paving with fixed formwork (left) and general view of the test section (right).

Table 5.5.3. Characteristics of the urban test section ‘ ‘U3’

ID test section	Length section (m)	RJD (%)	Slab length (m)	Base	Concrete grade
U3	70	35	3.50	Granular	C28/35

The results of the AvCW1st obtained from the field is 1.0 [mm] (after 100 hrs) which is the same AvCW1st predicted by the model (built at summer 4 pm, $T_{ampyear}$ = 10ºC and T_{ampday} = 5ºC).

5.5.2.2. Interurban JPCPs

Besides the Belgian interurban traditional JPCP on the N74, used in the adaptation phase, the test sections 'I2a' and 'I2b' of the Table 5.3.1 are part of an interurban (short slabs) JPCP located in the Province of 'Tierra del Fuego' in Chile (Fig. 5.5.2).

Fig. 5.5.2. Paving with slipformer (left) and (early-entry) saw-cutting of the transverse joints (rigth)

The Table 5.5.4 presents the characteristics of the test sections. Regarding to the Table 5.3.1 the saw-cutting depth and the slab thickness have been replaced by the Relative Joint Depth (RJD) which is actually the ratio of them.

Table 5.5.4. Characteristics of the interurban test sections 'I2a' and 'I2b'

ID test section	Length section (m)	RJD (%)	Slab length (m)	Base	Concrete grade
I2a	100	35	2.00	Granular	C35/45*
I2b	100	35	2.00	Granular	C35/45*

(*) Includes synthetic fibres type Barcship-54 (2.5 Kg/m^3 of concrete)

The Table 5.5.5 presents the results of the real-world AvCW1st and the one from the modelling (built at summer 4 pm, $T_{ampyear}$ = 10^{o}C and T_{ampday} = 6^{o}C).

Table 5.5.5. Comparison of real-world AvCW1st and modelled AvCW1st

ID test section	Time (hrs)	AvCW1st (mm)	
		Field	Model
I2a	80	0.20	0.22
I2b	70	0.25	0.23

The Table 5.5.5 shows similar results of the real-world AvCW1st and the ones predicted by the calibrated model.

5.5.2.3. Industrial facilities

The JPCPs in industrial facilities, as industrial floors and yards, can present length restrictions due to the layout of the industrial complex, the warehouses dimensions, discontinuities as offices, etc. As mentioned, these conditions are not the ones of which the model was originally developed. Hence, it is important to compare the AvCW1st predicted by the model with the real-world AvCW1st. In particular, the test sections 'Ind-a' and 'Ind-b' of the Table 5.3.1 are considered for this purpose. In effect, these test sections have only 24 [m] length each. The rest of the characteristics of the test sections 'Ind-a' and 'Ind-b' are given in the Table 5.5.6.

Table 5.5.6. Characteristics of the industrial test sections 'Ind-a' and 'Ind-b'

ID test section	Length section (m)	RJD (%)	Slab length (m)	Base	Concrete grade
Ind-a	24	25	2.00	Granular	C28/35
Ind-b	24	25	2.00	Granular	C28/35

The Fig. 5.5.3 shows the industrial floor where the test sections 'Ind-a' and 'Ind-b' are located. The Fig. 5.5.3 (left) presents an example of discontinuities of the industrial floor caused by the industrial layout. This kind of configurations result in short JPCP sections.

Fig. 5.5.3. Construction of industrial floor with shorter irregular sections (left) and general view of the industrial floor (right)

The Table 5.5.7 presents the results of the AvCW1st obtained from the field and the modelling (built at summer 4 pm, $T_{ampyear}$ = 10°C and T_{ampday} = 5°C). Again it can be observed that the model is able to predict correctly the real-world AvCW1st.

Table 5.5.7. Comparison of real-world AvCW1st and modelled AvCW1st

ID test section	Time (hrs)	AvCW1st (mm)	
		Field	Model
Ind-a	72	0.52	0.50
Ind-b	216	0.58	0.54

5.5.2.4. Airport aprons

The Fig. 5.5.4 shows the apron of the main Chilean airport where the test section 'A1' of the Table 5.3.1 was located. The cause of the restricted length (30 m) of the test section 'A1' was the necessity to continue the airport operations during the construction of the JPCP. Because of that, the replacement of the old concrete slabs by the new JPCP could only be done in a limited area each time. Actually in the Fig. 5.5.4 part of this closed work area and the airplanes on the apron just outside the fence can be seen. Furthermore, the Table 5.5.8 presents the characteristics of the test section 'A1'.

Fig. 5.5.4. Paving with fixed formwork (left) and general view of the test section (right).

Table 5.5.8. Characteristics of the airport test section 'A1'

ID test section	Length section (m)	RJD (%)	Slab length (m)	Base	Concrete grade
A1	30	25	4.0	Granular	C28/35

The real-world AvCW1st is 0.9 [mm] (after 120 hrs) which is very similar to the 1.0 [mm] predicted by the model (built at summer 4 pm, $T_{ampyear}$ = 10^{o}C and T_{ampday} = 5^{o}C).

5.5.3. Influence of the relative joint depth

One of the advantages of the system approach applied in the present investigation to model the cracking process of JPCPs, is the possibility to evaluate the effectiveness of the Relative Joint Depth (RJD) which is the ratio between the saw-cut depth and the thickness of the JPCP.

In the simulations presented in Chapter 4 it was already identified that the RJD has an influence on the value of the AvCW1st and on the presence of UnCrJ. In fact, the RJD is one of the most influential factors possible to control in order to obtain the desired results of AvCW1st and UnCrJ. On the other hand, the calibrated model has shown a very good behaviour when it was compared with the field reality (section 5.5.2). Hence, in the present section the calibrated model is applied to analyse the effects of different RJDs in traditional JPCPs with slab length L = 5 [m] and short slabs with 0.5L = 2.5 [m]; i.e. the same case as presented in Chapter 4. In fact, the analysed scenarios of time of construction are summer (4 pm) and winter (10 am), and the results given by the model before and after its calibration are presented and compared.

5.5.3.1. Construction in summer (4 pm)

Although the calibration was based in the AvCW1st, the model gave a correct representation of the fully developed cracking pattern as it was presented in section 5.3.3.3. And, in the present section, the cases of UnCrJ analysed are also with the cracking pattern fully developed (1 year after the construction of the JPCP). The Table 5.5.8 presents the cases of RJD 30% and 35%. The values in black apply to the traditional JPCP with slab length L = 5 [m], the values in red apply to the short slabs JPCP with slab length L = 2.5 [m].

Table 5.5.8. Pre and post calibration results of AvCW1st and UnCrJ for traditional and short slabs JPCPs (construction in summer 4 pm).

Saw-cutting (RJD %)	Friction	Concrete grade	Season and time of construction			
			Summer 4 pm (pre-calibration)		Summer 4 pm (post-calibration)	
			AvCW1st (mm)	UnCrJ (%)	AvCW1st (mm)	UnCrJ (%)
30	**1.0**	**C28/35**	2.2/1.1	0/0	1.8/0.9	36/67
		C35/45	3.4/1.7	0/48	2.2/1.1	36/67
35	**1.0**	**C28/35**	2.0/1.0	0/0	0.9/0.5	0/0
		C35/45	2.2/1.1	0/0	1.4/0.7	0/48

The values of AvCW1st are smaller in the post-calibrated model. In fact, in all cases presented in Table 5.5.8, AvCW1st ≤ 1.1 [mm] for short slabs JPCPs, being 1.1 [mm] the crack width limit for adequate LTE by aggregate interlock (Davids & Mahoney, 1999). And, in the traditional JPCP the post-calibrated model gives one case with AvCW1st ≤ 1.1 [mm] unlike the pre-calibrated calculations where there were none.

It is also possible to observe that for concrete grade C28/35, in both traditional and short slabs JPCPs, a RJD = 35% is required to obtain AvCW1st < 1.1 [mm] and 0% UnCrJ.

When concrete grade C35/45 is applied, the RJD = 35% results in 0% UnCrJ in traditional JPCPs. However, it is not able to produce AvCW1st ≤ 1.1 [mm]. Still, in traditional non-dowelled JPCPs, the AvCW1st = 1.4 [mm] could provide adequate LTE provided that high quality aggregates are applied (see Chapter 6 for more details about LTE with high quality aggregates).

In the case of short slabs JPCPs with concrete grade C35/45, the RJD = 35% produces AvCW1st < 1.1 [mm]. However, the percentage of UnCrJ is 48% (more joints to activate than in the case of traditional JPCPs). The presence of UnCrJ is particularly relevant in short slabs JPCPs because their postulated benefits depend on the fact that the slabs are effectively shorter. And when the joints remain uncracked, the effective slab length is not short anymore. In order to avoid the UnCrJ, further calculations were made with the calibrated model and larger RJD.

With RJD 40%, the AvCW1st = 1.0 [mm] for traditional JPCPs and AvCW1st = 0.5 [mm] for short slabs JPCPs. As it could be expected, the traditional JPCP still have 0% UnCrJ. And now the short slabs JPCPs have 0% UnCrJ as well. Pradena & Houben (2014) also report 0% UnCrJ in 1 section of 47 joints of short slabs with similar conditions to the modelled ones and 35% < RJD ≤ 40%.

5.5.3.2. Construction in winter (10 am)

As it was presented in Chapter 4, the construction in winter (10 am) leads to a higher percentage of UnCrJ and it can be reduced through a larger RJD. Considering these aspects and the fact that with RJD = 35% already the percentage of UnCrJ is over 70% (Chapter 4), the analyses of the present section start with RJD = 35%.

Table 5.5.9. Pre and post calibration results of AvCW1st and UnCrJ for traditional and short slabs JPCPs (construction in winter 10 am).

Saw-cutting (RJD %)	Friction	Concrete grade	Season and time of construction			
			Winter 10 am (pre-calibration)		Winter 10 am (post-calibration)	
			AvCW1st (mm)	UnCrJ (%)	AvCW1st (mm)	UnCrJ (%)
35	**1.0**	**C28/35**	1.5/0.8	73/86	0.4/0.2	67/82
		C35/45	1.7/0.9	83/91	1.0/0.5	80/89

The calibrated model gives smaller values of AvCW1st. Indeed, in the pre-calibrated calculations only the short slabs have AvCW1st < 1.1 [mm]. However, the calibrated model gives AvCW1st < 1.1 [mm] for both traditional and short slabs JPCPs.

The high percentage of UnCrJ is common in the pre and post calibrated calculations. As with larger RJD the percentage of UnCrJ is reduced, further calculations were made with the calibrated model and larger RJD values.

For concrete grade C28/35, the UnCrJ is 0% in traditional JPCPs when the RJD is 50%. However, in short slabs JPCPs a RJD = 55% is required to obtain 0% UnCrJ. In effect, the short slabs have more joints to activate.

For concrete grade C35/45, the UnCrJ is 0% in traditional JPCPs when the RJD is 50%. However, in short slabs JPCPs a RJD = 60% is required to obtain 0% UnCrJ.

RJDs 50%, 55% or 60% are not difficult to obtain in practice using a conventional sawer with thin blade (≤ 3 mm) associated to the unsealed joints studied in the present thesis (chapters 2 and 7). In fact, Pradena & Houben (2016) corroborate this in 12 (unsealed) joints of a JPCP built in winter in Chile. In this field experience, the saw-cut operator had not specific instruction about the RJD, more than the common one-third of the JPCP thickness. However, the 12 joints have RJD ≥ 50% and the average RJD was 72% (Fig. 5.5.5).

Although a deeper RJD reduces the area available for aggregate interlock, it also produces very thin cracks because the cracks are produced earlier and because of the absence of UnCrJ. Indeed for all the cases with RJD ≥ 50% (for both concrete grades) the $AvCW1^{st} \leq 0.4$ [mm] for traditional JPCPs and $AvCW1^{st} \leq 0.2$ [mm] for short slabs JPCPs. According to Davids & Mahoney (1999) the LTE by aggregate interlock associated to these crack width is ≥ 90%. Furthermore these RJDs produce 0% UnCrJ, i.e. the 'as-built' slab length is the same as the designed slab length, which is fundamental especially in short slabs JPCPs.

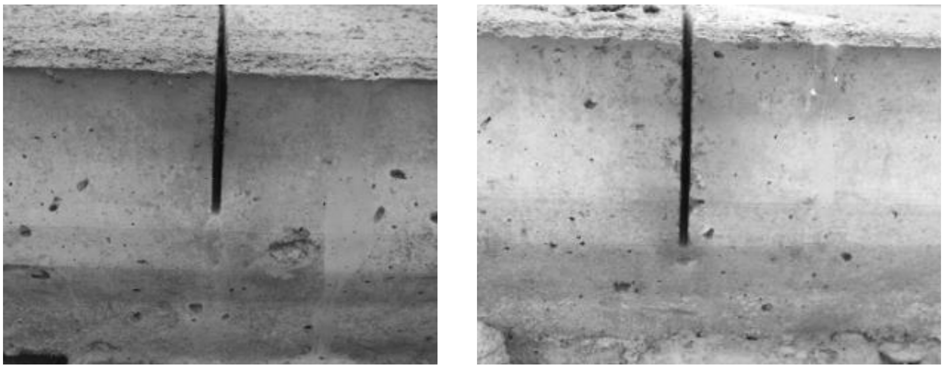

Fig. 5.5.5. Joint with RJD 50% (left) and joint with RJD 65% (right)
(Pradena & Houben, 2016)

5.5.3.3. Recommendations

Due to different reasons (economic, weather, work space availability, etc.) summer is a typical season of construction of JPCPs. However, it produces the widest cracks, i.e. the most unfavourable condition for the provision of LTE by aggregate interlock which is especially relevant in non-dowelled short slabs JPCPs. In this context, the practical recommendation is to regulate the RJD (i.e the ratio of the saw-cut depth and the JPCP thickness) in order to obtain $AvCW1^{st} \leq 1.1$ [mm] (and 0% UnCrJ). This practical recommendation is given considering the influence of the RJD and the possibilities to regulate it in a simple and economical way in the construction process of JPCPs.

In short slabs JPCPs the use of Early-Entry Saw-Cutting (EESC), which produces a shallow cut up to 30 [mm], is a common practice. Although the EESC is very useful to relieve internal tensions in the early-age concrete, there are some concerns about it effectiveness in the long-term performance of JPCPs, for instance due to the presence of UnCrJ. Actually, in an investigation of the Louisiana Transportation Research Center, the joints originally saw-cut with EESC, needed to be sawn deeper with conventional equipment to assure the joint activation (Rasoulian et al, 2005). Moreover, a research of the Illinois Center for Transportation recommends a phase II of the study in order to evaluate the effects of the EESC on the JPCP long-term performance under more unfavourable conditions because the first phase was based on very limited data collected from a single site built under very favourable conditions (Krstulovich et al, 2011). The recommendation of the present research (according to the modelled trends presented) is in the same direction, i.e. further investigations of the joint activation when EESC is applied, especially in short slabs JPCPs, where the postulated structural and functional benefits of this innovation require that the slabs are effectively short.

As the EESC is useful to relieve internal tensions in the early-age concrete, one possibility is to apply the EESC in combination with the conventional saw-cutting method, i.e. some joints can be sawn with EESC to relieve the tension at very early-age. Afterwards, the rest of the joints can be saw-cut with conventional equipment. And, a deeper second saw-cut can be applied with conventional equipment in the joints where the EESC was applied.

Another practical alternative observed in the field is the modification of the EESC equipment made by the contractor in order to produce deeper saw-cuts. Actually, that was the case of the short slabs JPCP mentioned in section 5.5.2.1 where RJD > 35% produced 0% UnCrJ (Pradena & Houben, 2014). In that case, only 1 saw-cut at very early-age was enough to reach the desired RJD.

Although favourable from the AvCW1st point of view (thin cracks), the construction in winter produces a high percentage of UnCrJ and deeper RJDs are required. These deeper saw-cuts are not necessarily difficult to perform in the field when a thin blade using conventional saw-cutting equipment is applied (Pradena & Houben, 2016). As the model has been calibrated with test sections built in summer, further specific field research with JPCPs built in winter is recommended. Meanwhile, the trends given by the model can be used as a reference.

5.6. CONCLUSIONS AND RECOMMENDATIONS

5.6.1. Conclusions

In order to obtain realistic values of the relevant results of the early-age concrete behaviour, the model of the cracking process of JPCPs has been calibrated with a procedure that considers the intended uses of the model (AvCW1st for the link with the in-service JPCP performance) and the necessity of being practical and useful for pavement clients as public agencies related with different JPCP applications (as urban, interurban, airports). In addition, it is a progressive calibration process with 3 phases; the 'verification' phase, next the 'validation' phase and finally the 'adaptation' phase. In the 'verification' phase, the special case of the RF was distinguished not only for it influence on the cracking process of JPCP, but also due to the lack of experimental data and the significant differences between the RFs when they were compared during a period of time (1 year) useful for the objectives of the present research. With this information from the 'validation' phase, a general equation of RF was proposed in the 'validation' phase. This general RF equation included the calibration constants 'a' and 'b' to be adjusted in the next phase of 'adaptation' of the model. In addition, in the 'validation' phase also further analyses of the cracking process of concrete pavements were made and their results compared with preliminary field inspections. The results of those analyses confirmed that the field measurements in the next phase of the calibration procedure (i.e. the 'adaptation' phase) should be concentrated in the transverse joints of JPCPs. This not only because their AvCW1st is directly related with the LTE of the predominant traffic direction, but also due to the very thin cracks under the longitudinal joints (if actually a crack is produced) and finally, due to the fact that the transverse joints are more available and easier accessible in the field than the longitudinal ones (if the JPCP actually has longitudinal joints).

The model was calibrated considering 2 fundamental 'windows' of the crack width development. The 1st one, at the early-age when all the changes in the concrete are developing (Chilean JPCP section) and the 2nd one after ≥ 1 year of the JPCP construction, when the crack width behaviour is more stable (Belgian JPCP section) and it can be linked with the LTE of the in-service JPCP. The calibration constants ‘a’ and ‘b’, originally with default values 1, were adjusted to the values ‘a = 1.1’ and ‘b = 2.3’ in order to improve the agreement between the modelled and the real-world AvCW1st. Actually this agreement was improved from 0.7 AGREE (average) before the calibration to 1.0 AGREE (average) after the calibration.

Although the calibration was based on the AvCW1st, the model gave a correct representation of the fully developed cracking pattern (≥ 1 year) in the Belgian JPCP.

10 test sections were considered to compare the modelling results with the real-world behaviour of the JPCPs. 2 test sections of traditional JPCPs (1 in Belgium and 1 in Chile) were used in the adaptation phase and 8 test sections located in Chile were considered in the post-calibration analyses. Between these 8 test sections, 3 correspond to traditional JPCPs and 5 to short slabs JPCPs. Besides, 3 of these 8 test sections are urban JPCPs, 2 interurban JPCPs, 2 test sections are on an industrial facility and 1 on an airport apron.

In particular, the post-calibrated analyses show the good behaviour of the calibrated model (in traditional and short slabs JPCPs). In particular, the model has been able to predict correctly the AvCW1st in the 4 main JPCPs applications, i.e. interurban, urban, industrial and airports. Furthermore, the model has been able to predict correctly the AvCW1st in the cases of restricted length of short urban streets, industrial floors with irregular layout, and an airport apron with restricted length caused by the necessity to continue the airport operations during construction.

As part of the post-calibration analyses, the reduction of 50% of crack width in short slabs with respect to the traditional JPCPs (when the slab length is also reduced 50%) was confirmed in real-world traditional and short slabs JPCPs.

Additional post-calibration analyses were performed in order to analyse the influence of the Relative Joint Depth (RJD). These post-calibration analyses confirmed that the RJD is one of the most influential variables possible to regulate in a practical, simple and economical way in the construction process in order to obtain the desired results of AvCW1st and UnCrJ. For the analysed conditions, the RJD must be between 35% and 40% in order to obtain AvCW1st ≤ 1.1 [mm] and 0% UnCrJ when the JPCP construction is done in summer (4 pm). With the same purpose, the construction in winter (10 am) requires deeper RJDs (between 50% and 60%) because the temperature increases shortly after the period of construction. On the contrary, in summer, after the JPCP is built the concrete shrinkage goes together with a temperature decrease.

From the post-calibration analyses of the influence of the RJD, also recommendations were obtained (section 5.6.2).

5.6.2. Recommendations

The modelling of the cracking process of JPCPs is a complex subject with numerous variables involved. It is a topic where, although big progress has been made (see chapters 2, 3 and 4), it is far from complete and further research is needed. In the present thesis, this has been explicitly acknowledged, and a calibration level has been established according to the intended uses of the model leaving open the option for continuous improvements (as changes in the model and new calibration processes). However, at the same time the calibration procedure of the model in the present research has the characteristic of being practical and useful for pavement clients (as transportation agencies). In this way, nowadays it is possible to adjust the calibration constants of the model to the real-world AvCW1st. This allows adapting the model to the particular characteristics of the pavements under study, for instance the JPCPs of specific geographical regions and/or particular JPCP applications.

In comparison with the calibration procedure described in this chapter, a simpler procedure focussing on in-service JPCPs (≥ 1 year after their construction) is proposed particularly for public agencies, with the objective to incorporate the direct cause of the LTE by aggregate interlock in their pavement design manuals. This simpler alternative is proposed considering the good behaviour of the calibrated model, the several successful field corroborations already made at early-ages, and the fact that the intended use of the model is the prediction of the AvCW1st to be linked with the in-service JPCP performance. In this way, the measurements of the AvCW1st can be made on the JPCPs representative for the conditions of interest with a simple fissurometer (Fig. 5.3.2 left). If the measured results are similar to the model predictions the process stops there. Otherwise, the calibration constants of the model can be adjusted according to the real-world AvCW1st.

In the present chapter different calculations have been made with the calibrated model, in particular for JPCPs built in summer and winter. Further calculations can be made for other realities of interest with the model details presented in Chapter 3 (and Appendix A).

Although construction in summer produces the widest cracks, so the most unfavourable condition of AvCW1st, it is a typical season of construction of JPCPs due to different reasons (economic, weather, work space availability in holidays, budget availability, etc.). Considering this reality, still it is possible to obtain AvCW1st ≤ 1.1 [mm] (and 0% UnCrJ), controlling the RJD which is one of the most influential variables possible to regulate in a practical, simple and economical way in the construction process.

Although favourable from the AvCW1st point of view (thin cracks), the construction in winter produces a high percentage of UnCrJ and then larger RJDs are required. These deeper saw-cuts are not necessarily difficult to perform in practice when thin blades using conventional saw-cutting equipment are applied, as it is the case of the unsealed joints studied in the present investigation. As the model has been calibrated with test sections built in summer, further specific field research at JPCPs built in winter is recommended.

The EESC produces a shallow cut up to 30 [mm] and it is is a common practice especially in short slabs JPCPs. The recommendation of the present research is similar to previous ones, i.e. before a general adoption of the EESC, further field investigations are necessary in order to evaluate the real potential of the EESC to activate the joints in different conditions, especially the most unfavourable ones. This recommendation is particularly relevant for short slabs JPCPs where the postulated structural and functional benefits of this innovation require that the slabs are effectively short (i.e. activated joints).

As EESC is very effective to relieve internal tensions in the early-age concrete, one possibility is to apply EESC in combination with the traditional saw-cutting method. Another successful alternative observed in the field is the modification of the EESC equipment to produce deeper saw-cuts.

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6. STRUCTURAL ANALYSIS OF JPCPs

6.1. INTRODUCTION

The third specific objective of the present thesis is to 'Analyse the structural performance of JPCPs including the relevant results of the early-age behaviour'. As mentioned in previous chapters, these relevant results for non-dowelled traditional and short slabs JPCPs are primarily the crack width under the joints and secondly the presence of Uncracked Joints (UnCrJ). The crack width at joints is the main link between the early-age behaviour and the in-service performance of non-dowelled JPCPs. This link is expressed in the relation between the crack width and the Load Transfer Efficiency (LTE) which allows the explicit incorporation of the effects of the concrete behaviour since early-age in the design of non-dowelled JPCPs. As important as it is, in general the Mechanistic-Empirical (M-E) design methods of JPCPs do not include it.

The M-E design methods that include the LTE do not make a direct relation with the most influential load transfer mechanism in non-dowelled JPCPs which is the aggregate interlock (Buch et al, 2000; Hanekom et al, 2003; IPRF, 2011) and thus the crack width under the joints.

More than a method of pavement design, the Mechanistic-Empirical Pavement Design Guide (MEPDG) is a detailed procedure of structural pavement analysis. As detailed as MEPDG is, it includes a simplified formula for the calculation of the joint opening (NCHRP, 2003) with fixed mean values of shrinkage and thermal deformations instead of the development in time of them, the concrete properties and the specific construction conditions of JPCPs as in the present thesis. Furthermore, MEPDG does not include directly the UnCrJ phenomenon. On the contrary, in the present thesis this phenomenon is incorporated due to the treatment of the pavement as a system where the development and final state of the joints is not only the result of the material changes but also of the location of the series of cracks in time. In this way, it is possible to know the joints that remain uncracked.

The study of non-dowelled short slabs JPCPs has been focussed on their structural analysis (Salsilli & Wahr, 2010; Covarrubias, 2008, 2011, 2012; Salgado, 2011; Roesler et al, 2012; Chilean Highway Agency, 2012; Salsilli et al, 2013, 2015). Using dimensional analysis (Ioannides, 1984), Salsilli et al. (2015) developed a practical M-E design method for short slabs with the objective to be available for practitioners. However, Salsilli et al (2015) recognize the necessity to make specific studies of LTE in the non-dowelled short slabs. Actually, although the development of short slabs JPCPs has been concentrated in their structural analysis, specific studies of the relation between the LTE by aggregate interlock with its direct cause (the crack width) were not found in the state of the art of this innovation. Considering that their provision of load transfer relies on aggregate interlock, the characterization of the relation LTE-crack width is fundamental. Therefore the focus of the present chapter is to develop the relation between the LTE by aggregate interlock and its direct cause (the crack width) specifically for the innovative non-dowelled short slabs JPCPs.

The Fig. 6.1.1 presents the structural analysis in the context (and approach) of the present investigation including specific field measurements into the LTE-crack width relation in short slabs JPCPs.

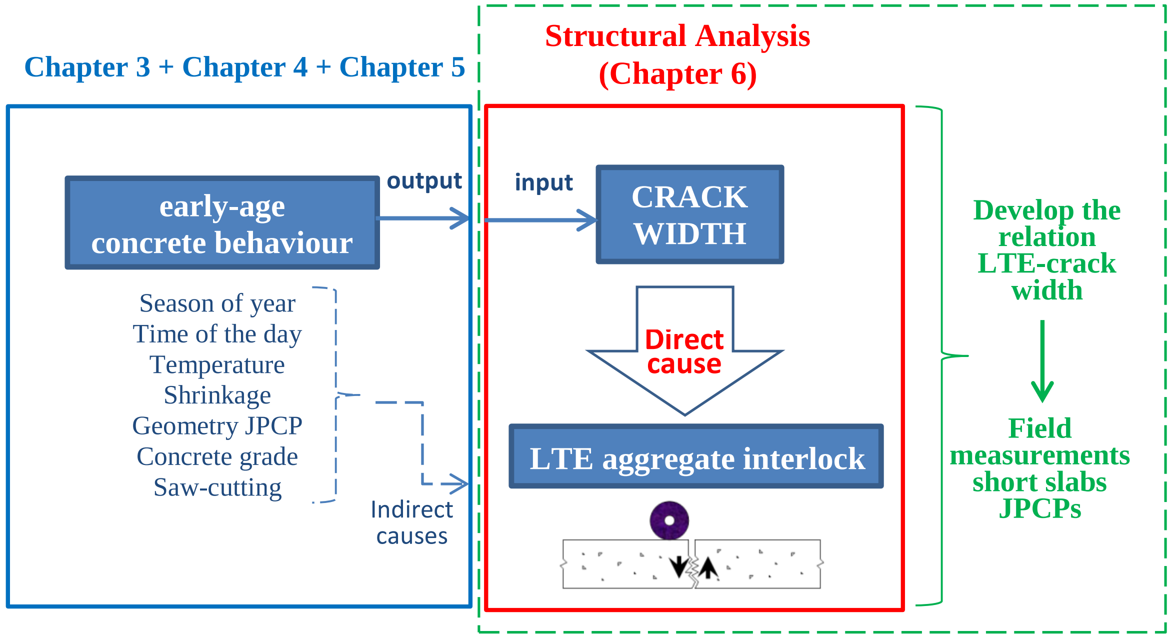


Fig 6.1.1. Structural analysis in the context (and approach) of the present investigation

The Fig 6.1.1 also shows the variables affecting the early-age concrete behaviour which finally determines the crack width that can be directly related with the LTE. Sometimes these variables (part of them) are also used to create relationships with the LTE of the in-service JPCP. However, it is important to highlight that they do not relate the LTE by aggregate interlock with the direct cause of it, but with indirect ones as it is shown in Fig. 6.1.1.

In addition, such an indirect approach does not allow the incorporation of the effects of the early-age concrete behaviour in the pavement design (which is the objective of this chapter) in a direct and effective way useful and relevant to the pavement clients.

Furthermore, in general these indirect relationships consider the effect of one variable on the LTE, neglecting the effects of the other variables (Fig. 6.1.1). On the contrary, as it has been expressed in the different chapters, the approach of the present thesis is different (Fig. 6.1.1). Indeed, the chapters 3,4 and 5 have been focussed on the analyses of the effects of multiple variables on the crack width. Even more, to avoid neglecting the contribution of the different variables involved, a factorial design was explicitly defined in Chapter 4. In this way, the crack width resulting from the early-age concrete behaviour is the result of the interaction of the variables defined in the factorial design, which included the variables shown in the Fig. 6.1.1. Thus, the main output of such a process is the crack width that can be related with the LTE.

In fact, in the present chapter, the crack width is the main input to develop the relation with the LTE because the crack width is the direct cause of the LTE by aggregate interlock. And when the direct cause of the LTE by aggregate interlock is being specifically investigated (and measured in the field), returning to correlations with indirect variables is not only unnecessary but even confusing for the intended purpose and contribution of the research. Because of that, the analysis and field measurements in the present chapter are explicitly focussed on the direct cause of the LTE by aggregate interlock instead of (returning to) indirect ones.

The use of the direct cause to develop the relation with the LTE allows the incorporation of the early-age concrete behaviour in the structural design in an effective way useful and relevant to increase the satisfaction of the pavement clients. In effect, as mentioned in previous chapters, the present investigation searches for increasing the pavement clients' satisfaction. In particular, the owners or agents acting at their behalf (as transportation agencies) assign priority to the structural adequacy of the pavement (Haas & Hudson, 1996). Hence, the structural analysis needs to be practical and useful for pavement designers and transportation agencies. Therefore, the focus is to incorporate the relevant results of the early-age concrete behaviour (crack width) in practical structural design methods useful for practitioners. Accordingly the emphasis is on M-E pavement design methods instead of a Finite Elements (FE) one because, although FE is an important tool for pavement analysis, it cannot easily be implemented as part of a design method due to the complexity, computational requirements and time of execution

6.2. STRUCTURAL BENEFITS OF REDUCING THE SLAB LENGTH

According to the principle of continual improvement (ISO, 9000) new possibilities of JPCPs have been developed. For instance new technologies associated to the construction and performance of joints of JPCPs. This has allowed to experiment with reductions of joint spacing despite the traditional practice of limiting the number of joints because of the potential distresses and costs of construction and maintenance. In fact, there are structural benefits of reducing the slab length as it is shown in the present section.

Bradburry (1938) used Westergaard's analysis to develop the basic expression of the maximum edge stress due to curling at the mid-span of a finite slab (Eq. 6.2.1).

$$\sigma = \frac{1}{2} * \alpha * E * \Delta t * C \qquad \text{(MPa)} \tag{6.2.1}$$

Where:
- σ = Edge curling stress (MPa)
- α = Coefficient of thermal expansion (1/°C)
- E = Elastic modulus of concrete (MPa)
- Δt = Temperature difference (°C)
- C = Coefficient of curling stress (-)

The coefficient of curling stress C is a function of the radius of relative stiffness and the slab length. The radius of relative stiffness is a function of the concrete layer and subgrade properties. Then, *ceteris paribus*, the shorter the slab length the lower the curling stress.

The Westergaard equation for slab edge stress due to an equivalent tire load includes different limiting assumptions that differ of real-world concrete slabs. In order to address these limitations, Ioannides (1984) outlined a methodology based on the application of the principles of dimensional analysis. This methodology received considerable attention among pavement engineers (Rauhut & Darter, 1990; Raad & Marhamo, 1991; Seiler, 1992) and it has been incorporated into different M-E design procedures (Ioannides 1990, 2006; Ioannides & Salsilli, 1989; Salsilli, 1991; NCHRP, 1992; Cabrera, 1998; Khazanovich et al. 2004; Bordelon et al, 2009, Salsilli et al, 2015; Salsilli, 2015). In particular, to overcome the Westergaard assumption of an infinite slab, Ioannides et al.

(1985); Salsilli (1991); Cabrera (1998) use the Eq. 6.2.2 as adjustment factor (due to the slab size) that multiplies the Westergaard equation for slab edge stress.

$$Fss = 0.65751 - 1.79166 * \left(\frac{a}{l}\right) - 0.00682 * \left(\frac{L}{l}\right)^2 + 0.70807 * \left(\frac{a}{l}\right) * \left(\frac{L}{l}\right) + 0.32194 * Ln\left(\frac{L}{l}\right) - 0.06809 * \left(\frac{a}{l}\right) * \left(\frac{L}{l}\right)^2 \quad (6.2.2)$$

Where: $a =$ Radius of the circular load contact area (m)
$l =$ Radius of relative stiffness of the concrete layer (m)
$L =$ Slab length (m)

Salsilli (2012) shows graphically the effect of slab size reduction on this adjustment factor that multiplies the Westergaard equation for slab edge stress (Fig. 6.2.1).

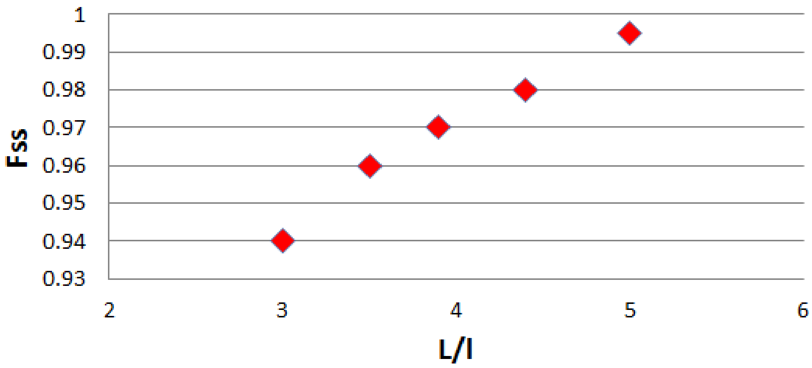


Fig 6.2.1. Variations of the adjustment factor by slab size with slab length (Salsilli, 2012).

In general, on traditional JPCPs the front and rear axles of trucks apply the load simultaneously near the transverse joints. If the slabs are shortened such that the slab length prevents the front and rear axles to be simultaneously on the same slab (Covarrubias et al., 2010), as seen in Fig. 6.2.2, the tensile stresses within the slab are significantly reduced. The stresses calculated in Fig. 6.2.2 are based on a 15 kN load and a -15ºC temperature difference over the slab thickness (Covarrubias et al., 2010).

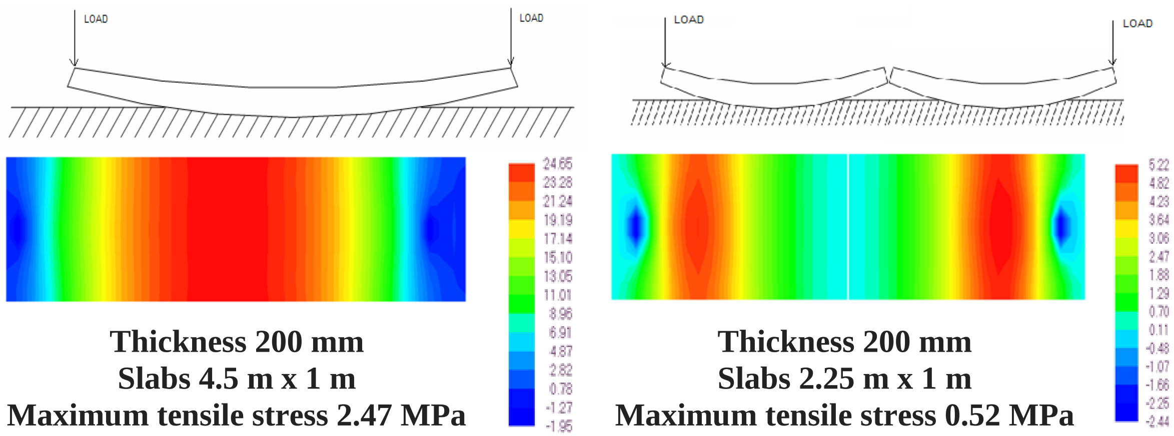


Fig 6.2.2. Tensile flexural stresses in traditional and short slabs with the same thickness (Covarrubias, 2008).

The reduction of the tensile flexural stresses in the slab allows for a longer service life or reduction in the slab thickness as it is shown in Fig. 6.2.3. Covarrubias et al. (2010) calculated the maximum tensile stresses shown in Fig. 6.2.3 based on a -14ºC temperature difference and truck loads of 70 kN for the steering axle and 180 kN for the tandem axle (Covarrubias et al., 2010).

Thickness 250 mm
Slabs 4.5 m x 3.6 m
Maximum tensile stress 2.47 MPa

Thickness 140 mm
Slabs 2.5 m x 1.8 m
Maximum tensile stress 2.44 MPa

Fig 6.2.3. Equivalent thickness for the same tensile flexural stresses in traditional and short slabs (Covarrubias et al., 2010).

The Fig. 6.2.4 shows the required thicknesses for a traditional slab (4.5 m) and a short slab (1.75 m) for the same tensile flexural stresses (Covarrubias, 2008).

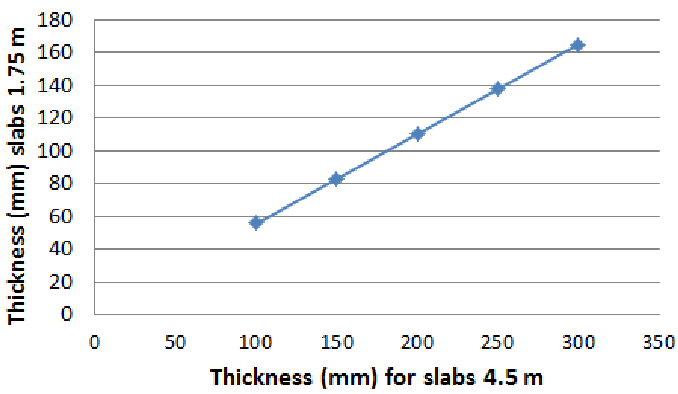


Fig 6.2.4. Equivalent thicknesses of traditional and short slabs JPCPs for the same tension flexural stresses (Covarrubias, 2008).

6.3. JPCP APPLICATIONS: URBAN, RURAL, INDUSTRIAL, AIRPORT

6.3.1. Urban and rural

The study of short slabs JPCPs has been focussed on the structural analysis of urban and interurban JPCPs. Therefore, there are many comparisons available between traditional and short slabs JPCPs with the same concrete thickness and with equivalent slab thickness.

Roesler et al (2012) compared the ESALs resisted by short and traditional slabs JPCPs with the same concrete thickness. The Table 6.3.1 presents the results for 150 mm and 200 mm slab thicknesses. The differences in ESALs between a specific slab thickness are given by the bases applied, i.e. asphalt concrete and granular bases with different stiffness.

The Chilean catalogue of urban sections includes slab thicknesses of traditional JPCPs for different types of roadways and different modulus of subgrade reaction, k (MINVU, 2008). Salsilli & Wahr (2010) compare these slab thicknesses with the ones required by short slabs. Table 6.3.2 presents the results for the Chilean urban roadways and k = 39 MPa/m.

Table 6.3.1. Comparison of ESALs between short and traditional slabs JPCPs with the same thickness (Roesler et al, 2012).

Slab Thickness (mm)	ESALs short slabs	ESALs trad. slabs	Ratio of ESALs
150	3.7 E7	3.2 E6	8.6
150	6.9 E7	4.2 E6	6.1
150	2.3 E7	2.8 E6	12.2
150	1.7 E7	1.6 E6	9.4
200	2.0 E7	6.8 E6	34.0
200	5.1 E7	5.9 E6	11.6

Table 6.3.2. Equivalent thickness of short slabs and traditional slabs JPCPs of the Chilean catalogue for urban sections (Salsilli & Wahr, 2010).

Roadway	ESALs	Thickness traditional slabs JPCPs (mm)	Thickness short slabs JPCPs (mm)
Passageway	≤ 0.05 E6	150	110
Local	≤ 0.2 E6	160	125
Service	≤ 1 E6	170	145
Collector	≤ 3 E6	180	165
Trunk	≤ 10 E6	220	195
Express	≤ 20 E6	250	210

In addition, the Table 6.3.3 shows the slab thickness comparisons between traditional JPCPs and real-world short slabs JPCPs built in Chile and Guatemala.

Table 6.3.3. Equivalent thicknesses of traditional JPCPs with real-world short slabs (Salgado, 2011; Covarrubias, 2009, 2011, 2012)

Country	Route	Thickness short slabs JPCPs (mm)	Thickness trad. Slabs JPCPs (mm)
Chile	Cristo Redentor	170	220
Chile	Longitudinal	160	220
Chile	Punta Arenas	150	180
Chile	Cerro Castillo – Cueva del Milodón	100	180
Chile	Ruta Dalcahue	100	150
Chile	Carretera Austral	130	180
Guatemala	Puerto Quetzal - Escuintla	150	220
Guatemala	Cuesta Villalobos	220	300
Guatemala	San Cristobal – San Lucas	180	250
Guatemala	Amatillan - Palin	200	260
Guatemala	Aguacaliente - Palencia	180	240
Guatemala	Rodriguitos - Palencia	200	250
Guatemala	Santa Lucia – Los Ocotes	125	165
Guatemala	Pueblo Nuevo Viñas - Barberena	180	260
Guatemala	Génova – Caballo Blanco	150	180
Guatemala	Tecpán – Los Encuentros	180	250
Guatemala	Momostenango – Pologua	150	200
Guatemala	Eje Exclusivo – Transmetro	200	280

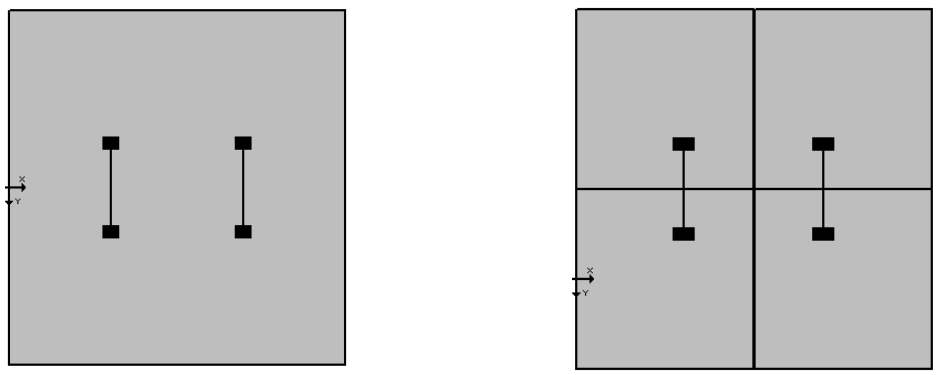
6.3.2. Industrial yards and floors

In the present thesis, industrial yards with traffic of trucks are studied. From the structural point of view, this case is the same as urban and interurban JPCPs (section 6.3.1).

In the case of industrial floors, the present thesis is focussed on the floors that are part of warehouses. Salsilli (2015) applied the principles of dimensional analysis to develop a practical M-E design method for industrial floors. Salsilli (2015) did not consider variations in the slab curvature because the concrete slabs in warehouses are isolated of significant temperature changes. Still, it is necessary to include the permanent slab curvature produced by the variations of moisture and temperature in the construction process (built-in curling).

Similar to the examples of section 6.2, in industrial floors the reduction of slab length also produces structural benefits. In effect, the slab length reduction generates lower tensile flexural stresses in the slabs due to the new traffic load configuration and slab curvature reduction. The Fig. 6.3.5 shows an example for industrial floors calculated with the 3D FE analysis program EverFE (Davids et al, 2003) considering all the design variables *ceteris paribus* with the exception of the slab size. For illustration purposes, a Forklift Toyota 7FBMF25 was used in the calculations. The loaded forklift has 61 kN at the front axle and 8 kN at the rear axle (www.toyota-forklifts.eu). Following the recommendations given by Leonards & Harr (1959) and Alhasani & Zaraei (1996) for enclosed floor slabs, a built-in curling of -12ºC was used in the example. The concrete properties of the slabs are as follows: elastic modulus 29000 MPa, Poisson´s ratio 0.2, coefficient of thermal expansion 1.1E-5 $^{o}C^{-1}$ and concrete density 2400 kg/m^3. Finally, a crack width 1 mm was assumed in the joints of the short slabs JPCP.

The Fig. 6.3.1 (left) shows the top view of a traditional slab JPCP with a loaded forklift (Toyota 7FBMF25). This configuration results in a maximum tensile flexural stress of 2.2 MPa. But if the traditional square slab of 4 m x 4m is divided in 4 smaller slabs of 2 m x 2 m the maximum tensile flexural stress is reduced from 2.2 MPa to 1.5 MPa (Fig. 6.3.1 right) as a consequence of the new traffic load configuration and slab curvature reduction.



Thickness 150 mm
Slabs 4.0 m x 4.0 m
Max. tensile flexural stress 2.2 MPa

Thickness 150 mm
Slabs 2.0 m x 2.0 m
Max. tensile flexural stress 1.5 MPa

Fig 6.3.1. Difference in the maximum tensile flexural stress between traditional slab JPCP (left) and short slab JPCP (right) applied in an industrial floor.

The reduction of the maximum flexural tensile stress in the short slabs JPCPs allows a reduction of the concrete slab thickness. Indeed, the Fig. 6.3.2 shows the change of the maximum tensile flexural stress with the variation of the slab thickness of the short slab JPCP (Fig. 6.3.1 right). In particular, the Fig. 6.3.2 shows the equivalent thickness required by the short slab JPCP (120 mm) for the same maximum tensile flexural stress as in the traditional slab JPCP (2.2 MPa).

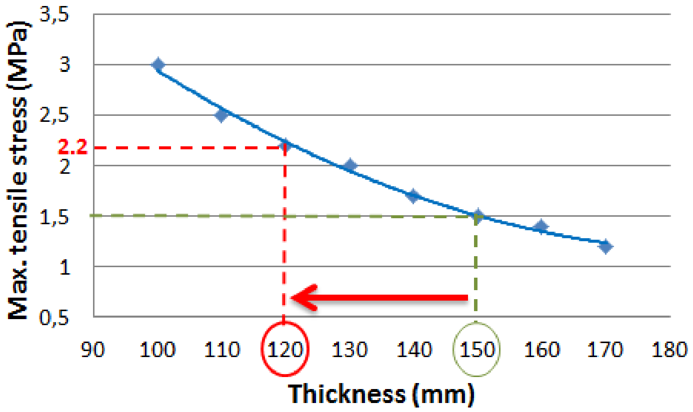


Fig 6.3.2. Relation between the maximum tensile flexural stress and slab thickness of the short slab JPCP of the analysed industrial floor.

6.3.3. Airport aprons

Originally Westergaard developed its work for JPCPs on airports. The dimensional analysis introduced in section 6.2 leads to extensions of the Westergaard equations in order to take into account variations of slab size, load transfer, and others (Ioannides, 1990; Ioannides & Salsilli, 1989; Salsilli, 1991; Cabrera, 1998; Bordelon et al, 2009, Salsilli et al, 2015; Salsilli, 2015). In effect, it is possible to apply the dimensional analysis for different JPCP applications (as airports), slab size and load configurations in order to find particular adjustment factors. For instance, Salsilli et al. (2015) extended the Westergaard equations to the case of short slabs in order to develop a practical structural design method for urban and interurban JPCPs useful to practitioners. Even when this kind of work is not available yet for airport aprons, the procedure of incorporation of the relevant results of early-age concrete behaviour in the M-E design of non-dowelled JPCPs presented in this chapter is applicable to airport aprons as well. Actually, the focus of the present study is to incorporate these relevant results (as the crack width at joints) in practical M-E structural design methods useful for practitioners, which is not possible with FE analyses due to the complexity, computational requirements and time of execution.

6.4. CRACK WIDTH AT JOINTS IN THE M-E STRUCTURAL DESIGN

6.4.1. Incorporation of the crack width in M-E design methods for JPCPs.

Besides the previously mentioned benefits produced by the slab length reduction of short slabs, this reduction also produces smaller crack widths (at joints) and then higher aggregate interlock. The relation LTE-crack width is specifically analysed in the present section.

M-E design methods of JPCPs in the Netherlands (Houben, 2006), Sweden (Soderqvist, 2006), USA (Hiller, 2007), Chile (Chilean Highway Agency, 2012; Salsilli et al, 2015;

Salsilli, 2015) between others, include the LTE as an adjustment factor of the Westergaard equation for slab edge stress.

In section 6.2 the benefits of the application of the dimensional analysis in the M-E design of traditional JPCPs (Ioannides, 1984) was introduced. Salsilli et al (2015) extended this application to the specific case of short slabs in order to develop a practical structural design method for short slabs JPCPs useful to practitioners. With this M-E method the thickness reductions provided by short slabs JPCPs with respect to the traditional slabs JPCPs were calculated as included in the Chilean catalogue of urban sections (Table 6.3.2).

As part of this practical M-E design method, Salsilli et al. (2015) extended the adjustment factor of the Eq. 6.2.2 for the specific case of short slabs JPCPs. In a similar way, Salsilli et al. (2015) extended the adjustment factor for load transfer originally developed for traditional JPCPs (Ioannides & Korovesis 1990; Salsilli, 1991; Ioannides & Hammons, 1996; Cabrera, 1998). The Eq. 6.4.1 shows this adjustment factor for load transfer.

$$F_{LTE} = 0.996573 + 0.00439187 * LTE - 0.00005851 * (LTE)^2 - 0.003124 * \left(\frac{a}{l}\right) * LTE - 0.03682 * Ln\left(LTE\right) \qquad (6.4.1)$$

To explicitly incorporate the aggregate interlock in the M-E structural design it is necessary to establish a relationship between the LTE and crack width (at joints) resulting of the early-age behaviour of the JPCP (Fig. 6.1.1). Indeed, the crack width directly determines the aggregate interlock which is the most influential load transfer mechanism in non-dowelled JPCPs (Buch et al, 2000; Hanekom et al, 2003; IPRF, 2011). In the present research, the creation of the LTE-crack width relation relies on the following 3 methods:

- Walraven´s nonlinear aggregate interlock model incorporated in the 3D FE program EverFE (validated)
- Laboratory results (previous investigations)
- Field measurements (present investigation)

6.4.2. Relation LTE-crack width in non-dowelled JPCPs

6.4.2.1. Direct relation instead of indirect ones

The development and analysis of relationships between the LTE with indirect causes as seasons of the year, geographical regions, air temperatures or even slab temperatures, can be useful for general guidelines and trends. However, as it has been indicated in the present and the previous chapters of this thesis, the focus and contribution of the present investigation is not the relationship of the LTE by aggregate interlock with indirect causes, but specifically with the direct cause of it which is the crack width (Fig. 6.1.1). And when the direct cause of the LTE by aggregate interlock is being specifically investigated (and measured in the field), returning to correlations with indirect variables is not only unnecessary but even confusing for the intended purpose and contribution of the research. In order to avoid that, in the present section the analysis of previous investigations, the field measurements and the presentation of information is explicitly focussed on the direct cause of the LTE by aggregate interlock instead of (returning again to) indirect ones.

Even more specifically the emphasis is to develop the relation LTE-crack width for non-dowelled short slabs JPCPs because, although their provision of load transfer relies on aggregate interlock, specific studies of the relation LTE-crack width were not found in the state of the art associated to this innovation.

6.4.2.1. Analysis of previous investigations

In the development of the program Hiperpav, Ruiz et al (2005) state that if in non-dowelled JPCPs the crack width is 0.6 mm or greater the aggregate cannot provide load transfer. This limit value appears too conservative when it is compared with other investigations.

The 3D FE program EverFE relies on the two-phase aggregate interlock model developed by Walraven (1981, 1994) to generate nonlinear aggregate interlock crack constitutive relations depending of the maximum paste strength, the paste-aggregate coeficient of friction, the aggregate volumen fraction and the maximum aggregate diameter (Davids, 2003). Davids & Mahoney (1999) validated this modelling using the experimental data of Colley & Humphrey (1967) which is the classic study for relationships LTE-crack width. Colley & Humphrey (1967) performed laboratory tests on concrete slabs 178 mm and 229 mm thick. Davids & Mahoney (1999) validated EverFE for these conditions and no significant differences were found between the two slab thicknesses. The Fig. 6.4.1 presents the case of slab thickness 178 mm. This slab thickness can represent the one of short slabs JPCPs and, according to what was presented in Chapter 5, the crack widths of short slabs are generally in the region of 1.1 mm or less, which has associated values of LTE ≥ 70% (Fig. 6.4.1), i.e. adequate provision of LTE even without dowels bars.

Colley & Humphrey (1967) used American aggregates in the laboratory tests. Similar is the case of Jensen (2001), where the relationship LTE-crack width was also studied, but especially focused on the influence of large sized coarse aggregates (using aggregates of USA).

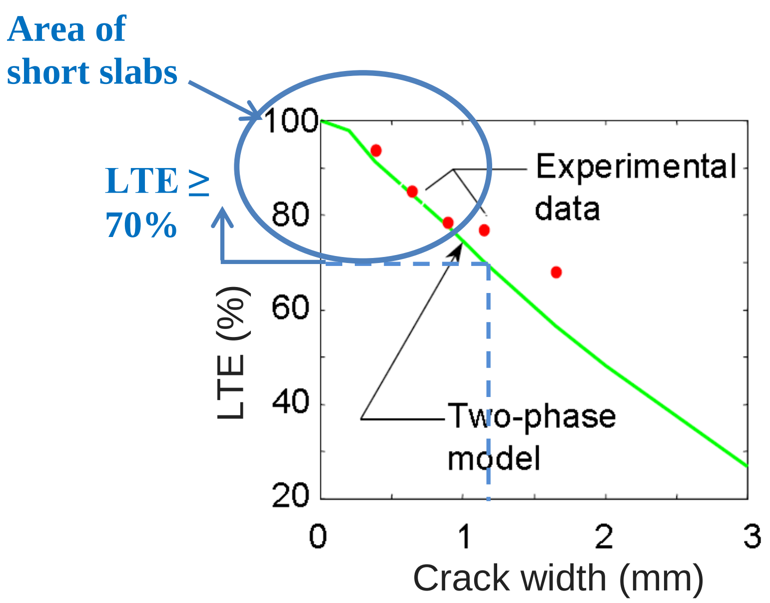


Fig. 6.4.1. Comparison of measured and computed LTE (Adapted from Davids, 2004)

Later, Hanekom et al (2003) compared the relationships LTE-crack width given by Jensen (2001) and EverFE (Davids & Mahoney, 1999) with the ones obtained in laboratory tests with typical South African aggregates. From this comparison they found significant differences in the LTE by aggregate interlock provided by the American aggregates and the South African ones. The Fig 6.4.2 presents, for the softest aggregate applied in pavements in South Africa, the results of LTE-crack width obtained in laboratory tests and modelling with the FE program EverFE (using the South African data). In effect, the South African study was made with the extremes of aggregate quality of the spectrum of concrete construction aggregates crushed in South Africa. In terms of the Elastic Modulus of the aggregates the values obtained were 42 GPa (for the hardest aggregates) and 29 GPa (for the softest aggregates). The Fig 6.4.2 shows the results for the 2 aggregate sizes studied in South Africa (19 mm and 37.5 mm) under static and dynamic conditions. Even with the softest South African aggregates the results do not drop below 90% before a crack width 2.3 mm.

The Fig. 6.4.2 also shows the differences obtained in the South African study with the EverFE modelling (using the South African data). EverFE gives lower LTE values than the ones obtained in the laboratory in South Africa, but higher LTE values than the ones in Fig. 6.4.1 with the aggregates of USA (maximum size 38 mm). In the study of Jensen and Hansen (2001) the LTE 70% is reached for crack width as small as 0.8 mm for both aggregates studied (maximum size 25 mm) with Elastic Modulus of the aggregates 22 GPa and 24 GPa respectively. The only exception was the stronger aggregate of 50 mm that is able to maintain an asymptotic behaviour around 80% LTE until 2.5 mm. Actually, the focus of the study of Jensen and Hansen (2001) was the contribution of the large size coarse aggregate in the LTE.

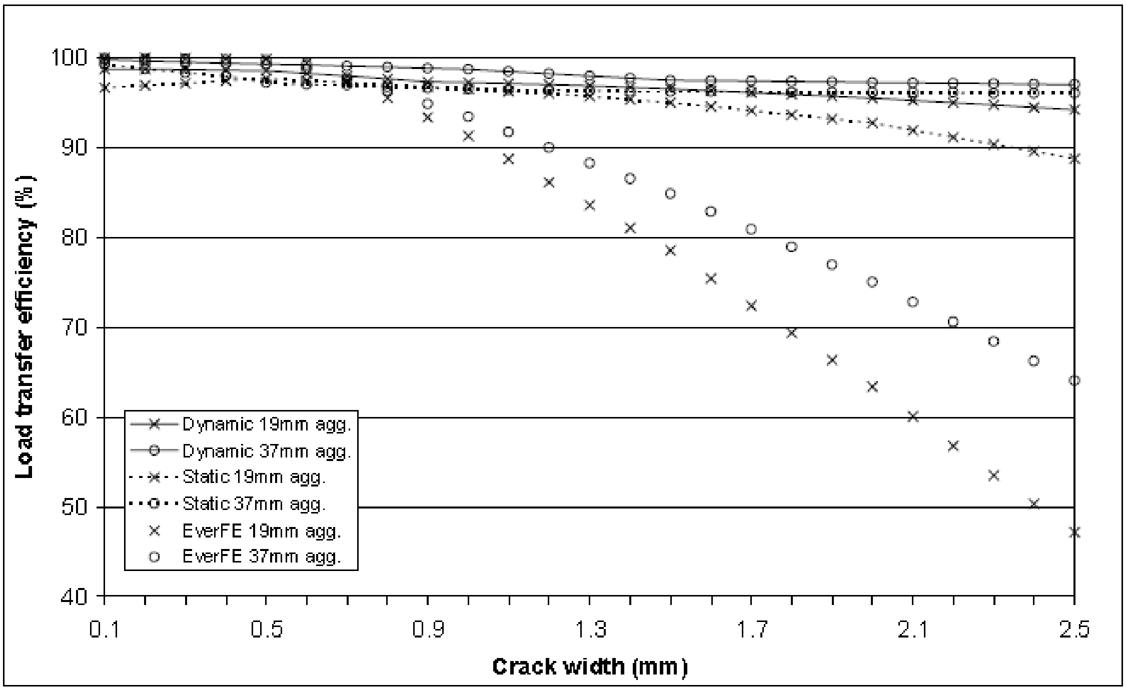


Fig. 6.4.2. LTE-crack width relations in South African study (Hanekom et al, 2001)

As it could be expected, Hanekom et al (2003) found higher values of LTE for the hardest aggregates of the spectrum of concrete construction aggregates crushed in South Africa. However, in the present thesis it is highlighted the fact that even the softest aggregate gives already the high LTE values of the Fig. 6.4.2 for static and dynamic loading (where dynamic loading yields the higher values). Finally, as it could be expected as well, the larger coarse aggregate (37.5 mm) provides higher LTE than the aggregate of 19 mm.

Jensen and Hansen (2001) and Hanekom et al (2003) agree that the base starts to play a role in the LTE when the crack width is 2.5 mm or more. Indeed, despite the differences in aggregates, the conclusion of these different investigations regarding the contribution of the base on the LTE is the same, i.e. the base begins to influence the LTE only since crack widths 2.5 mm. According to Davids & Mahoney (1999), and the crack width values of Chapter 5, the crack width in traditional slabs JPCPs is in general less than 2.5 mm, and for short slabs it is even less (50% of reduction regarding to the ones in traditional slabs JPCPs). Actually, according to what was presented in Chapter 5, the crack widths of short slabs are generally less than 1.2 mm. Hence, for non-dowelled JPCPs, and in particular for short slabs JPCPs (which is the emphasis of the present chapter), the analysis is focussed in the most influential load transfer mechanism in non-dowelled JPCPs which is the aggregate interlock (Buch et al, 2000; Jensen and Hansen, 2001; Hanekom et al, 2003; IPRF, 2011).

6.4.2.2. Field measurements LTE-crack width in short slabs JPCPs.

Salsilli et al (2015) recognize the necessity to perform specific studies of LTE in short slabs JPCPs. Although Roesler et al (2012) performed measurements of LTE in short slabs JPCPs, they did not study the relationship LTE - crack width, i.e. including the most influential load transfer mechanism in non-dowelled JPCPs, the aggregate interlock (Buch et al, 2000; Hanekom et al, 2003; IPRF, 2011). In effect, although the provision of load transfer of the non-dowelled short slabs JPCPs relies on aggregate interlock, specific studies of the relation LTE-crack width were not found in the state of the art associated to this innovation. Therefore, the field measurements of the present sub-section are concentrated on developing the relation LTE-crack width of this innovation.

Field measurements were performed in a short slabs JPCP test section located in Santiago city (latitude 33° 30' S) at the Metropolitan Region of Chile. The characteristics of the test section are as follows: slab length 1.8 m, slab thickness 140 mm, concrete flexural tensile strength 5 MPa, aggregates of 38 mm (maximum size), no dowel bars and granular base CBR 40% (thickness 120 mm) placed over the native soil (CBR 20%[6]).

Fig. 6.4.3. Measurements of LTE with FWD (left) and crack width with microscope (right)

[6] Estimated from the document "Investigation of the Bases and Subbases for Pavements" (MINVU, 2007)

It is important to emphasize that the focus of the present sub-section is to develop the relation between the LTE and the crack width in short slabs JPCPs (Fig. 6.1.1). Therefore, the emphasis in the field was to measure these 2 variables directly related instead of the indirect ones. In this way, the deflection LTE was measured with the Falling Weight Deflectometer (FWD, Fig. 6.4.3 left) and the absolute crack width obtained with fissurometer or digital microscope (Fig. 6.4.3 right). The measurements were performed according to the basis and procedure presented in Chapter 5 (section 5.3.3).

With the support of the National Highway Laboratory (NHL) of Chile, measurements of the relation LTE-crack width were performed in March and June 2012. The measurements of March were made on 8 joints at the Chilean summer, with 22ºC as the average temperature of the pavement surface. As the cracking pattern developed, the measurements of June were performed on 9 joints at the end of the Chilean autumn with an average temperature of the pavement surface of 12ºC. As minimum, the LTE was measured 2 times per joint in order to obtain the average LTE for the crack width at the time of the measurement. It is important to reiterate that the objective of the field measurements was to obtain a spectrum of different crack widths values to develop the direct relation with the LTE (Fig. 6.4.4). Most of the smaller crack widths values of the Fig. 6.4.3 (Zone I) were obtained in March (higher temperature). And the larger crack width values (Zone II) were obtained at the lower pavement temperature (June). The unusual large crack widths values (for short slabs JPCPs) can be explained by the presence of 60% of Uncracked Joints (UnCrJ) due to the shallow saw-cutting applied (more details in section 6.5.2).

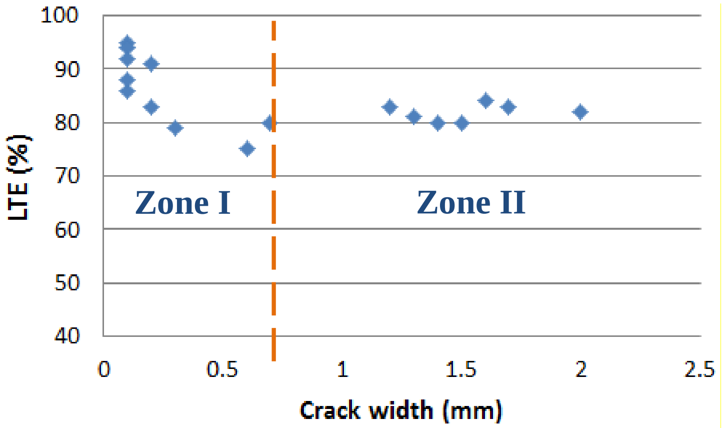


Fig. 6.4.4. Results of LTE-crack width obtained in the field measurements

The Fig. 6.4.4 can be divided in 2 zones, as follows:

- Zone I: Until crack width 0.7 mm the effects of the crack width on the LTE are clearly observed.
- Zone II: After the crack width 0.7 mm an asymptotic behaviour of the LTE around 80% can be observed.

In Zone I the LTE values basically decrease with the increase of the crack widths, from 95% for crack width 0.1 mm until 80% for crack with 0.7 mm (Fig. 6.4.4). When the crack width continues increasing the expected behaviour is the decrease of LTE. In fact, the Fig. 6.4.5 (right) shows the extrapolation of the function LTE-crack width obtained for Zone I (Fig. 6.4.5 left).

However, in Zone II the LTE presents an asymptotic behaviour around 80% (Fig. 6.4.5 right). This asymptotic behaviour is also observed in both, the South African and the American studies, especially related with the high quality and larger aggregates. Therefore, the high quality (and large) aggregates have a fundamental role in the limitation of further LTE reduction as the crack width increases (for the spectrum of crack width values analysed in the present section). In a similar way, the asymptotic behaviour shown in Fig. 6.4.5 (right) can be explained by the high quality of the Chilean aggregates (MINVU, 2007; Achurra, 2009) of 38 mm applied in the test section. Actually, Achurra (2009) studied the quality of the aggregates of different regions of Chile focussed on the polishing of aggregates. Although Achurra (2009) did not determine the elastic modulus of the aggregates, their mechanical abrasion was obtained with the Los Angeles method. For the aggregates of the Metropolitan Region (where the test section of NHL is located) the extremes of the Los Angeles abrasion values were 12%, for the hardest aggregate, and 15% for the softest aggregate (Achurra, 2009). As reference, the Los Angeles values of the South African aggregates were 21% (for the hardest aggregates) and 33% (for the softest one).

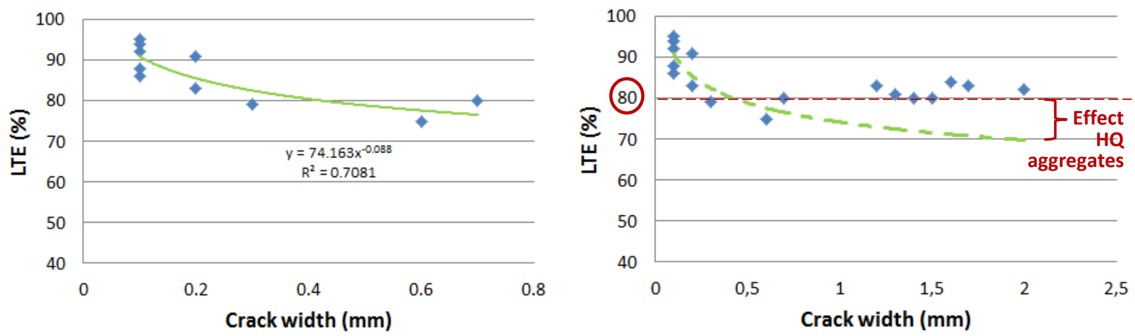


Fig. 6.4.5. Function LTE-crack width of Zone I (left) extrapolated in Zone II (right).

6.4.2.3. Complementary field measurements performed with the Faultimeter

Complementary measurements were performed with the Faultimeter of the Belgian Road Research Centre (BRRC). Although the Faultimeter does not measure the LTE, the BRRC has performed measurements with the Faultimeter and the FWD at many different JPCPs in Belgium in order to compare the LTE and the B1+B2 values (Perez et al, 2009; Perez & Van Geem, 2010). In addition, the Faultimeter is able to indicate if the joint is interlocked (or not) after the slabs have been relieved of the load (Perez et al, 2009; Perez & Van Geem, 2010). The determination of the existence (or not) of aggregate interlocking in the joints is useful complementary information for the purposes of the present research.

The Faultimeter is put over the joint between slabs (Fig. 6.4.5 left) and the Faultimeter gauge records the maximum relative movements of the slabs while the rear axle of a truck passes from the approach slab to the leave slab (Fig. 6.3.11 right). The first maximum displacement (B1) can be measured when the load is on top of the approach slab before the joint and the second one (B2) when the load is on top of the leave slab after the joint (Perez & Van Geem, 2010). The Faultimeter indicator is the sum of both maximal vertical displacements B1 and B2 measured during the passage of a heavy axle rear (11 tons) (Fig. 6.4.6 right). After the slabs have been relieved, the Residual Value (RV) of the gauge of the Faultimeter is recorded in order to determine if the joint is interlocked (or not).

Although Perez et al (2009) and Perez & Van Geem (2010) do not provide more details about the RV, its specific values or levels, they clearly distinguish the following 2 cases:

- When RV = 0 the joints are not interlocked
- When RV > 0 the joints are interlocked

Accordingly, in the field it is necessary to determine if RV = 0 or if RV > 0 reading the Faultimeter gauge after the slabs have ben relieved. Although Perez et al (2009) and Perez & Van Geem (2010) do not provide more details about the RV, the fact that the Faultimeter of the BRRC can indicate the existence (or not) of interlocking in the joint is useful complementary information for the purposes of the present research.

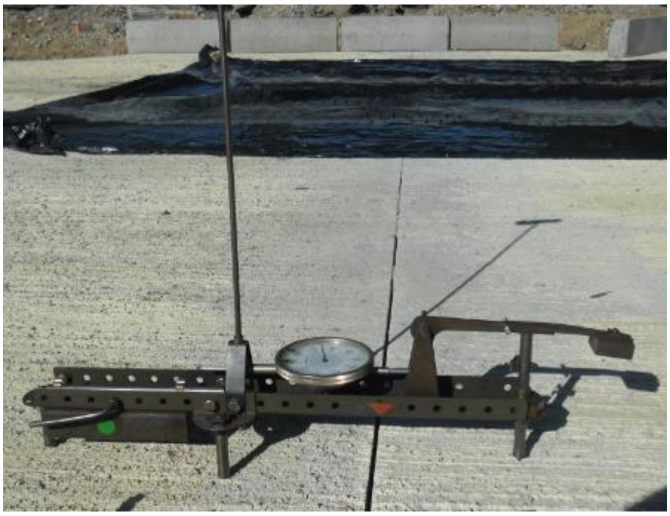


Fig 6.4.6. Faultimeter position on a joint (left) and reading of the gauge (right).

The Fig 6.4.6 actually represents measurements performed with the Faultimeter in the Octave Region of Bio Bio (latitude $36^{\circ} 58^{'}$ S), Chile. These measurements were performed according to the specifications of the BRRC (Perez et al, 2009; Perez and Van Geem, 2010) and in both traffic directions. Then, the RV and B1+B2 per joint were determined as the average of the results in every traffic direction.

The characteristics of the test section are as follows: slab length 4 m, slab thickness 260 mm, aggregates of 38 mm (maximum size), no dowel bars, granular base CBR 50% and concrete flexural tensile strength 4.8 MPa. In particular, 8 joints with crack widths between 0.1 mm and 1.2 mm were auscultated with the Faultimeter. Although the 8 joints were part of a traditional JPCP, the magnitude of the (narrow) crack widths can be explained by the high slab temperature at the time of the measurements (average 25^{o}C at the slab surface measured with infrared thermometer).

In all cases the field measurements gave results of B1+B2 $\leq$ 0.2 mm (Table 6.4.1). It is widely accepted that the slabs are stable when the value for B1+B2 is lower than 0.5 mm (Perez et al, 2009; Perez & Van Geem, 2010). This was an expected result because the slabs had not yet been opened to traffic. Actually the measurements were performed only 1 month after the JPCP was built. However, for values of B1+B2 < 0.4 mm there is not a conclusive relationship with the LTE (Perez et al, 2009; Perez & Van Geem, 2010).

Table 6.4.1. Results of crack width and B1+B2

Joint	Crack width (mm)	B1+B2 (mm)
1	0.1	0.20
2	0.2	0.20
3	0.5	0.20
4	0.6	0.20
5	0.7	0.19
6	0.8	0.20
7	0.9	0.19
8	1.2	0.20

Nevertheless, as mentioned before, the Faultimeter also measures the RV after the slabs have been relieved. In the 8 joints where the measurements were performed the result was RV > 0 which indicate the interlocking of the joints. This is an additional confirmation of the results of the present section regarding the provision of aggregate interlock of crack widths between 0.1 mm and 1.2 mm (Figures 6.4.2, 6.4.3 and 6.4.4).

Although the large slab thickness (260 mm) can play a role in the low vertical displacement of the slabs (B1+B2), the RV given by the Faultimeter clearly shows the aggregate interlock in the joints.

6.4.2.4. Recommendations to develop the relation LTE-crack width

The first recommendation is to develop the relationship LTE-crack width directly from field measurements or laboratory tests. When this is not possible the recommendations given in the present section can be useful. In effect, these recommendations are given from a pavement client´s perspective, i.e. with the objective to be practical and useful for practitioners in order to incorporate in the M-E pavement design the direct cause of the LTE by aggregate interlock which is the crack width resulting from the early-age concrete behaviour (Chapter 5).

When aggregates of high quality are applied, equivalent to the South African ones or the Chilean aggregates, the results of the present section can be used as a reference (Figures 6.4.2, 6.4.3 and 6.4.4). In this case, the application of a safety factor is also recommended to take into account potential differences.

When no high quality aggregates are applied, 3 possibilities are proposed to develop the relation LTE-crack width using the validated Walraven´s nonlinear aggregate model of EverFE:

- Specific curve: With EverFE it is possible to model the specific characteristics of the JPCP under study (including its aggregates) and to calculate its particular relation LTE-crack width.

- Set of characteristic curves: Curves for typical groups of JPCPs can be created with EverFE. For instance, as part of a pavement design manual, characteristic curves can be generated in advance for the representatives JPCP designs and aggregates of a region.

- General curve: It is also possible to use the curve of the experimental validation of EverFE (Fig. 6.4.1). This is a more direct method but it also could be a more conservative one.

It is important to note that in all cases, due to the (small) crack widths of short slabs JPCPs, these non-dowelled JPCPs are able to provide LTE ≥ 70%, i.e. adequate LTE by aggregate interlock.

6.4.2.5. Final remarks and future research

It is important to highlight that due to the reduction of the crack width in short slabs JPCPs (50% smaller than traditional JPCPs), this type of non-dowelled JPCPs provides LTE ≥ 70%, i.e. adequate provision of LTE even without dowels bars. Indeed, in this case the LTE relies on aggregate interlock and the results of the Faultimeter (RV > 0) have confirmed this interlocking in the joints for crack widths ≤ 1.2 mm which are the typical values of short slabs JPCPs (Chapter 5).

Although in short slabs JPCPs the LTE by aggregate interlock is already guarranted by the narrow cracks under the joints, the provision of an adequate base under the concrete slabs that contribute to the integral behaviour of the pavement is important (more details in section 7.4.3 of Chapter 7).

Traditional non-dowelled JPCPs still could provide adequate LTE when high quality aggregates are applied. About it, the recommendation is continuing the investigation of this effect and also the potential influence of the base under the concrete slabs.

The field measurements, analyses, results and recommendations given in the present section have been made considering natural aggregates. The case of recycled aggregates requires further specific research because Buch et al (2000) found lower LTE values provided by recycled aggregates when they are compared with natural ones. Although the nonlinear aggregate interlock model of EverFE includes the possibility of using weak aggregate (following a modification proposed by Walraven), in the present research the necessity of future specific investigation when recycled aggregates are applied is acknowledged.

In a similar way, although EverFE includes the possibility of using high quality aggregates, further investigation is recommended including specific experimental validation using high quality aggregates (as the South African or the Chilean ones for instance).

Further research is required to analyse the effects of dynamic loads over the provision in time of LTE by aggregate interlock. Nevertheless, Hanekom et al (2003) states “one of the conclusions reached from the study on South African aggregates was that there was no significant deterioration of the crack face during dynamic loading, which indicated that fatigue or abrasion of the aggregates at the joint face did not play a role”. Hanekom et al (2003) attributed this to the high quality crushed stone used in South Africa. In addition, even when Roesler et al (2002) did not study the relation LTE-crack width, they measured the LTE in short slabs JPCPs over a granular base even after 50 million ESALs. In all cases the LTE at the end of the tests was between 70% and 90% although the measurements were performed in USA, i.e. with typical American aggregates. Furthermore, Hanekom et al (2003) found higher values of LTE per crack width under dynamic conditions than under static ones (Fig. 6.4.2), i.e. the static case is more conservative. Therefore, although further research with dynamic loading is necessary, the static case provides an adequate indication of the relation LTE-crack width for the incorporation of the early-age concrete behaviour in the design of non-dowelled JPCPs, especially considering that, in general, the M-E design methods do not include the relation of the LTE with the crack width, that represents the most influential load transfer mechanism in non-dowelled JPCPs, i.e. the aggregate interlock (Buch et al, 2000; Hanekom et al, 2003; IPRF, 2011).

6.5. UNCRACKED JOINTS: STRUCTURAL EFFECTS, CAUSES AND RECOMMENDATIONS

6.5.1. Example of the structural effects of UnCrJ

The presence of UnCrJ is particularly relevant for short slabs JPCPs because the postulated benefits of this innovation are valid on the basis that the slabs are effectively shorter. One consequence of the presence of UnCrJ is the larger crack widths in the activated joints. The Fig. 6.5.1 (a) presents the case of 0% UnCrJ, i.e. the effective slab length is the same as the designed slab length (2 m in the example). On the contrary, in the Fig. 6.5.1 (b) only 50% of the joints were activated. In this case the effective slab length is twice the designed slab length (4 m in the example). According to what was presented in Chapter 5, if the crack width for 2 m is 1 mm for instance, the crack width for 4 m is the double, i.e. 2 mm.

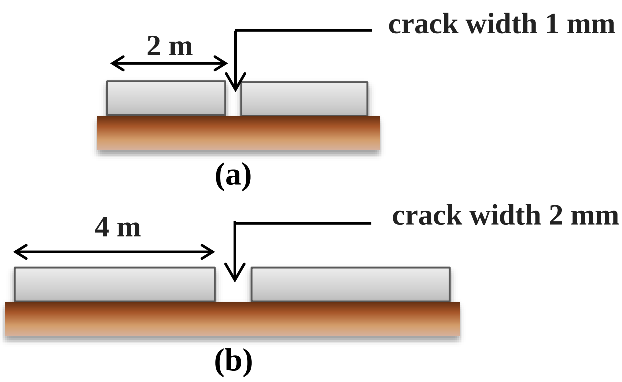


Fig 6.5.1. Effect of uncracked joints on the crack width of an activated joint

The Fig. 6.5.2 shows the variations of the LTE with the crack width, in particular the LTE associated to crack width 1.0 mm (70%) and 2.0 mm (39%). The calculations were made with the 3D FE program EverFE (Davids et al, 2003) considering the short slab JPCP of the section 6.3.2 (thickness 120 mm) but with the front axle of the Forklift Toyota 7FBMF25 positioned just before the joint. The base was treated as a dende liquid (k = 0.039 MPa/mm) and the nonlinear aggregate interlock model was generated with a maximum paste strength 30 [MPa], paste-aggregate coeficient of friction 0.3, aggregate volumen fraction 0.75 and maximum aggregate diameter 20 [mm].

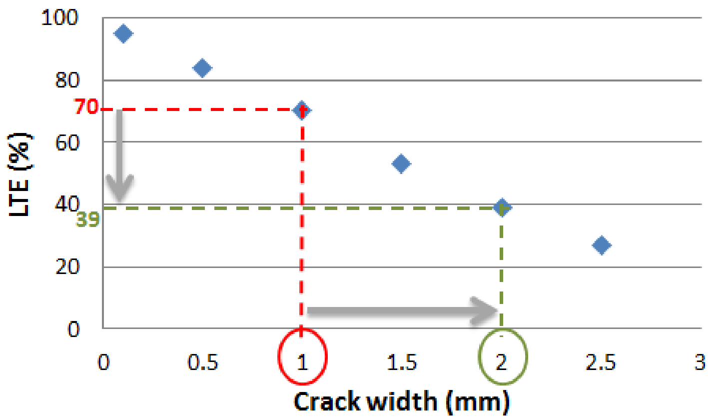


Fig 6.5.2. Variations of the LTE with the crack width

The Fig. 6.5.2 shows that when the joints are activated the LTE is 70%, i.e. adequate provision of LTE. However, with 50% of UnCrJ the LTE by aggregate interlock is reduced to 39% which is insufficient for an adequate in-service performance. Therefore, the presence of UnCrJ must be avoided in order to obtain the structural benefits of the non-dowelled short slabs JPCPs. For that, it is necessary to know the main causes and the most influential variables affecting the joint activation.

6.5.2. Most influential variables on the joint activation

As it has been presented in this thesis, the cracking pattern of JPCPs is the result of the early-age concrete behaviour. In particular, the chapters 4 and 5 include analyses of the most influential variables on the crack widths and presence of UnCrJ of traditional and short slabs JPCPs (see Fig. 6.1.1 as well). From those analyses it was found that the Relative Joint Depth (RJD), which is the ratio between the saw-cut depth and the thickness of the JPCP, is one of the most influential variables affecting the joint activation of JPCPs. Indeed, the short slabs JPCPs of the section 6.4.2.2 presented 60% of UnCrJ due to the (low) RJD of 15% to 20% (Pradena et al, 2012).

Regarding to the possible influence of the traffic in the joint activation, Lee & Stoffels (2001) and Beom & Lee (2007) analysed the development of the joint activation in traditional JPCPs geographically distributed across USA and Canada. After more than 1 year in-service (thus under the effects of traffic) they concluded that the joint activation is the result of the early-age concrete behaviour (Beom & Lee, 2007). As short slabs are indeed JPCPs as well, the joint activation is also product of the early-age concrete behaviour. However, as short slabs JPCPs can be thinner than traditional slabs JPCPs, the potential effect of the traffic on the joint activation of this innovation was measured in the test section of the NHL in Santiago city, Chile (section 6.4.2.2). For that, load repetitions of 73 kN were applied with the FWD on an uncracked joint (Fig. 6.5.3). After 400 load repetitions the joint remained uncracked.

Although it is necessary to extend these preliminary measurements, this result confirms what has been observed in traditional JPCPs and what has been presented in chapters 4 and 5, i.e. the activation of the joints of the short slabs JPCPs is mainly based on the early-age concrete behaviour. In effect, even when future research could show some influence of the traffic in the joint activation of this innovation, certainly the main cause of it is the early-age concrete behaviour in the pavement. Therefore, it is necessary to influence the variables affecting that behaviour in order to reduce the presence of UnCrJ. And, as mentioned before, the RJD is one of the most influential factors affecting the joint activation, in particular in short slabs JPCPs where the Early-Entry Saw-Cutting (EESC) is a common practice. In Chapter 5 the effects of the EESC on the joint activation was analysed.

Fig 6.5.3. Application of load repetitions with the FWD on an uncracked joint

6.6. CONCLUSIONS AND RECOMMENDATIONS

6.6.1. Conclusions

The present chapter has been focused on the direct cause of the LTE by aggregate interlock (i.e. the crack width) instead of indirect ones. In this way, it is possible to incorporate in M-E design methods for non-dowelled JPCPs the direct cause of the LTE by aggregate interlock. This is especially important for the non-dowelled short slabs JPCPs where the LTE relies on the aggregate interlock.

Due to the small crack widths, the non-dowelled short slabs JPCPs are able to provide adequate LTE (≥ 70%) even without dowels bars. Indeed, in this case the LTE relies on aggregate interlock and the results of the Faultimeter (RV > 0) have confirmed this interlocking for crack widths ≤ 1.2 mm, which are the typical crack width values of short slabs JPCPs.

Especially in short slabs JPCPs it is crucial that all the joints really crack. Therefore the relative joint depth should be between 30% and 50% depending on the conditions during construction of the pavement.

6.6.2. Recommendations

Although in short slabs JPCPs the adequate provision of LTE is already guaranteed by the small crack widths (≤ 1.2 m), the application of high quality coarse aggregates can provide even higher values of LTE.

Regarding the relation LTE-crack width, it is highly recommended to develop this relation directly from field measurements or laboratory tests of the LTE-crack width. When this is not possible, the recommendations given in the present chapter can be summarized as follows:

- When high quality aggregates are applied, the presented results of South Africa and Chile can be used as a reference with the application of a safety factor.
- If no high quality aggregates are applied, the FE program EverFE is useful to generate specific curves LTE-crack width or a set of characteristic curves. A more conservative approach is the direct application of the curve of validation of the nonlinear aggregate interlock model of EverFE.

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7. FUNCTIONAL ANALYSYS OF JPCPs

7.1. INTRODUCTION

The evaluation of short slabs concrete pavements has been focussed on the structural analysis (Salsilli & Wahr, 2010; Covarrubias, 2009, 2011, 2012; Salgado, 2011; Roesler et al, 2012; Chilean Highway Agency, 2012; Salsilli et al, 2013, 2015). However, the largest groups of pavement clients, i.e. users and owners (or agents acting on their behalf, as transportation agencies) assign priority to the functional condition of the pavements (Haas & Hudson, 1996). Therefore, an integral analysis of this innovation is necessary, including the priorities of the pavements clients, i.e. the functional performance of JPCPs with short slabs.

As it has been mentioned, this thesis includes traditional practices in JPCPs but also innovative joint configurations. As a new design configuration needs to be compared with the traditional one in order to evaluate if effectively it represents an improvement (Montgomery, 2012), the functional evaluation presented in this chapter is made from that point of view, i.e. a comparative analysis between the traditional practice and the innovative joint configuration.

As the analysis of short slabs has been focussed on the structural performance, before to study the incorporation of the early-age concrete behaviour in the functional performance, it is necessary to perform the functional analysis of the unsealed joints, to study the applicability of functional deterioration models developed originally for traditional JPCPs and to incorporate complementary methods useful for the evaluation of innovations (as are the short slabs). Indeed, even if the deterioration models of traditional JPCPs are applicable to short slabs, they have not been calibrated yet.

It has been mentioned that pavement clients assign priority to the functional pavement condition. Even more specifically the priority for them is the ride quality (Haas & Hudson, 1996). Then the functional evaluation presented in this chapter is always pointing in that direction. For instance, in the analysis of the unsealed joints, more than the joint performance itself, the evaluation takes into account the effect of the joint performance on the ride quality, so at the end upon the satisfaction of the clients. Similar is the case of the joint faulting which is the major contributor of the ride quality deterioration (Jung et al, 2011; Bustos et al, 2000).

The functional analysis presented in this chapter contains the following evaluations:

- Unsealed joints behaviour
- Deterioration modelling of joint faulting
- Ride quality evaluation using a MCDM method
- Variation of the ride quality of JPCPs
- Effects of the uncracked joints in the functional analysis

7.2. JPCPs APPLICATIONS: URBAN, INTERURBAN, INDUSTRIAL, AIRPORT

In general, calibrated deteriorations models are used to evaluate the functional performance of pavements. However, there is not enough data available for the development and calibration of deterioration models for the innovative short slabs. If this is valid for urban and interurban short slabs, it is even more valid for industrial and airport applications where functional deterioration models are unusual even for traditional JPCPs. Nevertheless, there is empirical evidence for a comparative functional evaluation in an explicit, traceable and well-sustained way using a Multi-Criteria Decision-Making (MCDM) method. This improves the transparency of the decisions, instead of overlooking the functional pavement performance in the evaluation of short slabs.

As can be expected, the empirical evidence available of short slabs is mainly concentrated in urban and interurban applications. And the present chapter reflects this reality. Actually, a further analysis is made in order to evaluate the variations of the ride quality due to the slab curvature of traditional and short slabs JPCPs. The ride quality is quantified by the International Roughness Index (IRI) that is an indicator used for interurban and urban pavements but not in industrial floors or pavements for airports.

7.3. UNSEALED JOINTS BEHAVIOUR IN RELATION TO THE IN-SERVICE PERFORMANCE OF JPCPs

7.3.1. Why unsealed joints?

The traditional approach of sealing transverse contraction joints is estimated to account for between 2 and 7 percent of the initial construction cost of a JPCP (Hall, 2009). In fact, the sealing of joints has associated costs due to material, labour, construction, repair, traffic and lane closure. When the costs of keeping the joints sealed for 10 years is added, the JPCP with sealed joints ends up costing up to 45% more than the one with unsealed joints (Shober, 1986). And because the joints were constructed wide enough to receive the seal, when they are not well maintained water and incompressible materials can enter the joint producing extreme stress concentration against the edges, then spalling, and pumping that influence the production of Joint Faulting (JF). The repair costs can be significant, and especially in countries with important budget limitations where maintenance cannot be assigned priority, the joints can remain without reparation increasing the negative impact over the JPCP performance and the road users.

In addition, the joint seals are not working well enough, not keeping the joints free of water and long-term JF data shows a strong correlation with annual rainfall. In fact, the average service life of the joint seals is less than 10 years (Jung et al, 2011). Already in 1995, 10 States of USA did not rely on the sealants and 1 State (Wisconsin) reported that it had dispensed with joint sealing entirely (Jung et al, 2011). This "no-seal" policy has saved Wisconsin 6,000,000 US dollars annually with no loss in pavement performance (Shober, 1997).

Therefore, there is increased interest in eliminating transverse joint sealants as a means of lowering concrete pavement costs.

7.3.2. Technical basis of unsealed joints

The joint seals are not working well enough. They commonly have adhesive and/or cohesive failures. But also they can suffer from hydraulic pressure due to the combine action of the water and the traffic (Jung et al, 2011). The failures of the seals can give space for the contamination of the joint (Fig 7.3.1).

The function of the seals is to avoid the contamination of the joints but when the seals exhibit failures and are not well maintained, they allow the ingress of water and incompressible materials. The infiltration of water can drag fines from the base (pumping), so less support, and then joint faulting. The problem with coarse incompressible materials in the joint is the potential to produce excessive pressure against the edges of the joint, so spalling (Figure 7.3.1 right) or even splitting cracks.

Fig. 7.3.1. Joint contamination at the apron of Eindhoven airport, the Netherlands (left) – contamination and joint spalling of an urban street, Cabrero city, Chile (right)

The most common failures of joint seals are the adhesive and cohesive ones (Jung et al, 2011). In effect, in visual inspections made in bus stations of the cities of Eindhoven, Zwolle, Amersfoort, Rotterdam and Apeldoorn in the Netherlands these failures were the typical ones found in the joint seals. In addition, in bus stops of the cities of Arnhem and Maastricht in the Netherlands, sealed joints were contaminated with incompressible material even when the seals did not present the typical adhesive and/or cohesive failures (Fig. 7.3.2).


Fig. 7.3.2. Sealed joints contaminated with incompressible material at bus stop, Arnhem city, the Netherlands.

The situation presented in Figure 7.3.2 was observed in sealed joints of a bus stop in the city of Wroclaw, Poland as well (Fig. 7.3.3). In both cases the location of the seal was deep enough to allow the contamination of the joint. This location of the seal could be the result of the installation process or the hydraulic pressure of the water trapped on the joint when heavy traffic passes.

Fig. 7.3.3. Sealed joints contaminated with incompressible material at bus stop, Wroclaw city, Poland.

In the case of common adhesive and/or cohesive failures or in the situation observed in Figs. 7.3.2 and 7.3.3, the seals are not accomplishing their function. As the joints were cut wide enough to receive the seal, in-service JPCPs easily can have 10 [mm] width available for the contamination of the joints. The sealed joints shown as an example in the Figures 7.3.2 (right) and 7.3.3 (right) have available 12 [mm] and 10 [mm] respectively to the penetration of incompressible materials. In the UnJs, a saw-cut as narrow as possible (current technology allows cuts ≤ 3 mm) impede the penetration of coarse material in the joint saving the cost of sealing and re-sealing the joints. The UnJs become uniformly filled full with fine incompressible material. When the pavement expands the stress is uniformly distributed across the entire pavement cross section. This uniform stress can only amount 7 – 14 [MPa] maximum, well below the compressive strength of the concrete (Shober, 1997).

In addition, in case of UnJs the fines content of the underlying base/soil layer of the concrete slabs should be limited (≤ 8% passing 75 μm). Hence, the water cannot drag fines, then no pumping, and no joint faulting for this concept.

7.3.3. Evaluation basis

7.3.3.1. Effects of joint performance on the in-service JPCP performance

The satisfaction of the clients is a goal toward which any service or product provider, including pavement engineers, should strive (Haas & Hudson, 1996). Therefore, more than the joint performance itself, the evaluation needs to consider the effect of the joint performance on the pavement performance, so at the end upon the satisfaction of the pavement clients. The largest groups of them, i.e. road users and transportation agencies, assign priority to the ride quality. And authors as Darter & Barenberg (1976), Al-Omari & Darter (1992), ERES (1995), Yu et al (1998) and Bustos et al (2000) recognize the spalled joints as part of the direct causes of the deterioration of the JPCP´s ride quality. Indeed, the Eq. 7.3.1 represents, between others, the effect of the spalled joints in the International Roughness Index (IRI) (ERES, 1995).

$$IRI = IRI_0 + 0.00265*(TFAULT) + 0.0291*(SPALL) + 0.15*10^{-6}*(TCRACK)^3 \quad (7.3.1)$$

Where IRI = International Roughness Index (m/km); IRI_0 = initial roughness at construction (m/km); $TFAULT$ = transverse joint faulting (mm/km); $SPALL$ = spalled joints (%); $TCRACK$ = transverse cracks (Nº/km)

As sealing the joints produces more costs than leaving the joints without seals, from a pavement performance point of view, the joint seals need to enhance the JPCP performance in a way that justify the higher costs. In effect, the joint seals need be cost-effective, including all the effects upon the users, as delays and safety costs.

7.3.3.2. Variables considered in the evaluation

The evaluation of the use of UnJs considers reported experiences of different countries and new field measurements performed in Chile as part of the present research. Both are made in various JPCPs applications, different times in-service and climatic conditions, especially the amount of rainfall.

JPCPs applications
Roadways serve two primary travel needs: access to/egress from specific locations and travel mobility (FHWA, 2003). In general, there is a relationship between posted speed limits and functional classification (FHWA, 2003) and the traffic demands are different in streets, bus corridors, highways, low volume roads, industrial yards and floors. Hence, it is necessary to take into account the particular characteristics of a JPCP and its functional objective, in order to evaluate the UnJs feasibility in that JPCP application. Certainly the analysis will be different for a residential street with different functional objectives than a highway with higher traffic demand and speed. In the case of industrial yards and floors, the criteria to establish the effect of the joint performance in the JPCP performance is whether the joint performance affects the industrial operations. The evaluation scope of the present thesis is JPCPs on common industrial yards and floors (warehouses) where accessibility (for storage for instance) is the basic function.

Time in-service
This variable needs to be considered in order to define the scope of the evaluation. In some cases the accumulated number of Equivalent Single Axle Loads (ESAL) can be used as an indicator of the in-service period of the pavement. The ideal situation is to compare duplicate sections of sealed and unsealed JPCPs in their life-cycle. Nevertheless, conclusions can be obtained directly from experiences with UnJs, and indirectly using the assimilation of traffic demands to other cases. For instance, the traffic of 5 years of a bus corridor can be assimilated to the traffic that will receive a residential street in its entire life-cycle.

Climatic conditions
The main climatic condition to consider is the rainfall, due to the importance in the joint behavior. The Table 7.3.1 shows a general classification of rainfall levels related with the potential to produce joint deterioration. In addition, zones whit freeze-thaw cycles can be considered.

Table 7.3.1. Limits of rainfall levels

Rainfall Level	Lower limit (mm/year)	Upper limit (mm/year)
Low	0	250
Moderate	250	500
Medium	500	750
High	750	1000
Ultra-High	1000	-

7.3.3.3. Identification and measure of deteriorations in the field

The identification and measure of the deteriorations in the field is consistent with the section 7.3.3.1, i.e. the effects of joint performance on the in-service JPCP performance. In this way, for the joints affected by spalling the method of the FHWA of USA is applied. According to this method a joint is affected by spalling if the total length of spalling is 10 % or more of the joint length (Miller & Bellinger, 2003). Then, in the field the first step is to measure the length of spalling. If this length is more than 10% of the joint length, then the quantification of the spalling is made according to the levels of the FHWA distress manual, i.e. low level (< 75 mm), medium (75 mm ≤ spalling ≤ 150 mm) and high level (> 150 mm) (Miller & Bellinger, 2003).

In the cases of joint faulting and pumping the Chilean catalogue of pavement deterioration is applied (Chilean Highway Agency, 2013). The identification of JF is made performing a visual inspection. In the case of JF is noticed the measurement of the height difference at the joint is made at 300 [mm] from the external edge of the pavement (Chilean Highway Agency, 2013). The identification of pumping is made performing a visual inspection. In case of pumping is detected, the measurements consist of recording the number of occurrences of pumping and the length in meters of affected pavement (Chilean Highway Agency, 2013).

7.3.4. Experiences of unsealed joints in JPCPs

7.3.4.1. USA

Distress and deflection data were collected on 117 test sections located in 11 States in different climatic conditions (all rainfall levels of Table 7.3.1 and 8 states with freeze-thaw action), that were in-service about 10 years, in projects representing a range of concrete pavement designs, sealant material types, and transverse joint configurations. The results are as follows (Hall, 2009):

• the narrow width of UnJs limited the penetration of coarse incompressibles to a degree comparable to any of the 3 types of sealed joints (silicone, hot pour, and preformed);

• medium and high severity spalling was higher at a statistically significant level in sealed-joint sections (preformed sealants) than in unsealed-joint sections;
• transverse edge slab support was, in general, no more or less common in the unsealed-joint sections than in the sealed sections. The joint sealing treatment has a fairly minor influence, if any, on the quality of slab support. And, using a 95 percent confidence interval, no significant difference in average JF was detected between sealed joints of any kind and UnJs.

For 50 years the Wisconsin Department of Transportation (WisDOT) has investigated joint filling/sealing in urban and rural areas, for various traffic levels and truck loadings, on open and dense graded bases, on sandy to silty-clay soils, with short and long joint spacings, with and without dowels bars, etc. The results have always shown that filling/sealing does not enhance pavement performance (Shober, 1997). Wisconsin's unsealed joints are sawn 3 - 6 [mm] wide, they become uniformly filled full with fine incompressible material. Hence the uniform stress over the joint edges is 7 – 14 [MPa] maximum, well below the compressive strength of the concrete (Shober, 1997). They have concluded that the pavements with unsealed joints performed better than the pavements with sealed joints and the pavements with shorter joint spacings performed better than the pavements with longer joint spacings (Shober, 1986). WisDOT's findings were verified by two independent teams from Federal Highway Administration (FHWA) and Minnesota (Shober, 1997). The average annual rainfall in Wisconsin is 876 [mm] (so high rainfall area) and freeze-thaw cycles do occur.

In 1995 only 1 State (Wisconsin) reported that it had dispensed with joint sealing entirely. In 2000, 3 states (Alaska, Hawaii and Wisconsin) reported that they do not apply joint sealing (Jung et al, 2011). Hawaii is the wettest state of USA with an average annual rainfall of 1785 [mm], and Alaska has oceanic climate in the occidental coast and continental and arctic climates in the rest of the state, hence rainfall and freeze-thaw effects in the joints.

7.3.4.2. Austria, Belgium and Spain

These countries have achieved a suitable service life for up to 30 years with unsealed and undoweled JPCPs for country roads with light truck traffic (Burke & Bugler, 2002). Although Burke & Bugler (2002) declare that they were unable to determine other pavement characteristics, they affirm 'Nevertheless, 30 years of effective service is a remarkable accomplishment for unsealed pavements. Because of such reported service, one is forced to conclude that these lightly loaded secondary roads have been successful, cost-effective designs'. Austria and Belgium are countries with high rainfall all over the country, and ultra-high rainfall in a big part of the territory. Both countries include extensive regions with temperatures below zero as well (freeze-thaw).

7.3.4.3. Guatemala

Guatemala is a mountainous country with an ultra-high level of rainfall. Auscultations were made on interurban and urban unsealed JPCPs placed over a granular base and old pavements. The average initial IRI of these unsealed JPCPs was 2.0 [m/km] and the maximum change of IRI was only 0.25 [m/km] after 22,000,0000 ESALs (Covarrubias, 2011). Although there are not specific measurements of joint spalling available on these projects, the excellent results of IRI obtained for the in-service unsealed JPCPs leads to the conclusion that the UnJs have no negative effects on the pavement performance (Covarrubias, 2011).

7.3.4.4. Chile

In addition to the previous reported experiences, new field measurements over unsealed JPCPs were performed in Chile where there is an increasing application of JPCPs with innovative joint configurations as UnJs. The common practice in Chile consists of non-dowelled JPCPs over a granular base. These conditions are critical for the development of JF and IRI. Even when Chile has had big improvements in road maintenance, it is a country with budget limitations where the maintenance cannot be necessarily assigned priority. In urban areas, JPCPs are a traditional alternative with good behaviour despite the lack of maintenance. There are in-service JPCPs with joints over 10 [mm] wide, partially sealed or without seals at all. The Fig. 7.3.4 shows a typical case of an urban street (the red arrows point the presence of coarse incompressible material).

Fig. 7.3.4. JPCP with coarse incompressible material and spalling (joint width 20 mm).

Streets

The Fig. 7.3.5 shows a JPCP with UnJs of a street in an ultra-high rainfall region (latitude 36° $58'$ S). After 2.5 years in-service, with the traffic shown in Fig. 7.3.5 (left), the pavement is in excellent condition and the general behaviour of the joints is the one shown in the Fig 7.3.5 (right). In the 75 [m] long test section there are no joints affected by spalling.

Fig. 7.3.5. Traffic and UnJs state in JPCP in a street in an ultra-high rainfall zone (saw-cut 3 mm) (Pradena & Houben, 2015a).

Wisconsin's UnJs are 3 - 6 [mm] wide, they become uniformly filled with fine incompressible material. Besides the Fig. 7.3.5 (right), the Fig. 7.3.6 shows the joint behaviour of a similar case in Chilean sections of 200 [m] in ultra-high rainfall area at a residence zone (latitude 36° $49'$ S). No joints affected by spalling could be observed after 6.5 years in-service.

Fig. 7.3.6. UnJs filled with fine incompressible material (saw-cut 4 mm)
(Pradena & Houben, 2015a)

Other streets in ultra-high rainfall areas, but with lower temperatures (away from the Pacific Ocean and closer to the Andes Mountains) and potential freeze-thaw, show a similar good behaviour. This is the case of a 40 [m] long section with UnJs (latitude $37^{\circ}\,27'$ S) where no joints affected by spalling were detected after 3 years in-service. In addition, two adjacent residential streets of 70 [m] were built under practically the same conditions, but one of the streets with sealed joints and the other one with UnJs (latitude $37^{\circ}\,1'$ S). After 2.5 years in-service, both streets exhibit no joints affected by spalling. In the street with sealed joints adhesive failures (Fig. 7.3.7 left) and joint contamination (Fig. 7.3.7 right) can be observed, similar to the bus stops in the Netherlands (Fig. 7.3.2) and Poland (7.3.3).

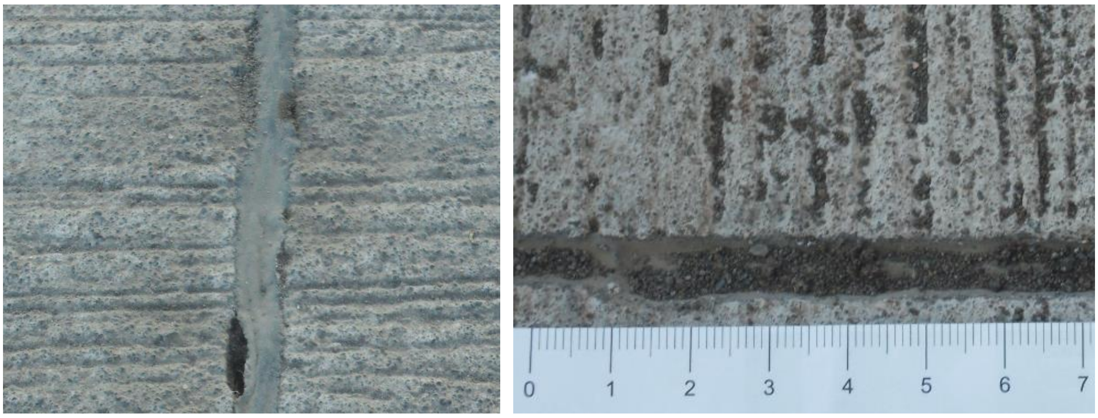

Fig. 7.3.7. Adhesive seal failure (left) and sealed joints contaminated (right)

Bus Corridors

In Santiago, the capital city of Chile (latitude $33^{\circ}\,30'$ S), the new public transportation system started in 2007 and includes articulated buses with a steering axle, a single axle – double tire at the centre and a single axle – double tire at the rear (Figure 7.3.8). These articulated buses are the typical traffic over the bus corridors built to the new public transportation system. These Bus Corridors (BCs) can include UnJs between their design features (Metropolitan SERVIU, 2005) as the ones presented in this section. The rainfall in Santiago city is 340 mm/year, i.e. an area with moderate rainfall.

Fig. 7.3.8. Typical traffic of articulated buses of the public transportation system of Santiago city (Pradena & Diaz, 2016).

As part of the present thesis field measurements were made on 5 BCs with UnJs at Santiago city. In every bus corridor, three test sections of 10 joints each were randomly chosen for the analysis. Hence, in total 150 joints were investigated in the field as follows:

- 30 joints at the bus corridor "Avenue Vicuña Mackena"
- 30 joints at the bus corridor "Avenue Pedro Aguirre Cerda"
- 30 joints at the bus corridor "Avenue Las Industrias"
- 30 joints at the bus corridor "Intermediate ring"
- 30 joints at the bus corridor "Avenue Dorsal"

One of the factors considered in the election of these bus corridors was the in-service time of the JPCPs. In effect, the first four bus corridors were built in 2007, so at the very beginning of the new public transportation system of Santiago city. The fifth bus corridor was built in 2009. The five BCs were built with the same type of concrete (mean flexural tensile strength 5.0 MPa after 28 days) and the joints were performed using the early-entry saw-cutting method with a thin blade of 2 mm wide.

The accumulated traffic in terms of the Equivalent 80 kN Single Axle Loads (ESALs) was estimated based on the frequency of the buses. The annual increment of traffic (1.8 %) and the load equivalency factor (4.22) are the typical values used in the pavement design of BCs for the public transportation system of Santiago city (Michell, 2007; Testing, 2011).

In order to make a comparison with the performance of the UnJs in the BCs, field measurements were made on UnJs on adjacent lanes of the 4 first BCs, i.e. 120 unsealed joints of the oldest BCs. These adjacent and parallel lanes are part of the same avenue as the BCs but they are dedicated to private traffic. The "Avenue Dorsal" is an exception, because it includes private and public traffic together. In this case, the accumulated ESALs were obtained from the pavement design of this avenue (Michell, 2007). The adjacent lanes have the same construction characteristics, materials and granular base as the BCs.

The Table 7.3.2 shows that 93% of the 150 joints of the 5 bus corridors were not affected by spalling even after 5 million ESALs. Only 7% of the 150 unsealed joints, i.e. 10 joints, were affected by spalling. However, these 10 joints of the bus corridors "Intermediate Ring" and "Avenue Dorsal" have very low levels of spalling with an average of 30 mm (75 mm being the lower severity level for the FHWA) which can be explained by the construction conditions. In fact, the 10 joints are localized in only three test sections, two on the "Intermediate Ring" and one on the "Avenue Dorsal". These three sections were constructed in summer under high air temperature (≈30 °C) and with late application of the curing compound, which are favourable conditions for the production of spalling even without traffic, i.e. as a result of the construction process and not necessarily due to the performance of the UnJs. In fact, Liu et al (2012) highlight the importance of the curing of concrete at early-age in the formation of the microstructure of the concrete.

Table 7.3.2. Joint spalling, in-service time and accumulated traffic of BCs with UnJs.

Bus Corridor	Years in-service	Accumulative ESALs	Spalling (joints)	Average spalling (mm)
Vicuña Mackena	9	5,000,000	0	-
Pedro A. Cerda	9	5,100,000	0	-
Las Industrias	9	1,300,000	0	-
Intermediate Ring	9	3,400,000	5	30
Dorsal	7	3,100,000	5	30

In the case of the adjacent lanes dedicated to private traffic, 100% of the joints were unaffected by spalling, i.e. all 120 joints chosen for the study. The adjacent lanes have estimated accumulated ESALs similar to that on BCs.

Although the 5 BCs (and their adjacent lanes dedicated to private traffic) have a granular base and they are non-dowelled JPCPs, no pumping and no measurable joint faulting was identified in the 15 test sections of the 5 BCs analysed (nor in the 12 test sections of the 4 adjacent lanes). Considering this and that only 7% of the UnJs is affected by a very low level of spalling of 30 mm (due to the construction conditions) it is possible to conclude that the performance of the UnJs of the 5 BCs after 8 years in-service (average), and 5 million ESALs, is adequate in terms of the potential effects upon the pavement clients. Fig. 7.3.9 shows the BCs “Avenue Vicuña Mackena” after 9 years in-service (and 5 million ESALs).

Fig. 7.3.9. Bus corridor “Avenue Vicuña Mackena” after 9 years in-service

Bus Corridors (additional comparison with sealed joints)
The BCs built for the new public transportation system of Santiago city are basically unsealed JPCPs. However, the BCs ‘Avenue Departamental’ and ‘Avenue Pajaritos’ are sealed JPCPs. The first one was built in 2013; it has 2 sections with different traffic demands, and even when it has been in-service for only 3 years, the accumulated ESALs at the time of the measurements are similar to those on the 5 BCs with UnJs. Additionally, an adjacent lane dedicated to private transportation but with similar traffic demands as a BC is included. In total, the measurements in ‘Avenue Departamental’ are thus made in 3 test

sections. ‘Avenue Pajaritos’ is an older JPCP built in 2005 and used officially as BC of the new public transportation system since 2007. A section of ‘Avenue Pajaritos’ with similar traffic demand to the previous measurements in UnJs was taken into account for comparison purposes. Every section has 3 sub-sections, and in every sub-section 10 joints are randomly chosen for the evaluation. Hence, in total 120 joints are investigated in the field as follows:

- 30 joints at the section ‘Departamental 1’ (BC)
- 30 joints at the section ‘Departamental 2’ (BC)
- 30 joints at the section ‘Departamental 3’ (adjacent lane for private traffic)
- 30 joints at the section of ‘Pajaritos’ (BC)

Additionally, a section of ‘Avenue Pajaritos’ with higher traffic demand is included in the present study, not for comparison purposes, but for illustration ones.

‘Avenue Departamental’ and ‘Avenue Pajaritos’ have the same type of concrete, i.e. mean flexural tensile strength 5.0 MPa after 28 days.

The traffic of the sections of ‘Avenue Departamental’ is obtained directly from its official pavement design (Testing, 2011). The accumulated ESALs of ‘Avenue Pajaritos’ after the initiation of the new public transportation system of Santiago city in 2007 are estimated based on the frequency of the buses. The annual increment of traffic (1.8%) and the load equivalency factors (4.22 for articulated buses; 1.37 for normal big buses and 0.68 for small buses) are the typical ones used in the pavement designs of the BCs of this public transportation system (Michell, 2007; Testing, 2011, 2016). Considering that in the period 2005-2007, ‘Avenue Pajaritos’ also carried traffic of buses, the ESALs in this period are estimated based on an assumed frequency, increment of traffic (1.8%) and load equivalency factor (1.37) similar to the actual ones (Michell, 2007; Testing, 2011, 2016).

The Table 7.3.3 shows the results of the measurements at sealed joints. Low levels of spalling (average 40 mm) were detected in 10 sealed joints, i.e. 8 % of the 120 sealed joints. However, typical adhesive failure was identified in the seals and even generalized joint faulting in one section of “Avenue Pajaritos” (average 5 mm).

Table 7.3.3. Joint spalling, in-service time and accumulated traffic of avenues with sealed joints.

Avenue	Years in-service	Accumulative ESALs	Spalling (joints)	Average spalling (mm)
Departamental 1	3	3,000,000	2	40
Departamental 2	3	1,500,000	0	-
Departamental 3	3	4,000,000	4	30
Pajaritos	11	3,500,000	4	40

Despite the low level of spalling, the sealed joints will require maintenance interventions in order to avoid further joint deteriorations. In effect, these joints were constructed wide enough to receive the seal, and with adhesive failure, there is space available for the contamination of the joint that can produce further joint spalling and joint faulting with the increment of traffic. Actually, for illustration purposes further measurements were made in another section of “Avenue Pajaritos” with higher traffic demand (9,000,000 accumulated

ESALs). Indeed, the joint deterioration increases (Fig. 7.3.10) including higher generalized joint faulting (average 7 mm).

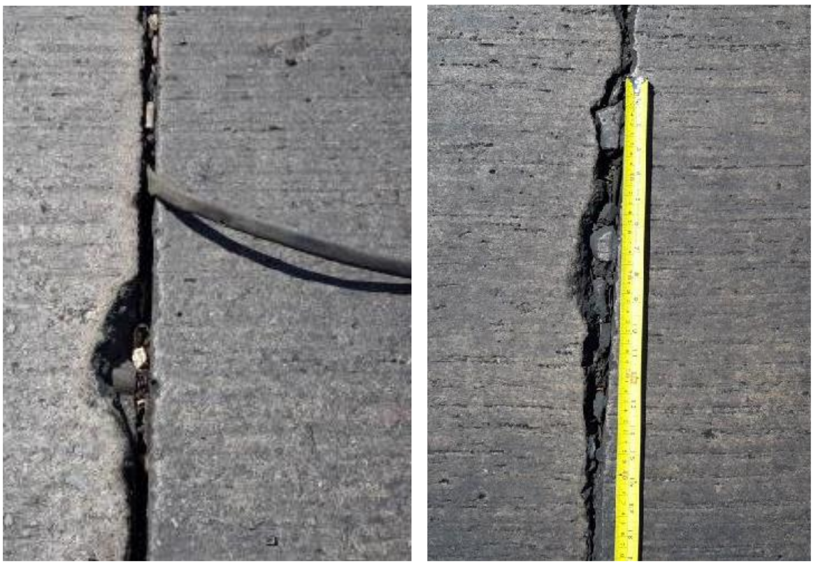

Fig. 7.3.10. Deteriorations in sealed joints of 'Avenue Pajaritos' (Pradena & Diaz, 2016).

The maintenance interventions required for the sealed joints (in order to avoid further deteriorations) are not required for UnJs because the thin saw-cut (2-3 mm) impede the penetration of coarse incompressible materials in the joint. And when the fines content in the underlying base/soil layer is limited (≤ 8% passing 75 µm), the water cannot drag fines, there is no pumping, and no production of joint faulting for this concept.

According to the comparative field measurements performed in bus corridors with sealed and unsealed joints, the sealed joints do not justify their extra costs enhancing the pavement performance. This is in agreement with the more than 50 years of experience of Wisconsin (USA) where the results, for various truck loadings, have always shown that sealing does not enhance pavement performance. They have even concluded that the pavements with unsealed joints perform better than pavements with sealed joints (Shober, 1987).

Industrial yards and floors

In the present thesis, industrial yards in which the traffic primarily consists of trucks are evaluated. For the study of the applicability of UnJs, this case is similar to a BC and, in terms of ESALs, even similar to other JPCP applications where the UnJs have shown excellent results (as it was presented previously).

In the case of industrial floors, as part of the present thesis new field measurements were made at 20 UnJs of a warehouse of cement bags, i.e. with primarily traffic of forklifts. This industrial floor was constructed in 2011 and the joints saw-cut with a thin blade of 2 mm. The warehouse is located in Santiago city, Chile (Latitude 33° 30' S).

The estimation of the load repetitions is based in the information of the frequency of forklifts provided by the cement company, i.e. 1 forklift every 10 minutes. It is assumed a forklift loaded with 25 cement bags and annual traffic increment of 1%. In this way 500,000 repetitions of a loaded forklift (50 kN) were obtained for 20 years.

The Fig. 7.3.11 shows a general view of the state of the warehouse floor and an example of one of the UnJs. In effect, no joints affected by spalling, pumping or joint faulting after 5 years in-service (125,000 load repetitions) were detected in the field measurements.

Fig. 7.3.11. General view of the industrial floor and an example of joint state (Pradena & Diaz, 2016)

Although warehouses are commonly covered by a roof (thus no rainfall), when the joints have seals they can be damaged or removed by the forklifts traffic. This was the case in a warehouse of timber products after less than 1.5 years in-service in Concepcion, Chile (latitude 36° $49'$ S). At the same location, and with similar age and purposes, the UnJs do not exhibit damage. Furthermore, the industrial complex includes a traditional JPCP, hence originally with sealed joints, over granular base with more than 15 years in-service and without seals anymore as a result of the forklifts traffic. This industrial floor serves to a warehouse of steel products as it is observed in Fig. 7.3.12.

Fig. 7.3.12. Industrial floor in warehouse of steel products (Pradena and Houben, 2015b).

Although some joint spalling was observed, it basically does not produce negative effects on the floor performance, that affect the functionality of the warehouses operations (Fig. 7.3.12 right), even with wider joints (≥ 10 mm) than the UnJs (≤ 3 mm). Therefore, *ceteris paribus*, the application of UnJs can be considered feasible.

7.3.5. Cost-effective alternative for JPCPs

From a pavement performance point of view, the joint seals need to be cost-effective enhancing the JPCP performance, including all the effects upon the users, as delays and safety costs. This is a big challenge considering the extra costs of sealing (and re-sealing) the joints, the problems with the joint seals effectiveness and the comparison with the UnJs performance. For instance, the comparison made by means of the new field measurements performed for the present research on sealed and unsealed joints of BCs under the same climatic conditions and until a similar level of accumulated traffic (even more over UnJs). The results obtained are in agreement with the more than 50 years of experience of Wisconsin (USA), i.e. sealing does not enhance pavement performance. Taking into account these comparisons and the other experiences of UnJs presented in this chapter, the UnJs are a cost-effective alternative for JPCPs considering the needs of the pavement clients. In addition, more specific conclusions can be obtained of the work described in the present section:

The UnJs are highly recommended for JPCPs where the functional objective is the accessibility, for instance local streets or country roads. The UnJs are also highly recommended for industrial floors of warehouses used for general storage. In effect, these JPCPs are usually covered by a roof. Hence, the UnJs are not affected by the action of water.

The excellent results of the UnJs at the BCs of Santiago city in Chile (moderate rainfall) are very significant because BCs are the pavements with one of the highest traffic demands possible to find in cities. Then, from these experiences important conclusions can be obtained with respect to the use of UnJs in JPCP applications with less traffic solicitations. In Chile for instance, the BCs are classified between the highest functional categories with one of the highest traffic demands (MINVU, 2008). In particular the field measurements in UnJs were made up to 5,000,000 ESALs. These traffic demands can correspond to the accumulated ESALs in the complete life-cycle of other urban JPCPs applications as passageway (≤ 50,000 ESALs), local roads (≤ 200,000 ESALs), service roads (≤ 1,000,000 ESALs) and collector roads (≤ 3,000,000 ESALs) (MINVU, 2008). But also the UnJs can be considered feasible to use in industrial yards and rural roads with similar traffic demands as the mentioned categories of urban roads. For instance, the typical Chilean low volume roads need to resist less than 1,000,000 ESALs in their life-cycle (20 years).

Regarding the use of UnJs in ultra-high rainfall areas, WisDOT at USA has more than 50 years of excellent experience with UnJs in different conditions, even when the WisDOT UnJs are 3-6 mm wide and not less than 3 mm as current technology allows. The unsealed JPCPs of Guatemala (ultra-high rainfall) show values similar to the initial IRI even after 15,000,000 and 22,000,000 ESALs (Covarrubias, 2011). Hence, besides the JPCP applications at moderate rainfall mentioned previously, the UnJs can also be considered feasible to use in urban trunk roads (≤ 10,000,000 ESALs), expressways (≤ 20,000,000 ESALs) (MINVU, 2008), and also rural roads and industrial yards with similar traffic demands to the trunk roads or expressways in their life-cycle.

It is important to notice that if the UnJs are feasible to use in JPCPs at ultra-high rainfall, they can also be considered feasible for the same JPCP applications but in high, medium, moderate and low rainfall areas.

7.4. MODELLING THE DEVELOPMENT OF JOINT FAULTING

7.4.1. Deterioration model to predict the joint faulting

The available deterioration models were developed originally for traditional JPCPs. Consequently it is necessary to evaluate their applicability to short slabs. In particular, the model to predict the joint faulting is fundamental because JF is the major contributor to the deterioration of the ride quality of JPCP (Bustos et al, 2000; Jung et al, 2011).

There are different deterioration models to predict the JF in traditional JPCPs (Wu et al, 1993; Simpson et al, 1994; ERES, 1995; Owusu-Antwi et al, 1997; Yu et al, 1998; Titus-Glover et al, 1999; Hoerner et al, 2000). However, it is necessary to take into account some factors for the potential applicability of these models to short slabs. In this way, the useful models for this purpose need to fulfil the following requirements:

- Have a mechanistic component, i.e. to incorporate relevant variables that influence the development of joint faulting.
- Directly incorporate (and be sensitive to) variables that allow the comparison between traditional and short slabs JPCPs
- Allow the evaluation of non-dowelled JPCPs
- Have demonstrated to yield suitable predictions of joint faulting in traditional JPCPs (especially at the geographical locations of the short slabs projects)
- The variables required need to be available in projects of short slabs with joint faulting information

The fulfilment of these requirements provides a well-established base to model and evaluate the joint faulting in the new developed JPCPs with shorts slabs. In this way the model of ERES (1995) for non-dowelled JPCPs is chosen because not only it includes directly the slab length itself but also others relevant variables that influence the development of joint faulting, such as rainfall, characteristics of the base and traffic loads. In addition, the model of ERES (1995) has been included in the Highway Development and Management system HDM-4 which has been widely used around the world for the prediction of pavement deterioration.

As shorts slabs are a recent development, the number of projects with information of the joint faulting is limited. Then the availability of the variables required by the model is important. A Chilean company has patented certain aspects of the technology of short slabs (Covarrubias, 2011) and nowadays there is a concentration of short slabs projects in Chile. Information of the joint faulting and the variables required by the model of ERES (1995), represented by the Eq. 7.4.1, is available for these short slabs JPCPs. In addition, the models of HDM-4 have been calibrated for different conditions, in particular for traditional non-dowelled JPCPs at Chilean conditions (Límite Ingeniería, 2009). This contributes to a well-established base to the modelling of the joint faulting in short slabs (i.e. non-dowelled JPCPs) especially for the Chilean projects.

$$FaultND = f\,(ESALs, -\frac{h}{L}, -Cd, -Base, FI, MMP, -Days32, -Widened) \quad \text{(mm)} \qquad (7.4.1)$$

where:
FaultND = mean JF at transverse non-dowelled joints (mm)

ESALs = cumulative number of equivalent single axle loads of 80 kN (millions/lane)
h = slab thickness (mm)
L = slab length (m)
Cd = drainage coefficient of AASHTO
Base = 1 if there is a stabilized base, otherwise 0
FI = mean freezing index (°C-days)
MMP = mean monthly precipitation (mm/month)
Days32 = days with maximum air temperature greater than 32°C
Widened = 1 if there is a widened traffic lane, otherwise 0.

Roesler et al (2012) made a structural comparison between short slabs and traditional JPCPs using the same thickness for both alternatives. The same principle is applied for the functional comparison. In effect, from the analysis of the Eq. 7.4.1 it is possible to observe that *ceteris paribus*, with the exception of the slab length, the traditional JPCP should bring higher values of JF than the new design configuration with short slabs. This is show in Fig. 7.4.1 using the following data: h = 200 mm; Cd = 1.15; Base = 0 (not stabilized); FI = 55 °C-days; MMP= 27 mm; Days32 = 45; Widened = 0; L short slabs = 1.2 m; L traditional = 3.5 m.

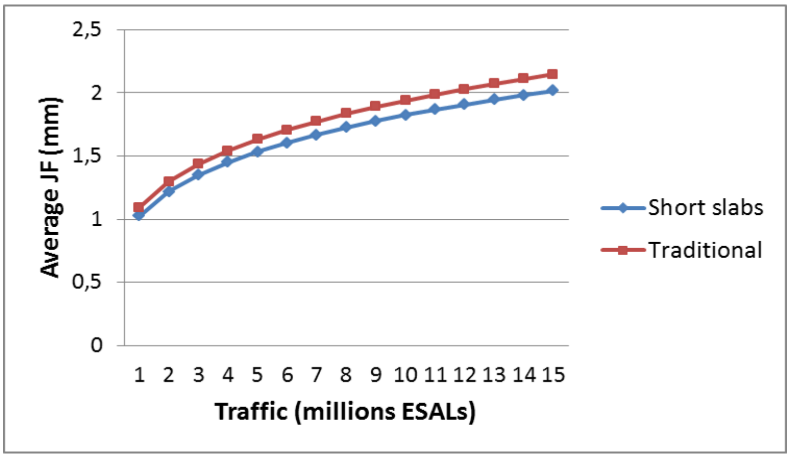

Fig.7.4.1. Development of average JF in traditional and short slabs JPCPs

7.4.2. Comparison of model results with trends of JF in short slabs

As shorts slabs are a recent development, the number of projects with information available of JF is limited. The projects included in the JF analysis of the present chapter are short slabs located in the cities of Santiago, Temuco and Puerto Montt in Chile (Salsilli et al, 2015), an Accelerated Pavement Testing (APT) at Illinois, USA (Roesler et al, 2012) and a bus lane at Concepcion city in Chile (Fig. 7.4.2). These projects represent different climatic conditions, they have different geometric configurations and traffic demands. In addition all these short slabs are built over a granular base which is the most unfavourable one for the production of JF.

Even when a comprehensive process of calibration of deteriorations models for short slabs (modification or creation of a new model) requires an improvement of the JF information of this new development, nowadays some interesting results and conclusions can be obtained from the clear trends that shorts slabs JPCPs are showing.

Fig. 7.4.2. JPCP with short slabs at a bus lane in Concepción city, Chile (Pradena and Houben, 2014a)

The data used in the modelling of the Fig. 7.4.1 correspond to one of the sections of the bus lane of Santiago city in Chile. And when the calibration factor of 0.51 for Chilean conditions is applied (Límite Ingeniería, 2009), the model yields an average JF 1.0 mm after 12,500,000 ESALs (Table 7.4.1). However, in the field no JF was detected (Salsilli et al, 2015). The Table 7.4.1 presents model results and the JF observed in the real-world short slabs JPCPs.

Table 7.4.1. Characteristics of the test sections, traffic demands and JF (Based on Roesler et al, 2012; Salsilli et al, 2015; Pradena and Houben, 2015c)

City	Latitude	Climate	Slab length (m)	Slab thickness (mm)	Traffic (millions ESALs)	Calculated JF (mm)	Real-world JF (mm)
Santiago	33º27' S	Mediterranean dry-summer	1.20	130	14.0	1.2	0
				140	14.0	1.2	
				150	12.5	1.1	
				170	14.0	1.1	
				200	12.5	1.0	
Concepcion	36º49' S	Maritime tempered with mediterranean influence	2.00	200[1]	1.5[2]	1.0	0
Temuco	38º45' S	Oceanic with mediterranean influence	0.88	80	0.35	0.7	0
			1.17	80		0.7	
			1.75	80		0.8	
				100		0.7	
				120		0.7	
Puerto Montt	41º30' S	Oceanic with abundant rainfall	1.75	80	0.20	0.7	0
				100		0.6	
				120		0.6	
Illinois (USA)	39º83' N	Humid continental	1.80	150	23.0	4.4	0
					17.0	4.0	
				200	20.0	4.0	
					51.0	5.0	

(1) Estimated from the Chilean structural design guidelines (MINVU, 2008)
(2) Estimated on basis of the frequency of buses and the in-service time (7 years).
(*) For the Chilean sections the calibration factor 0.51 was applied (Límite Ingeniería, 2009)
(**) The traffic is expressed in ESALs of 80 kN.

7.4.3. Analysis

As an example the Figure 7.4.3 shows the effect of the slab thickness in the modelled JF using the different slab thicknesses of the bus lane at Santiago city. According to the model, the thinner the slab the higher the joint faulting. However in the field no measurable JF was detected in any of the sections with different slab thicknesses (Salsilli et al, 2015). Hence, the model of ERES is not able to predict correctly the development of JF in short slabs.

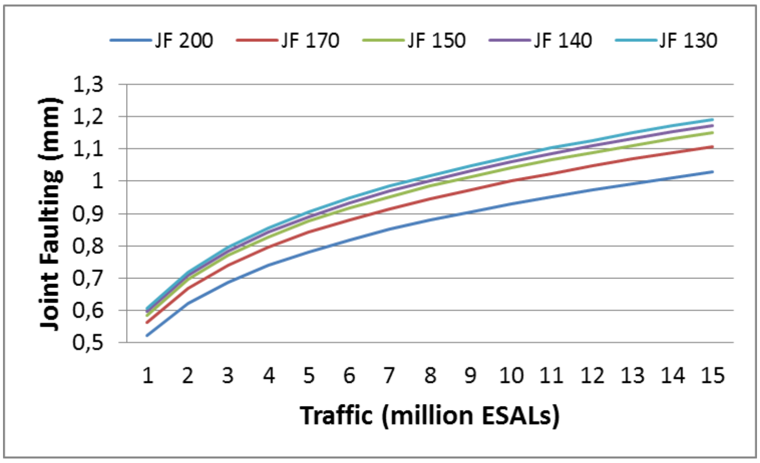

Fig. 7.4.3. Modelling results of JF for different concrete thicknesses of short slabs

The model of ERES is sensitive to the changes of slab thicknesses (Fig. 7.4.3), but it is not able to model correctly the effects of the slab length reduction of short slabs. The model of ERES was developed for predicting the JF of traditional JPCPs where the magnitude of the slab length reductions does not produce a radical difference in the LTE as in short slabs JPCPs. In effect, as presented in Chapter 5, short slabs JPCP with reduction of 50% of the slab length (with respect to the traditional JPCP) yields a reduction of 50% of the crack width under the joints. The Fig. 7.4.4 shows that this reduction produces a radical difference in the LTE by aggregate interlock, being the LTE fundamental in the production of JF (Ioannides et al, 1990; SHRP 2, 2011; NCHRP, 2012).

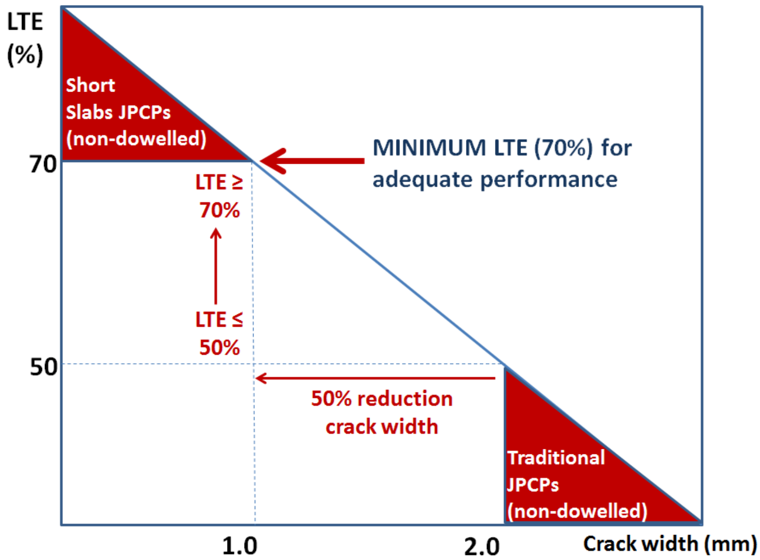

Fig. 7.4.4. LTE-crack width representation based on the results obtained in Chapter 5

In addition to the lower average JF per joint, the projects are showing that more joints not necessarily result in more accumulated JF per length of pavement (that can increase the JPCP roughness). In effect, even when the short slabs of the Table 7.4.1 have 2, 3 or even 4 times more joints than traditional JPCPs, no measurable JF was detected in the field. In fact, short slabs JPCPs in Guatemala show a maximum increase from the initial condition of only 0.25 m/km IRI after 22,000,0000 ESALs (Covarrubias, 2011). In effect, these short slabs have IRI ≤ 2.3 m/km after 9 million ESALs (average). This IRI value is not only good in interurban pavements but even more in urban ones where the speed is in general low. According to Shafizadeh et al (2002) the IRI is the most significant factor associated with drivers' acceptability of a roadway's condition. In addition, pavement roughness is strongly related to pavement performance providing a good, overall measure of the pavement condition and correlates well with subjective assessments (Loizos and Plati, 2008). Hence, despite the traditional practice to avoid more joints, the short slabs are showing that more joints not necessarily means more accumulated JF per length of pavement that can affect the ride quality and the satisfaction of the clients for pavements.

At the same avenue with the short slabs at Santiago of Chile, hence with a similar kind of traffic (Fig. 7.4.5), a thin concrete overlay (white topping) of a bus lane does not present signs of JF after more than 7 years in-service. This thin concrete overlay of 100 mm thickness has square slabs of 0.6*0.6 m. Hence, under a correct design and construction process, more joints (even more than short slabs) does not necessarily mean more accumulated JF per length of pavement.

Fig. 7.4.5. Concrete overlay at a bus lane of Santiago city, Chile
(Pradena and Houben, 2014a)

In the case of the short slabs of thin concrete overlays the stiffness of the underlying old pavement contributes to the integral behaviour of the concrete overlay allowing the reduction of the concrete thickness, provided that there is a durable good bond between the concrete overlay and the underlying asphalt. As JPCPs with short slabs can be thinner than the traditional ones, higher deflections can be expected (Roesler et al, 2012). Similar to the case of thin concrete overlays it is important to provide adequate stiffness of the layers below the short concrete slabs that contribute to the integral behaviour of these pavements. In effect, the JPCPs with short slabs presented in this section have good support under the short concrete slabs. For instance, in the bus lane of Santiago city the California Bearing Ratio (CBR) of the granular base is 85%. Similar is the case in Temuco and Puerto Montt (Chile), with CBR 90% and 88% respectively. In the case of the Guatemala short slabs some of the sections were built over the old deteriorated asphalt pavement and others over a granular base with an equivalent modulus of subgrade reaction (k) of 110 MPa/m (Covarrubias, 2011).

The amount of rainfall can affect the development of JF as well. Hence, precautions need to be taken especially in JPCPs situated in areas with high rainfall. For instance, a nonwoven geotextile prevents mixing of the subgrade and aggregate base layer, and adequate lateral drainage prevents lowering the support stiffness and unbound material shear strength (Roesler et al, 2012).

7.4.4. Necessity of a new JF model for short slabs

Taking into account the clear trends of JF shown by short slabs JPCPs, it is recommended to modify the model of ERES (1995) or to develop a new model able to represent better the JF of this new development. For this, a dedicated study is necessary including comprehensive field measurements of JF at short concrete slabs in different conditions and considering the LTE in the model. In effect, the LTE is fundamental in the production of JF (Ioannides et al., 1990; SHRP 2, 2011; NCHRP, 2012) and specifically in the case of non-dowelled JPCPs the prevention of JF relies on effective load transfer through aggregate interlock (Ioannides et al., 1990). In this way the 50% reduction of crack width under the joints of short slabs contribute decisively to the low JF of this innovation. Then, despite the traditional practice to avoid a large number of joints in the JPCPs, the reduction of joint spacing not only produces structural benefits (Roesler et al, 2012; Covarrubias, 2011; Salsilli et al., 2015) but also functional ones that not necessarily can be reflected by JF models developed for traditional JPCPs. For instance, the relevance that the model of ERES gives to the changes in slab thickness with respect to the reduction of slab length of short slabs is inadequate. Therefore, a model to predict the JF of short slabs must include the load transfer between slabs. As, in non-dowelled JPCPs, the LTE basically depends on the aggregate interlock, the LTE can be related with the crack width under the joints.

The final objective of the model is to serve to the necessities of the pavement clients. As it was mentioned, road users and transportation agencies assign priority to the ride quality of the pavement. In addition, the transportation agencies assign priority to the life-cycle cost effectiveness as well (Haas & Hudson, 1996). Then the model needs to predict adequately the JF in short slabs JPCPs and at the same time it needs to be practical and useful to the transportation agency. It includes the variables required by the modelling and the calibration process to evaluate short slabs JPCPs at different conditions.

7.5. EVALUATION OF THE RIDE QUALITY WITH AHP

7.5.1. The necessity of using a MCDM method

An integral analysis of pavements must include the needs of the clients for pavements which assign priority to the ride quality (Haas & Hudson, 1996). To evaluate the ride quality deterioration models with extensive databases to their development and calibration are required. As short slabs are a recent development, nowadays there are no databases available for this purpose, but there is empirical evidence for a functional evaluation in a transparent, explicit, traceable and well-sustained way using Multi-Criteria Decision-Making (MCDM) methods. This provides a transparent formal decision-making framework to support the decisions that anyway the owners or the agencies need to make (but neglecting the functional evaluation that is priority for the pavement clients). The main advantage of the multi-criteria analysis is the utility to simplify complex situations, decomposing, structuring them and then advancing to an objective and reasoned solution (Villegas, 2009). In the particular case of pavement-related decisions, MCDM methods have been used extensively (Azis, 1990; Masami, 1995; Kim & Bernardin, 2002; Cafiso et al, 2002; van Leest et al, 2006; Brownlee et al, 2007; Selih et al, 2008; Sharma et al, 2008; Selih et al, 2008; Farhan & Fwa, 2009; Sun & Gu, 2011; Kabir et al, 2014). In particular, Kabir et al (2014) made a classification of the MCDM methods used in infrastructure management between 1980-2012. In the particular cases of roads and pavements they found the Analytic Hierarchy Process (AHP) method (Saaty, 1980) the most used by the scientific community. And according to Cafiso et al (2002) the AHP method lends itself for application in road-related decisions.

The AHP method defines the problem and determines the kind of knowledge sought, structures the decision hierarchy from the top with the goal of the decision until the alternatives and constructs a set of pairwise comparison matrices using a scale 1 to 9 (Saaty, 1980). Mathematically the contribution of each attribute is determined using the priority vector or eigenvector. The result is the score of the relative importance of the alternatives involved in the AHP. The detailed mathematical methodology AHP can be found in Appendix C.

Although in the present thesis the AHP matrices are based on empirical evidence, variations in the weightings and sensitive analyses are performed in order to test if changes in the original preference are produced, or on the contrary, the original decision is confirmed after all the combinations analysed with AHP.

7.5.2. Objective and attributes of the AHP method

Perhaps the most important component of the MCDM process is the identification of the objective relevant to the problem of defining alternatives, together with their associated attributes. As the new design configuration needs to be compared with the traditional one in order to evaluate if effectively it represents an improvement (Montgomery, 2012) the objective is to select the non-dowelled JPCP with the better ride quality between the alternatives traditional and short slabs JPCPs (Fig. 7.5.1).

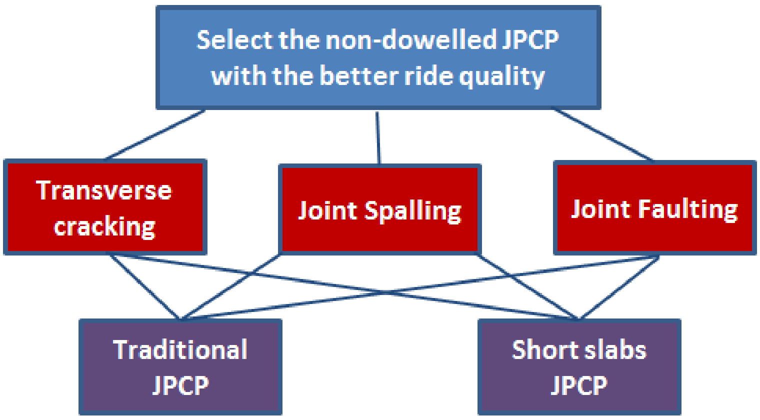

Fig. 7.5.1. Decomposition of the problem into a hierarchy of attributes and alternative decisions

According to Darter & Barenberg (1976), Al-Omari & Darter (1992), Salsilli et al (1991), ERES (1995), and Bustos et al (2000) the direct causes of the deterioration of the JPCPs ride quality are the transverse joint faulting, spalled joints and transverse cracked slabs. Even more, different deterioration models express the increment of the JPCP roughness as the sum of these deteriorations (or subtraction if the deterioration of the ride quality is modelled) (Al-Omari and Darter, 1992; ERES, 1995; Yu et al, 1998; Bustos et al, 2000). Then clearly these are the attributes to evaluate the ride quality of JPCPs (Figure 7.5.1).

The ride quality is also affected by the curvature of the concrete slabs. Still, it is not included in the attributes of the Fig. 7.5.1 because, as it was expressed in the previous paragraph, there is clear empirical evidence of the effects and interaction of the transverse cracking, the joint spalling and the joint faulting in the increment of the surface irregularity. However, this interaction is not that clear when the slab curvature is added. As the present work is based on empirical evidence, the slab curvature is not included directly in the AHP attributes, but considered later in section 7.5.7 (results and analysis) as an additional criterion for making the decision.

7.5.3. Quantification of the attributes

To build the matrices of the AHP method it is necessary to express the existing empirical evidence in the AHP scale from 1 to 9. The available empirical evidence of short slabs and unsealed joints is basically in urban and interurban JPCPs. However, the engineering principles behind are basically the same for the different applications. For instance, the JF is the major contributor to the deterioration of the JPCP ride quality (Bustos et al, 2000; Jung et al, 2011) and different studies have determined that the load transfer at joints is fundamental in the production of JF (Ioannides et al., 1990; SHRP 2, 2011; NCHRP, 2012). In non-dowelled JPCPs the main mechanism of load transfer is the aggregate interlock, and thus the crack width at joints. In the present thesis the model to predict this crack width at joints has been validated on airport aprons and industrial facilities. In this way, the AHP matrices for the evaluation of the ride quality of airport aprons and industrial floors (and yards) are similar to the ones for interurban and urban JPCPs. When differences are present, they are explicitly described in the sections 7.5.8 (industrial floors) and 7.5.9 (airport aprons).

The scope of the analysis is given for the available empirical evidence available (more details in section 7.5.10).

7.5.3.1. Transverse Cracking (TC)

Roesler et al (2012) made a comparison of short slabs and traditional JPCPs in terms of the Equivalent Single Axle Loads (ESALs) that both types of JPCPs can resist (with the same concrete thickness until the same level of cracking). In addition, Roesler et al (2012) found that short slabs have only 6% of transverse cracks, being the longitudinal cracking and the corner ones the prevalent cracking. Similar was the cracking observed in 13 test sections with shorts slabs in Chile (Salsilli et al, 2015). As the longitudinal cracking has a negligible effect on the ride quality, it is considered appropriate to express the matrix AHP based on the average ESALs resisted by traditional JPCPs (2.63 million ESALs) and short slabs JPCPs (22.32 million ESALs) found by Roesler et al (2012).

Table 7.5.1. Matrix AHP for the transverse cracking

	Traditional JPCPs	**Short slab JPCPs**
Traditional JPCPs	1	9
Short slab JPCPs	0.111	1

Additionally, variations in the values of Table 7.5.1 that reduce the gap between both types of JPCPs are made in the sensitivity analyses.

7.5.3.2. Joint Spalling (JS)

The UnJs are part of the design features of short slabs. On the contrary, traditional JPCPs have sealed joints. As presented in section 7.3.5, the experience of 50 years of WisDOT (USA) and the new field measurements performed in Chile lead to the conclusion that the joint seals do not enhance the pavement performance. Hence, it is possible to consider an AHP matrix full of 1, i.e. the same performance for both alternatives in terms of the joint spalling affecting the pavement performance. Nevertheless, sensitivity analyses are performed considering more joint spalling in the unsealed short slabs than in the sealed traditional JPCPs (sections 7.5.4 and 7.5.6)

7.5.3.3. Joint Faulting (JF)

The Table 7.5.2 shows field evidence of short slabs JPCPs projects without any JF in Santiago city (Latitude 33º27' S) (Salsilli et al, 2015) and in Illinois (Latitude 40º12' N) (Roesler et al, 2012) with 3 and 2 times more joints respectively than a traditional JPCP of 3.6 m slab length. Then more joints not necessarily mean more accumulated JF per km that can increase the JPCP roughness. In addition, short slabs JPCPs in Guatemala show an increase of only 0.2 m/km IRI after 9 million ESALs (Covarrubias, 2011).

Table 7.5.2. JF comparison between non-dowelled traditional and short slab JPCPs

Location	**Cumulative ESALs**	**Thickness (mm)**	**JF short[1] (mm)**	**JF trad[2] (mm)**	**Joints short/Joints trad**
Santiago (Chile)	14,000,000	200	0	1.2	3
Illinois (USA)	51,000,000	200	0	5.0	2

1.- Measured in the field 2.- Calculated with HDM-4 model (developed and calibrated for traditional JPCPs). For the JF Chile the calibration factor 0.51 for Chilean conditions is applied.

This excellent behaviour of the short slabs JPCPs can be explained by the drastic reduction of 50% of the crack width at joints which produces a radical change of the LTE, which is fundamental in the production of JF (Ioannides et al., 1990; SHRP 2, 2011; NCHRP, 2012). These fundamental changes of crack width and LTE were graphically represented by Fig. 7.4.4 where the LTE changes radically from the zone of insufficiency in the graph to the zone of adequate performance.

Based on this empirical evidence, and using a conservative approach, the JF per km of short slabs is considered 50% of the one of traditional non-dowelled JPCPs (Table 7.5.3).

Table 7.5.3. AHP matrix of the JF

	Traditional JPCPs	Short slab JPCPs
Traditional JPCPs	1	2
Short slab JPCPs	0.5	1

Additionally, variations in the values of Table 7.5.3 that reduce the gap between both types of JPCPs are made in the sensitivity analyses.

7.5.4. Sensitivity analysis of the attributes

Even when the present work is based on objective evidence, variations in the attributes values are made in order to understand how they influence the outcomes, if they can produce a different preference or, on the contrary, additional support is given to the original preference. The Table 7.5.4 presents reductions of the gap in transverse cracking and JF, and more joint spalling in short slabs JPCPs than in traditional JPCPs.

Table 7.5.4. Changes in the attributes for sensitivity analyses.

	Transverse Cracking		**Joint Spalling**		**Joint Faulting**	
	Trad JPCP	Short slabs	Trad JPCP	Short slabs	Trad JPCP	Short slabs
Trad JPCP	1	6	1	0.667	1	1.5
Short slabs	0.167	1	1.5	1	0.667	1

7.5.5. Weightings of the attributes

The weights represent the relative importance of the attribute. A weighting set can be deemed well substantiated if it can be easily explained, that is quite different than ‘using it as you see fit’ (van Leest et al, 2006).

7.5.5.1. Equal attributes weightings

In this case all the values of the AHP matrix are 1 in agreement with the principle of the use of equal weightings if the decision-maker has no reason to prefer one attribute to another (Brownlee et al, 2007).

7.5.5.2. Different attributes weightings

The major contributor to the JPCPs roughness is the JF (Bustos et al, 2000; Jung et al, 2011), and this is expressed in Table 7.5.5 with a double relative importance of the JF with respect to the joint spalling and transverse cracks.

Table 7.5.5. Double weighting of JF with respect to TC and JS

	Transverse cracking	Joint spalling	Joint faulting
Transverse cracking	1	1	0.5
Joint spalling	1	1	0.5
Joint faulting	2	2	1

In the Table 7.5.5 is possible to observe that the double relative importance of the JF automatically represents a 50% decreasing of the joint spalling and transverse cracks. However, from ERES (1995), Yu et al (1997) and Bustos et al (2000) it is possible to identify a higher relative effect of the joint spalling with respect to the transverse cracking in the increment of the JPCP roughness. This relative difference is expressed in Table 7.5.6 where it is observed the higher influence of JF only over the transverse cracking.

Table 7.5.6. Double weighting of JF with respect to the TC

	Transverse cracking	**Joint spalling**	**Joint faulting**
Transverse cracking	1	1	0.5
Joint spalling	1	1	1
Joint faulting	2	1	1

7.5.5.3. Sensitive analysis of the weightings

The weightings are changed for the same reasons that the variations of the attributes, i.e.to understand how they influence the outcomes, and find out whether a different preference is obtained or, on the contrary, additional support is given to the resulted preference. The relative influence of the joint faulting over the other deteriorations is changed to 3 times higher instead of 2 of Table 7.5.5. And similar to Table 7.5.6 (but with 3 instead of 2) it is considered that this influence is only over the transverse cracking. Moreover these changes are considered with 4 times instead of 3.

7.5.6. Extra-demanding sensitive analysis of the attributes

According to the empirical evidence (and the engineering principles behind) the scenarios of evaluation presented in Table 7.5.7 are hardly realistic. For instance, a gap of (only) 3 times between the transverse cracking of traditional and short slabs JPCPs. Anyway, these extra-demanding changes are included for further testing of the original preference.

Table 7.5.7. Extra-demanding changes in the attributes for sensitivity analyses.

	Transverse Cracking		**Joint Spalling**		**Joint Faulting**	
	Trad JPCP	Short slabs	Trad JPCP	Short slabs	Trad JPCP	Short slabs
Trad JPCP	1	3	1	0.5	1	1
Short slabs	0.333	1	2	1	1	1

7.5.7. Results and analysis

The Table 7.5.8 summarizes the results obtained from the 189 comparisons analysed with AHP. In effect, 189 are the possible combinations of the scenarios described in the sections 7.5.3 to 7.5.6.

The nomenclature of the attributes indicates the relative importance of the attribute in the short slabs with respect to the traditional JPCP (AHP scale). In this way, TC (1/9) for instance, indicates the AHP scale of the transverse cracking, i.e. 1 for short slabs and 9 for traditional JPCPs (Table 7.5.1).

The nomenclature of the weightings indicates the relative weighting importance of the JF with respect to the TC and the JS. In this way, JF (2-2) for instance, indicates the double weighting of JF with respect to TC and JS (Table 7.5.5) and JF (2-1) indicates the double weighting of JF only with respect to the TC (Table 7.5.6).

Every cell represents one combination of the scenarios under evaluation. For instance, the first row of the attributes with the second column of the weightings, i.e. the original attributes (sections 7.5.3.1 to 7.5.3.3) with the weightings JF (2-2) of the Table 7.5.5. The contribution of each attribute is determined using the priority vector of the AHP (details in Appendix C). The result is the score of the relative importance of the alternatives under analysis, in this case 0.68/0.32 (0.68 for traditional JPCP and 0.32 for short slabs). The higher the number in the cells is, the higher the deterioration of the ride quality.

Table 7.5.8. AHP results of the relative deterioration of the ride quality.

Attributes		Weightings			Sensitive weightings			
		Equal	JF(2-2)	JF(2-1)	JF(3-3)	JF(3-1)	JF (4-4)	JF(4-1)
Original:	TC(1/9)+JS(1/1)+JF(1/2)	**0.69/0.31**	**0.68/0.32**	**0.67/0.33**	**0.68/0.32**	**0.67/0.33**	**0.68/0.32**	**0.66/0.34**
Sensitivity attributes	TC (1/6)	**0.67/0.33**	**0.67/0.33**	**0.66/0.34**	**0.67/0.33**	**0.66/0.34**	**0.67/0.33**	**0.65/0.35**
	JS (1/0.7)	**0.66/0.34**	**0.66/0.34**	**0.64/0.36**	**0.66/0.34**	**0.63/0.37**	**0.66/0.34**	**0.63/0.37**
	JF (1/1.5)	**0.67/0.33**	**0.65/0.35**	**0.65/0.35**	**0.64/0.36**	**0.63/0.37**	**0.63/0.37**	**0.63/0.37**
	TC (1/6)+ JS (1/0.7)	**0.64/0.36**	**0.65/0.35**	**0.63/0.37**	**0.65/0.35**	**0.62/0.38**	**0.65/0.35**	**0.62/0.38**
	TC (1/6)+ JF(1/1.5)	**0.65/0.35**	**0.64/0.36**	**0.63/0.37**	**0.63/0.37**	**0.62/0.38**	**0.63/0.37**	**0.62/0.38**
	JS (1/0.7)+JF(1/1.5)	**0.63/0.37**	**0.62/0.38**	**0.61/0.39**	**0.62/0.38**	**0.60/0.40**	**0.62/0.38**	**0.60/0.40**
	TC(1/6)+ JS (1/0.7))+JF(1/1.5	**0.62/0.38**	**0.60/0.40**	**0.60/0.40**	**0.61/0.39**	**0.59/0.41**	**0.61/0.39**	**0.59/0.41**
	TC(1/3)	0.64/0.36	0.65/0.35	0.63/0.37	0.65/0.35	0.63/0.37	0.65/0.35	0.63/0.37
	TC(1/3)+ JS (1/0.7)	0.61/0.39	0.62/0.38	0.60/0.40	0.63/0.37	0.60/0.40	0.64/0.36	0.60/0.40
	TC(1/3)+JF(1/1.5)	0.62/0.38	0.61/0.39	0.61/0.39	0.61/0.39	0.60/0.40	0.61/0.39	0.60/0.40
	TC(1/3)+ JS (1/0.7)+JF(1/1.5)	0.58/0.42	0.59/0.41	0.57/0.43	0.59/0.41	0.57/0.43	0.59/0.41	0.57/0.43
	JS (1/0.5)	0.63/0.37	0.64/0.36	0.62/0.38	0.65/0.35	0.61/0.39	0.65/0.35	0.61/0.39
	JS (1/0.5)+TC(1/6)	0.62/0.38	0.63/0.37	0.61/0.39	0.64/0.36	0.60/0.40	0.64/0.36	0.60/0.40
	JS (1/0.5))+JF(1/1.5)	0.61/0.39	0.61/0.39	0.59/0.41	0.61/0.39	0.58/0.42	0.61/0.39	0.58/0.42
	JS (1/0.5)+TC(1/6)+JF(1/1.5)	0.60/0.40	0.60/0.40	0.58/0.42	0.60/0.40	0.57/0.43	0.60/0.40	0.57/0.43
	JS (1/0.5)+TC(1/3)	0.58/0.42	0.60/0.40	0.58/0.42	0.62/0.38	0.58/0.42	0.63/0.37	0.58/0.42
	JS (1/0.5)+TC(1/3)+JF(1/1.5)	0.56/0.44	0.57/0.43	0.55/0.45	0.58/0.42	0.55/0.45	0.58/0.42	0.55/0.45
	JF(1/1)	0.63/0.37	0.60/0.40	0.60/0.40	0.58/0.42	0.59/0.41	0.57/0.43	0.58/0.42
	JF(1/1)+TC(1/6)	0.62/0.38	0.59/0.41	0.59/0.41	0.57/0.43	0.58/0.42	0.56/0.44	0.57/0.43
	JF(1/1)+ JS (1/0.7)	0.60/0.40	0.58/0.42	0.57/0.43	0.56/0.44	0.56/0.44	0.55/0.45	0.55/0.45
	JF(1/1)+ TC (1/6)+JS (1/0.7)	0.59/0.41	0.56/0.44	0.56/0.44	0.55/0.45	0.55/0.45	0.54/0.46	0.54/0.46
	JF(1/1)+TC(1/3)	0.58/0.42	0.56/0.44	0.57/0.43	0.55/0.45	0.56/0.44	0.54/0.46	0.55/0.45
	JF(1/1)+T C(1/3)+JS (1/0.7)	0.55/0.45	0.54/0.46	0.53/0.47	0.53/0.47	0.52/0.48	0.53/0.47	0.52/0.48
	JF(1/1)+JS (1/0.5)	0.58/0.42	0.56/0.44	0.55/0.45	0.55/0.45	0.54/0.46	0.54/0.46	0.53/0.47
	JF(1/1)+JS (1/0.5)+TC(1/6)	0.56/0.44	0.55/0.45	0.54/0.46	0.54/0.46	0.53/0.47	0.53/0.47	0.52/0.48
	JF(1/1)+JS (1/0.5)+TC(1/3)	0.53/0.47	0.52/0.48	0.51/0.49	0.52/0.48	**0.50/0.50**	0.51/0.49	**0.50/0.50**

The upper highlighted section of the Table 7.5.8 includes the original scenario of evaluation and the sensitive analyses, i.e. the scenarios based in the empirical evidence plus variations to test if a different preference is produced or, on the contrary, the resulted preference is confirmed as it is in the present case. In effect, in the 56 combinations of the highlighted frame the value of the cells is always higher for traditional JPCPs, i.e. this non-dowelled JPCP has higher deterioration of the ride quality. In this way, short slabs are the non-dowelled JPCP with the better ride quality.

According to the empirical evidence (and the engineering principles behind) the scenarios of evaluation outside of the highlighted upper section of Table 7.5.8 are hardly realistic. Anyway, these extra-demanding changes have been included for further testing of the original preference. The Table 7.5.8 presents 133 combinations outside the upper highlighted section, in 131 of them the value is higher for traditional JPCPs and in only 2 cells the value is the same for both alternatives (hardly realistic scenarios). In this way it is possible to conclude that, even after being intensely tested, the original preference is confirmed, i.e. the short slabs are the non-dowelled JPCPs with the better ride quality.

The Fig. 7.5.2 incorporates the slab curvature as additional criterion of comparison. Short slabs can have even ¼ of the slab length of traditional JPCPs. Using a conservative approach, a reduction of only 50% of the traditional slab length has been considered, i.e. 50% of the slab curvature (Fig. 7.5.2 right). Even with a conservative approach for the slab curvature and the AHP score (lowest difference between alternatives of the highlighted part of Table 7.5.8, i.e. 0.59/0.41), the traditional non-dowelled slabs clearly have a higher surface irregularity, result of the AHP evaluation and the slab curvature (Fig. 7.5.2). Indeed, besides the higher relative curling, the Fig. 7.5.2 shows the higher AHP score for the traditional JPCP (0.59), i.e. higher surface irregularity.

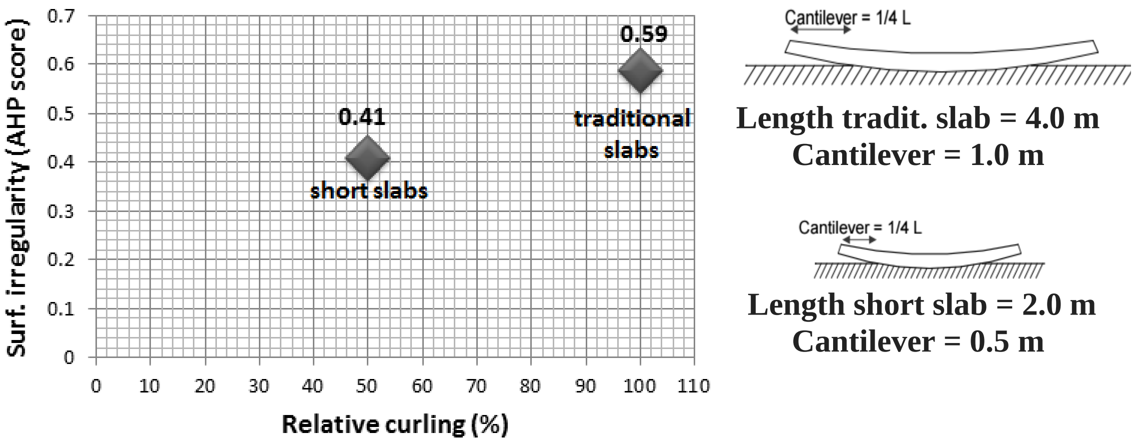

Fig. 7.5.2. Slab curvature added to the evaluation of the ride quality

7.5.8. AHP for industrial yards and floors

7.5.8.1. Industrial yards

In the present thesis also industrial yards, in which the traffic primarily consists of trucks, are evaluated. This traffic demand is similar to the one in urban and interurban JPCPs. Hence, the AHP evaluation is basically the same as the one previously presented for urban and interurban JPCPs.

7.5.8.2. Industrial floors

In the case of industrial floors the surface irregularities can affect the industrial operations (traffic of forklifts for instance). The direct cause of the increment of the surface irregularity is the sum of deteriorations in the pavement, in particular the joint faulting, joint spalling and transverse cracking (Al-Omari and Darter, 1992; ERES, 1995; Yu et al, 1998; Bustos et al, 2000) which are basically the deteriorations that a JPCP applied to industrial floors experiences as well.

Regarding the transverse cracking, the forklifts frequently change direction and then the longitudinal cracks could be transverse cracking as well. Still in these cases the differences in the traffic resisted by both JPCPs remain. However, using a conservative approach, a reduction in the gap between alternatives is made. In this way the Table 7.5.1 is replaced by the Table 7.5.9 for the case of industrial floors.

Table 7.5.9. Matrix AHP for the transverse cracking in industrial floors

	Industrial floors	
	Trad slabs	Short slabs
Trad slabs	1	6
Short slabs	0.167	1

In section 7.3.4.4 an evaluation of the UnJs in industrial floors was made. Accordingly the AHP scale of joint spalling presented in the section 7.5.3.2 remains (with the sensitive analyses indicated by the Tables 7.5.10 and 7.5.11).

Regarding the joint faulting, the excellent behaviour of short slabs can be explained by the drastic reduction of 50% of the crack width at joints which produces a radical change of the aggregate interlock. This reduction was corroborated by Pradena and Houben (2015b) in the validation of the model to predict these crack widths in JPCPs with short slabs applied to industrial floors. Hence, the Table 7.5.3 remains.

In the case of the sensitivity analysis of the attributes, the Table 7.5.4 only changes in the values for the transverse cracking shown in Table 7.5.10.

Table 7.5.10. Changes in the attributes for sensitivity analyses in industrial floors.

	Transverse Cracking		**Joint Spalling**		**Joint Faulting**	
	Trad JPCP	Short slabs	Trad JPCP	Short slabs	Trad JPCP	Short slabs
Trad JPCP	1	3	1	0.667	1	1.5
Short slabs	0.333	1	1.5	1	0.667	1

In this way the extra-demanding sensitive analysis for industrial floors is represented by the Table 7.5.11.

Table 7.5.11. Extra-demanding changes for sensitivity analyses in industrial floors.

	Joint Spalling		**Joint Faulting**	
	Trad JPCP	Short slabs	Trad JPCP	Short slabs
Trad JPCP	1	0.5	1	1
Short slabs	2	1	1	1

The 126 combinations obtained for the case of industrial floors are represented in the Table 7.5.12. The upper part of Table 7.5.12 includes a highlighted area that corresponds to the original scenario of evaluation and the sensitive analyses of the industrial floors, i.e. the scenarios based on the empirical evidence plus variations to test if a different preference is produced or, on the contrary, the resulted preference is confirmed as it is in the present case. In effect, in the 56 combinations of the highlighted part, the value of the cells is always higher for traditional JPCPs, i.e. this non-dowelled JPCP has higher deterioration of the surface regularity. In this way, short slabs are the non-dowelled JPCP with the better ride quality.

Table 7.5.12. AHP results of the relative ride quality deterioration in industrial floors.

Attributes[1]	Weightings[2]			Sensitive weightings[2]			
	Equal	JF(2-2)	JF(2-1)	JF(3-3)	JF(3-1)	JF (4-4)	JF(4-1)
Original: TC (1/6)+JS(1/1)+JF(1/2)	**0.67/0.33**	**0.67/0.33**	**0.66/0.34**	**0.67/0.33**	**0.66/0.34**	**0.67/0.33**	**0.65/0.35**
TC (1/6)+JS (1/0.7)	**0.64/0.36**	**0.65/0.35**	**0.63/0.37**	**0.65/0.35**	**0.62/0.38**	**0.65/0.35**	**0.62/0.38**
TC (1/6)+ JF(1/1.5)	**0.65/0.35**	**0.64/0.36**	**0.63/0.37**	**0.63/0.37**	**0.62/0.38**	**0.63/0.37**	**0.62/0.38**
TC(1/6)+JS (1/0.7)+JF(1/1.5)	**0.62/0.38**	**0.60/0.40**	**0.60/0.40**	**0.61/0.39**	**0.59/0.41**	**0.61/0.39**	**0.59/0.41**
TC(1/3)	**0.64/0.36**	**0.65/0.35**	**0.63/0.37**	**0.65/0.35**	**0.63/0.37**	**0.65/0.35**	**0.63/0.37**
TC(1/3)+JS (1/0.7)	**0.61/0.39**	**0.62/0.38**	**0.60/0.40**	**0.63/0.37**	**0.60/0.40**	**0.64/0.36**	**0.60/0.40**
TC(1/3)+JF(1/1.5)	**0.62/0.38**	**0.61/0.39**	**0.61/0.39**	**0.61/0.39**	**0.60/0.40**	**0.61/0.39**	**0.60/0.40**
TC(1/3)+JS (1/0.7))+JF(1/1.5)	**0.58/0.42**	**0.59/0.41**	**0.57/0.43**	**0.59/0.41**	**0.57/0.43**	**0.59/0.41**	**0.57/0.43**
JS(1/0.5)+TC(1/6)	0.62/0.38	0.63/0.37	0.61/0.39	0.64/0.36	0.60/0.40	0.64/0.36	0.60/0.40
JS(1/0.5)+TC(1/6)+JF(1/1.5)	0.60/0.40	0.60/0.40	0.58/0.42	0.60/0.40	0.57/0.43	0.60/0.40	0.57/0.43
JS(1/0.5)+TC(1/3)	0.58/0.42	0.60/0.40	0.58/0.42	0.62/0.38	0.58/0.42	0.63/0.37	0.58/0.42
JS(1/0.5)+TC(1/3)+JF(1/1.5)	0.56/0.44	0.57/0.43	0.55/0.45	0.58/0.42	0.55/0.45	0.58/0.42	0.55/0.45
JF(1/1)+TC(1/6)	0.62/0.38	0.59/0.41	0.59/0.41	0.57/0.43	0.58/0.42	0.56/0.44	0.57/0.43
JF(1/1)+ TC (1/6)+JS (1/0.7)	0.59/0.41	0.56/0.44	0.56/0.44	0.55/0.45	0.55/0.45	0.54/0.46	0.54/0.46
JF(1/1)+TC(1/3)	0.58/0.42	0.56/0.44	0.57/0.43	0.55/0.45	0.56/0.44	0.54/0.46	0.55/0.45
JF(1/1)+TC(1/3)+JS (1/0.7)	0.55/0.45	0.54/0.46	0.53/0.47	0.53/0.47	0.52/0.48	0.53/0.47	0.52/0.48
JF(1/1)+JS(1/0.5)+TC(1/6)	0.56/0.44	0.55/0.45	0.54/0.46	0.54/0.46	0.53/0.47	0.53/0.47	0.52/0.48
JF(1/1)+JS(1/0.5)+TC(1/3)	0.53/0.47	0.52/0.48	0.51/0.49	0.52/0.48	0.50/0.50	0.51/0.49	0.50/0.50

(1) The nomenclature indicates the relative attribute importance of the short slabs with respect to the traditional JPCP
(2) The nomenclature indicates the relative weighting importance of the JF with respect to the TC and the JS

According to the empirical evidence (and the engineering principles behind) the 70 combinations outside the highlighted part are hardly realistic. Anyway, these extra-demanding changes have been included for further testing of the original preference. In 68 of the combinations the value is higher for traditional slabs and in only 2 cells it is the same for both alternatives (hardly realistic scenarios). Then even after being intensely tested, the original preference is confirmed, i.e. the short slabs are the non-dowelled JPCPs with the better ride quality.

The reduction of slab curvature due to the shorter slabs is valid in industrial floors as well.

7.5.9. AHP for airport aprons

Jointed Plain Concrete Pavements (JPCPs) have been applied traditionally in airport aprons where pilots, passengers, airline owners and shippers of goods assign high priority to the ride quality (Haas & Hudson, 1996).

The Federal Aviation administration (FAA) of USA (2000) studied the effects of the slab size (joint spacing) on the performance of JPCPs up to 30 years old of 174 airports in USA (plus Hawaii and Japan) with a total area of 288 million square feet. The study included statistical and finite element analyses, and the performance of JPCPs was quantified by means of the Pavement Condition Index (PCI) which includes the deteriorations TC, JS and JF. The study concluded the better performance of shorter slabs and the faster deteriorations of longer slabs (results confirmed by Parsons and Pullen, 2014). The empirical evidence is particularly clear in aprons. Indeed, the study strongly recommends smaller slabs for airport aprons (FAA, 2000). This study was made for traditional JPCPs but it includes analyses with finite elements of the total stresses in the slabs, i.e. not only the wheel load induced ones, but also the stresses induced by the differences in the slab curvature due to the different slabs length. This is basically the same structural analysis as the one presented in Chapter 6 that shows the lower induced stresses of the innovative short concrete slabs.

Considering the studies mentioned in the previous paragraph, and using a conservative approach, the AHP evaluation for airport aprons is considered the same as that for industrial floors (including the sensitive analyses indicated by Tables 7.5.10 and 7.5.11) due to the following reasons as well:

- To take into account the possible differences in the traffic loads at airports with the empirical evidence available, in the case or airport aprons also a reduction of the gap is made in the original AHP matrix of transverse cracking. In this way the Table 7.5.9 is considered appropriate to represent the AHP scale for airport aprons. It is important to highlight that the evaluation of the PCI in airports JPCPs includes the linear cracking, corner break and divided slabs.

- As it has been mentioned, the JF is the major contributor of the ride quality deterioration in JPCPs. And in non-dowelled JPCPs, the load transfer at joints (depending on the aggregate interlock) is fundamental in the production of JF. Then, the excellent behaviour of short slabs regarding the JF can be explained by the crack width at joints. And in the present thesis the model to predict the crack width at joints of JPCPs has been validated on airport aprons. Hence, the Table 7.5.3. is also considered appropriate to represent the AHP scale of JF in airport aprons

In this way the AHP combinations expressed in Table 7.5.12 are valid for airport aprons as well. Accordingly it is possible to conclude that the short slabs are the non-dowelled JPCPs with the better ride quality at airport aprons as well. Furthermore, the slab curvature reduction due to the shorter slabs is valid at airport aprons as well.

7.5.10. Recommendations

In Chapter 9 recommendations are given in order to respect the engineering principles the favourable functional behaviour of short slabs is based on. In this way, recommendations are given for the minimum saw-cut depth with respect to the pavement thickness (to avoid UnCrJ), the maximum saw-cutting width and the fines content in the granular base, between others.

In addition, it is recommended to continue the follow-up of the short slabs projects and to improve the functional database of this innovation. Even when the empirical evidence presented in this chapter (and applied to the AHP evaluation) is up to 20 million ESALs, the engineering principles behind the favourable functional performance of short slabs leads to think optimistically over the application of short slabs to cases with higher traffic demands. Anyway, as it was presented in this chapter, 20 million ESALs is the traffic demand of the entire life cycle of numerous JPCPs applications.

7.6. VARIATION OF THE RIDE QUALITY OF JPCPs

7.6.1. Functional effects of the slab concavity

In section 7.5.7 the slab curvature was considered as an additional criterion for the comparative ride quality provided by non-dowelled traditional and short slabs JPCPs. However, the slab concavity not only depends on the permanent component but also on the transient one, depending on the changes of temperature and moisture. Due to this transient component, the users can perceive a different ride quality at different times of the day, for instance in the morning, when most of the users will use the pavement to commute to their daily activities and a general upward concavity is present (Poblete et al, 1988). These potential differences in the ride quality also present a challenge for the transportation agencies when they need to decide about the reception of new JPCPs or rehabilitation works.

Different investigations have evaluated the effects of the slabs curvature on the ride quality of JPCPs (Karamihas et al., 2001; Byrum, 2005; Siddique & Hossain, 2005; Chang et al., 2008; Johnson et al., 2010; Karamihas & Senn, 2012). However, all these investigations have evaluated traditional JPCPs, but not short slabs JPCPs.

Byrum (2005) established that for the same magnitude of slab curvature longer slab lengths result in more joint uplift and steeper slopes at joints. Thus, if the slabs are shorter the joint uplift is less and the slopes at joints are less steep. Here an important question arises; Are short slabs able to maintain a stable ride quality at different times of the day? The objective of the present section of the thesis is to compare the variation of the ride quality in traditional and short slabs JPCPs. The ride quality is quantified by the International Roughness Index (IRI), which is the best single predictor of drivers-perceived road roughness and drivers´ acceptability of a roadway´s condition, according to Shafizadeh et al. (2002).

7.6.2. Basis of the evaluation

A high concavity of the slabs is in general related with a high built-in curling which has been reported at different regions due to the local climate conditions (Roesler et al, 2012). In particular Poblete et al (1988) and Poblete et al (1990) summarize the results of a comprehensive study performed in Chile regarding curling and warping at different climatic conditions (21 test sections between the latitudes 33°2′S and 40°10′S). The work of Poblete et al (1988) has been part of the baseline investigations for different research studies (ERES, 2001; Chou et al., 2004; Hiller & Roesler, 2005; Rao & Roesler, 2005; Kohler & Roesler, 2005; Wells et al. 2006; Vandenbossche & Wells, 2006; Kim, 2006; Bordelon et al, 2009; Lederle & Hiller, 2011; Lederle et al., 2011; Roesler et al., 2012) and it is the base of the present study as well, not only for the valuable and recognized information of the study itself but also because the investigations of Poblete et al. (1988) and Poblete et al. (1990) were done in Chile where nowadays there is a concentration of projects of the innovative short slabs JPCPs. In fact, certain aspects of this technology have been patented by a Chilean company (Covarrubias, 2011). In addition, the National Highway Laboratory of the Chilean Highway Agency is an organism committed with research that has provided devices (as the walking profiler), equipment, staff and safety assistance to perform the field measurements of the present section.

The measures carried out periodically by Poblete et al (1988) and Poblete et al (1990) have clearly shown the existence of an upward concavity in the slabs. This upward concavity is the product of an upward curling (Poblete et al, 1988) and an upward warping which has a significant permanent component that has to be added to the predominant thermic concavity (Poblete et al, 1990).

Byrum (2000) establishes as well that a majority of rigid pavements experience a permanent upward curl regardless of the cyclical temperature gradient. Later, Byrum

(2005) made an analysis over 1000 JPCP profiles confirming that there is a general upward curvature. Hiller and Roesler (2005) report that this upward curling of concrete slabs has been observed in a variety of different climatic regions and particularly in Chile (Poblete et al, 1988) where the present study has been performed.

7.6.3. Timing of the measurements

Karamihas et al (2001) investigated the diurnal changes in the profiles of JPCPs. Following the premise that the changes in slab curvature on a daily cycle are primarily caused by the temperature gradient (Lowrie and Nowlen, 1960), Karamihas et al. (2001) made measurements between 5:07 am until 3:42 pm including the data of the air temperature. Karamihas and Senn (2012) also reported the collection of profiles between early morning and afternoon.

Poblete et al. (1988) report the minimum value of the upward concavity (i.e. when the slab is fully supported by the base) is reached in summer at times of strong positive gradient between 11:00 am to 3:00 pm. In addition Poblete et al. (1988) establish that the maximum upward concavity occurs generally around 7:00 am. Then in order to capture the maximum difference that the user can experience during a day, the field measurements in the present research are made at summer (January) early in the morning (around 7:00 am), i.e. maximum upward concavity and around noon (11:00 am to 2:00 pm), i.e. minimum upward concavity.

The daily variations of slab curvature are basically produced by the changes of the slab temperature between early in the morning and around noon (Lowrie and Nowlen, 1960; Poblete et al, 1988). For the quantification of these temperature differences, Karamihas et al (2001) used the air temperature. However, in the present study an infrared thermometer is used to quantify these temperature differences at the slabs surface. The objective is to ascertain that the variation of the temperature is the same in all the test sections, i.e. the differences in the slab surface temperature between the measurements of IRI early in the morning and noon.

7.6.4. Measurements of IRI with Walking Profiler

To compare the effects on the ride quality produced by the changes in the slab curvature in traditional and short slabs JPCPs, precise measurements of IRI every 10 m are made on these 2 types of JPCPs. In order to obtain this detailed comparison of IRI, the measurements need to be made with a device that guarantees high quality precision. The Walking Profiler (WP) was developed by the Australian Road Research Board as an alternative to the precision profilers as stationary inclinometers. The WP offers a faster operational speed than the stationary inclinometers keeping the high quality precision with sampling interval (length) 241.3 mm, height measurement precision ± 0.01 mm/step, and correlation R^2 = 0.999 between WP and rod and level derived IRI. The WP is not only classified as a Class 1 device in the classification of the World Bank (Sayers et al, 1986) but also WP is reference equipment for calibrating and assessing other World Bank Class 1 equipment as high speed profilometer. In addition, Bertrand et al (1991) indicates that stationary inclinometers are extremely sensitive to how they are operated. On the contrary, Morrow et al (2005) corroborates that the WP is relatively operator independent as the 'walking' is automated, with the operator responsible only for pushing the device (Fig. 7.6.1). Furthermore, the WP is an improvement in terms of keeping the track of the measurements (Fig. 7.6.1 left) which helps to avoid the variations of the lateral position reported by Karamihas et al (2001) when the measurements are made with the high speed profiler (variations up to 300 mm).

Fig.7.6.1. Measurements performed with the WP. Checking the track position (left) and measurements with safety assistance (right)

The measurements of IRI are made at short slabs of the Route 60-Ch ‘Christ the Redeemer’ (Chile) which is the main transport route between Chile and Argentina and so carries quite heavy traffic (1500 trucks/day) (Fig. 7.6.2 left). As the short slabs are non-dowelled JPCPs, no comparative measurements are necessary between dowelled and undowelled pavements. However, these kinds of comparative measurements are necessary at traditional JPCPs. The IRI measurements over dowelled traditional JPCPs are performed in the Route 60-Ch, Chile. And the measurements of IRI over non-dowelled traditional JPCPs are made in the O´Higgins Park of Santiago City, Chile. This JPCP is part of the so-called ellipse of the park, a space used for different citizen activities and events (Fig. 7.6.2 right). This JPCP does not have regular traffic of vehicles and then no special safety assistance is required for the measurements. On the contrary, this safety assistance is necessary on the Route 60-Ch (Fig. 7.6.1 right).

Fig. 7.6.2. Measurements performed with WP at Route 60 Ch (left) and Santiago city (right)

The Table 7.6.1 shows the characteristics and number of test sections per type of JPCP.

Table 7.6.1. Characteristics and number of test sections per type of JPCP

Location	Latitude	Type of JPCP	Slab length	Test sections
60-Ch	32° 50′S	Short slabs	2.0	2
60-Ch	32° 50′S	Short slabs with fibres	2.0	3
60-Ch	32° 50′S	Traditional dowelled	4.0	1
Santiago city	33° 27'S	Traditional non-dowelled	5.5	2

As in the present study the emphasis is on short slabs JPCPs, additional measurements of IRI are made in these sections at 4:00 am for further comparison. And taking advantage of the absence of regular vehicles at the JPCP of the O´Higgins Park, further measurements are made in 2 additional sections (at 0.3 m from the longitudinal joint).

The locations of the measurements of IRI in the pavement itself (tracks) are defined following 2 criteria:

- Measurements at the wheel path (Track 1): 0.75 m ± 0.25 m from the longitudinal joint or pavement edge (Pasetto and Manganaro, 2007).
- Measurements to include the slab corner uplift effect (Track 2): 0.3 m from the longitudinal joint.

As mentioned, the measurements of IRI are made every 10 m in order to have a detailed survey of the effects of the slab curvature on IRI. The length of the test sections has been defined as 100 m considering this detailed survey and the maximum continuous length of the sections of short slabs on the Route 60-Ch (500 m). In addition, the Route 60-Ch is a sinuous mountainous road (Fig. 7.6.2 left) and then the measurements of IRI can be distorted by the so-called 'Geometric IRI', i.e. the alteration of the IRI-value due to the changes of the transverse slopes at curves (more details can be found in Pradena et al, 2009). For that it is necessary to concentrate the measurements in sections between curves, considering the space required by the safety assistance as well (Fig. 7.6.1 right).

As in the present study the emphasis is on short slabs JPCPs, measurements of IRI performed with high speed laser profilometer at the Route M-50 'Cauquenes-Chanco' in Chile (Latitude 36 ° 00'S) are included in the analysis as well. The IRI is obtained every 50 m at the wheel-tracks of 2 lanes over 12 km of the pavement with short slabs of 2.2 m (without fibres). In this case sections of 1 km have been defined and the timing of the measurements is at the pre-established 7:00 am and 1:00 pm. Furthermore measurements of IRI at 10:00 am, 4:00 pm and 7:00 pm are included for further comparison with the IRI at 7:00 am and 1:00 pm.

7.6.5. Results and analysis

At reception of new JPCPs or maintenance interventions at pavement management, a representative value of IRI is used. In general the average IRI of a pavement section is the representative value useful for decision purposes. In the present study it is the same, for decision purposes the representative IRI of every section is the average of the individual IRI-values. Then this average IRI is the useful value to evaluate the variation of the ride quality, making the comparison of the IRI measurements at different times of day. For the sections of 100 m the average of the IRI-values every 10 m is the representative IRI useful to make the comparison of the IRI measurements between noon and 7:00 am ($\Delta IRI_{7am\text{-}1pm}$)

and 4:00 am ($\Delta IRI_{4am-1pm}$) respectively as it is shown in Figs. 7.6.3 to 7.6.5. In these figures the values in blue represents the average IRI_{1pm} and the values in red represents the $\Delta IRI_{7am-1pm}$ and $\Delta IRI_{4am-1pm}$. The constant differential temperature of the slabs between the measurements early in the morning and noon is 30°C. In all the figures the results are presented for every track/section, i.e. the specific track where the measurement of IRI were performed and the section of that track. For instance, track/section 1/2 represents the results for the track 1 of the section 2.

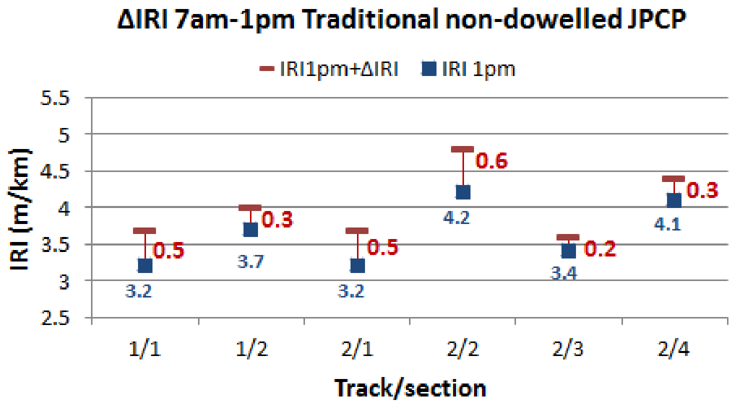

Fig. 7.6.3. Results of $\Delta IRI_{7am-1pm}$ for traditional non-dowelled JPCP

The Fig. 7.6.3 includes the $\Delta IRI_{7am-1pm}$ results of the 2 sections of Table 7.6.1 (track 1 and 2) and the 2 additional sections of O´Higgins Park (only track 2). The results of Fig. 7.6.3 for non-dowelled traditional JPCPs are in agreement with the ones obtained by Byrum (2005), Chang et al (2008), Johnson et al (2010), and Karamihas and Senn (2012) in terms of an increment of the IRI with the slab curvature. In particular Chang et al (2008) quantified the impact of curling and warping based on 38 traditional JPCPs finding that diurnal impacts can be as high as 0.6 m/km, which is in agreement with the results shown in Fig. 7.6.3. Karamihas et al (2001) found that these diurnal changes can be 0.4 m/km. In traditional non-dowelled JPCPs, the maximum ΔIRI is produced at the track 2 (0.3 m of the longitudinal joint). The possible explanation of it is the uplift of the slab corner. However this effect is not bigger due to the restriction produced by the adjacent slabs of the JPCP at the O´Higgins Park. This kind of restriction was described by Poblete et al (1988) as well.

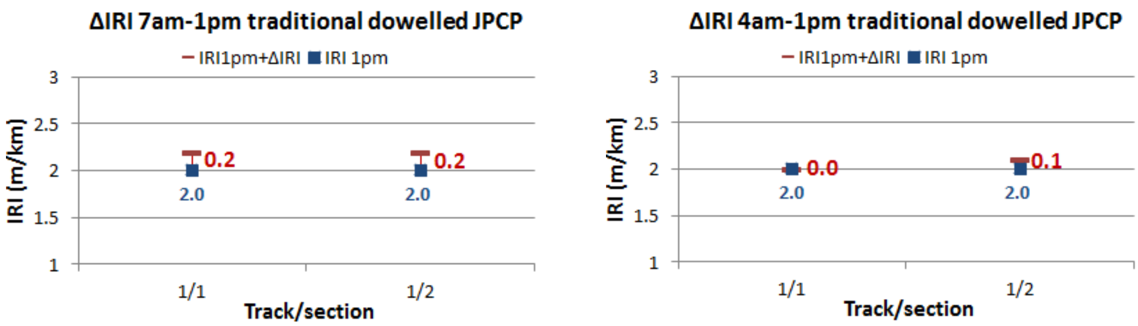

Fig. 7.6.4. Results of $\Delta IRI_{7am-1pm}$ and $\Delta IRI_{4am-1pm}$ for traditional dowelled JPCP.

The Fig. 7.6.4 confirms that the use of dowel bars in traditional JPCPs reduces the ΔIRI, i.e. increases the stability of the ride quality which is in agreement with the results of Byrum (2005). Fig. 7.6.5 shows the stability of the ride quality provided by short slabs, even when no dowel bars are applied. This stability is independent on the use of fibers (Fig. 7.6.5) because it is related with the fact that the slabs are shorter and then the length available for the changes of the slab curvature is less than in a traditional JPCP. But also the reduction of slab length produces thinner cracks under the joints which is fundamental for the aggregate interlock that restricts the movement between adjacent slabs, i.e. the possibilities of changes in the slab curvature are restricted by effective aggregate interlock (Poblete et al, 1987; Rao and Roesler, 2005) even when no dowel bars are applied. In effect, as was presented in the section 7.4.3, the drastic reduction of crack width at joints of short slabs produces a radical increment of the LTE (value ≥ 70 %, being the LTE 70% considered appropriate for the adequate performance of JPCPs).

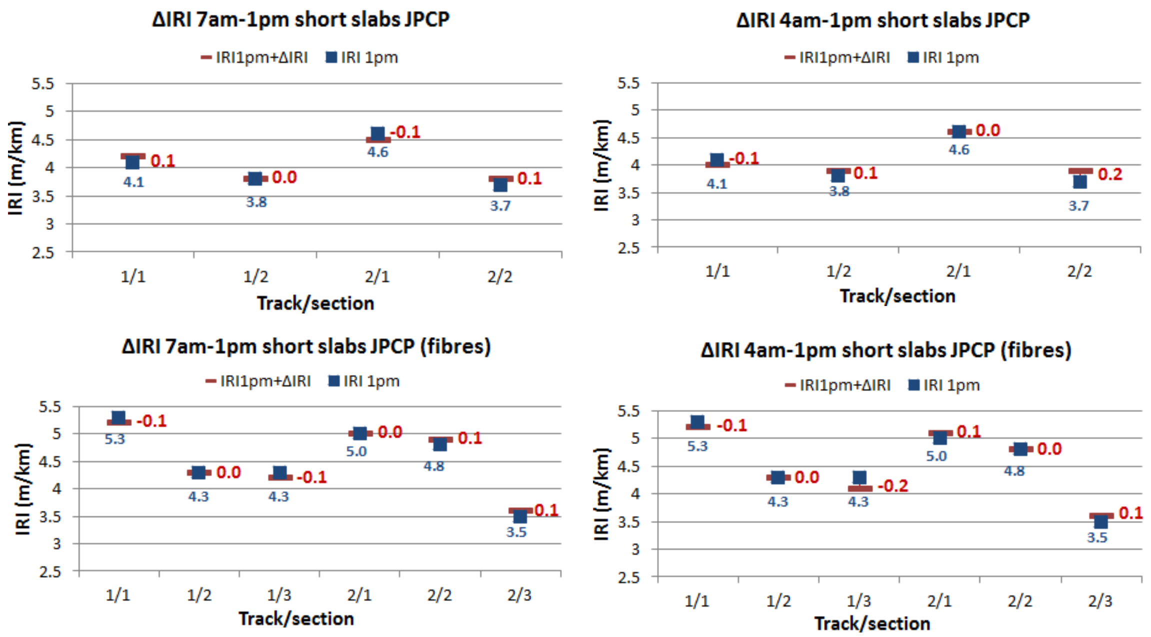

Fig. 7.6.5. Results of $\Delta IRI_{7am\text{-}1pm}$ and $\Delta IRI_{4am\text{-}1pm}$ for short slabs of Route 60-Ch.

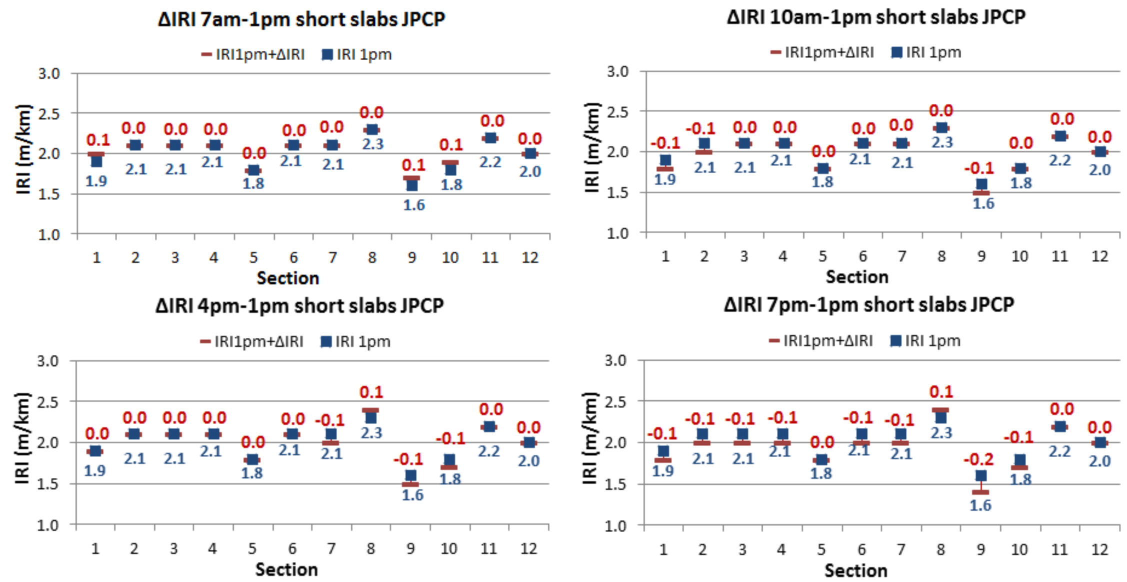

Fig. 7.6.6. Results of ΔIRI short slabs lane 1 Route M-50.

The stability of the ride quality found at the short slabs of the Route 60-Ch are confirmed by the additional measurements made on the Route M-50. The Figs. 7.6.6 and 7.6.7 present the ΔIRI per lane between different times of the day, in particular the $\Delta IRI_{7am\text{-}1pm}$.

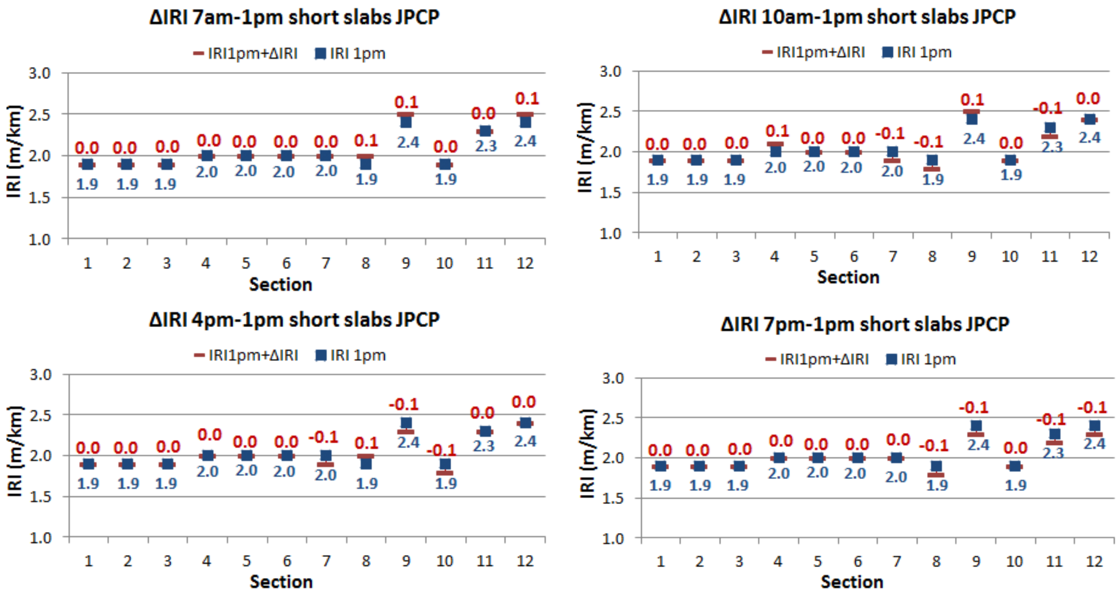

Fig. 7.6.7. Results of ΔIRI short slabs lane 2 Route M-50.

The Figs. 7.6.5 to 7.6.7 show the stability of the ride quality provided by short slabs for different levels of IRI, in particular at the order of magnitude of the IRI of construction (Figs. 7.6.6 and 7.6.7) and of the IRI of maintenance (Fig 7.6.5). These higher levels of IRI on the Route 60-Ch are the result of the combination of time of paving, construction techniques and demanding climatic conditions (Covarrubias et al (2012) reports effective built-in temperature differences up to -41°C). The short slabs on the Route 60-Ch are test sections (500 m) and the Route M-50 is a road (13 km). The lessons learned at the test sections were incorporated in new technical specifications (Chilean Highway Agency, 2012) applied for instance in the construction of the Route M-50. The new specifications include indications about the time of paving and optimized construction techniques to reduce the built-in curling (Chilean Highway Agency, 2012).

The changes of IRI during the day at traditional non-dowelled JPCPs affect the road users and produce a challenge for the transportation agencies in terms of the representative measurement of the ride quality of the JPCP when they need to decide about the reception of new JPCPs or maintenance works. This is especially relevant with the increment of the public private partnership, performance based specifications and contracts for level of service, DBFM (Design, Build, Finance and Maintain), different types of roadways concessions, etc, where an IRI-value needs to be defined for the control and payment of the pavement investments. In effect, a traditional non-dowelled JPCP could have an IRI below the ride quality threshold defined as acceptable for the road users but due to the changes of the slab curvature can be out of the threshold in certain periods of the day. For instance, considering the magnitude of IRI (3.5 m/km – 4.0 m/km) and ΔIRI (0.5 m/km) found on a traditional non-dowelled JPCP in the present study, if the result of IRI measurements performed at noon is 3.6 m/km and the threshold for maintenance is 4.0 m/km, the JPCP would not require maintenance intervention. However as the ΔIRI is 0.5 m/km, the users of this JPCP commuting to their daily activities early in the morning (around 7:00 am) would experience an IRI 4.1 m/km, i.e. over the threshold defined as acceptable for maintenance intervention.

On the contrary, the results of ΔIRI obtained on JPCPs with short slabs show the stability of the ride quality perceived by the user at different times of the day. This stability offered by short slabs does not produce the problem for the transportation agency to include the timing of measurements within specifications or another way to find a representative IRI-value for the decision-making process of reception of new JPCPs or decisions about maintenance interventions.

7.7. EFFECTS OF UNCRACKED JOINTS IN THE FUNCTIONAL ANALYSIS

The presence of UnCrJ is particularly relevant for short slabs JPCPs because the postulated benefits of this innovation are valid on the basis that the slabs are effectively shorter.

In the present chapter the ride quality between the traditional JPCPs and the innovative short slabs JPCPs was compared, resulting in a better ride quality of short slabs JPCPs. However, what does occur when UnCrJ are present? This section is focussed on answering that question analysing the factors affecting the ride quality.

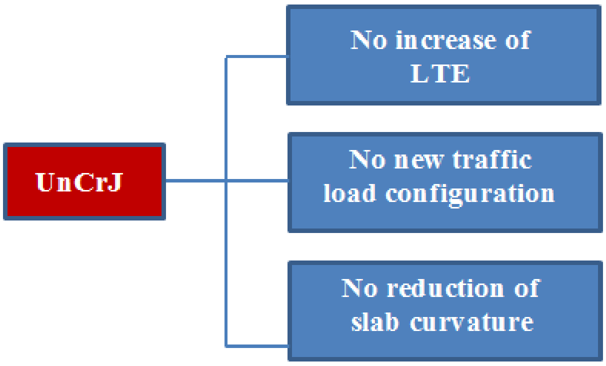

Fig. 7.7.1. Effects of UnCrJ in the factors affecting the ride quality of short slabs

The Fig. 7.7.1 presents the effect of the UnCrJ on the factors affecting the ride quality, in particular on the major contributor of the ride quality deterioration, i.e. the joint faulting (Bustos et al, 2000; Jung et al, 2011). When UnCrJ are present, the postulated increase of LTE in short slabs is not valid anymore because the reduction of the crack width at joints is not produced. In this case, no better JF can be expected in the thinner short slabs.

When UnCrJ are present, short slabs not necessarily can provide a better ride quality due to the slab curvature reduction. Furthermore, the short slabs not necessarily provide better stability of the ride quality. For instance, if 50% of the joints remain uncracked in a short slabs JPCP, the effective slab length is the one of a traditional JPCP, i.e. 4 m instead of the originally designed slab length of 2 m (Fig. 7.7.2). Hence, in this case there is no reduction of slab curvature compared to the traditional JPCP.

Moreover, when UnCrJ are present the postulated new traffic load configuration acting over the designed short slabs is not valid anymore (Fig. 7.7.1) and then the lower transverse cracking level of short slabs is not necessarily real. This is reinforced by the non-existent increase of LTE neither the slab curvature reduction (Fig. 7.7.1). This is even more clear considering that short slabs can have 70 to 100 mm less thickness (Roesler et al., 2012) than traditional JPCPs.

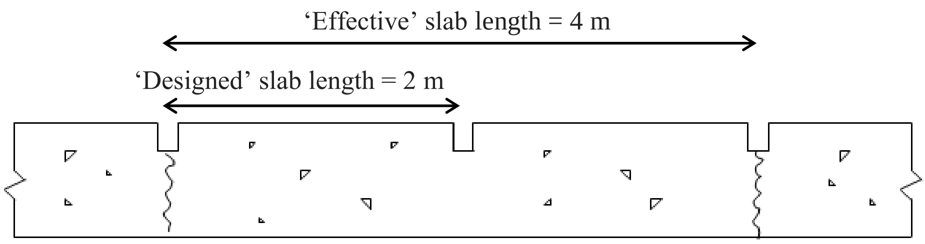

Fig.7.7.2. Example of section of short slabs with uncracked joint.

The use of EESC is common in the thinner short slabs JPCPs. In Chapter 5 the effect of the EESC in the joint activation was analysed. According to that analysis a relative joint depth between 30% to 50% is recommended depending on the specific construction conditions. In other words the ratio between the saw-cut depth and the thickness of the JPCP needs to be in the range of 30% to 50%.

7.8. CONCLUSIONS

As the analysis of short slabs JPCPs has been focussed on the structural performance, before the incorporation of the early-age concrete behaviour in the functional performance, it was necessary to perform the functional analysis of unsealed joints, to study the applicability of deterioration models originally developed for traditional JPCPs and to incorporate complementary methods (as AHP) useful for the evaluation of innovations.
Considering the pavement clients satisfaction, the functional evaluation was directed to the ride quality which is high priority for these clients. And in order to evaluate if the innovative joints configurations represent an improvement (of the traditional practice) that can increase the pavement clients’ satisfaction, the functional analysis compares the traditional practice with the innovative joint configurations. In this way, specific conclusions are obtained for: unsealed joints, joint faulting modelling, ride quality evaluation with AHP, stability of the ride quality of JPCPs, and finally, the effects of the early-age behaviour on the in-service functional performance of JPCPs.

7.8.1. Unsealed joints

Considering the challenge of the joint seals in order to be cost-effective enhancing the JPCP performance, the reported problems that joint seals present in the field, and the comparison with the UnJs performance, the UnJs arise as a cost-effective alternative for JPCPs. In particular, the UnJs are highly recommended in JPCPs where the main functional objective is the accessibility (local streets, country roads, etc.) and industrial floors used for general storage because they are commonly covered by a roof and then protected towards rainwater. Moreover, the UnJs are recommended for the roadways mentioned in section 7.3.5.

7.8.2. Joint faulting modelling

The JF modelling is fundamental in the functional analysis because it is the major contributor to the ride quality deterioration of JPCPs. The available deterioration models of

JF were originally developed for traditional JPCPs. Consequently it was necessary to evaluate their applicability to short slabs.

The model of HDM-4 was developed for traditional JPCPs and then the relevance the model gives to the reduction of slab length of short slabs is not adequate. Indeed, in non-dowelled traditional JPCPs the changes in the slab length do not represent a change as radical in the LTE as when short slabs are being modelled. In effect, a reduction of 50% of the slab length of a traditional JPCP produces a reduction of 50% of the crack width under the joints which results in a radical increase in the LTE by aggregate interlock (being the LTE fundamental in the production of JF). Accordingly, it is recommended to modify the HDM-4 model or to develop a new model that includes the LTE in order to represent better the JF of the non-dowelled short slabs JPCPs. For this, a dedicated study is necessary including comprehensive field measurements of JF at short concrete slabs in different conditions.

7.8.3. Ride quality evaluation with AHP

To develop and calibrate deterioration models for short slabs JPCPs, an extensive database is required. Although no data is available for that purpose, there is empirical evidence for a comparative functional evaluation in an explicit, traceable and well-sustained way with Analytical Hierarchy Process (AHP) that has been used extensively in pavement-related decisions. This provides a formal decision-making framework for decisions that anyway the owners or the agencies need to make (but neglecting the functional performance which is priority for the pavement clients). Such formal decision-making framework improves the transparency of the decisions which is fundamental, especially when public funds are involved as in the case of public agencies.

Although the AHP matrices were based on empirical evidence, sensitivity analyses were performed in order to test if changes in the original preference were produced, or on the contrary, the original decision was confirmed. With the same purpose, extra-demanding sensitivity analyses were performed. Hence, the original preference was confirmed after being intensely tested. Accordingly, the short slabs are the non-dowelled JPCPs that provide the better in-service ride quality.

In order to respect the engineering principles behind the favourable functional performance of short slabs JPCPs, the following factors are important:

- Activations of the joints: so the slabs can be effectively shorter and the crack width smaller
- Thin saw-cutting: to avoid the penetration of coarse incompressible in the UnJs
- Limited fines content in the granular base: in order to avoid pumping and erosion of the base
- Adequate stiffness of the granular base: to contribute to the integral behaviour of short slabs JPCPs

7.8.4. Stability of the ride quality of JPCPs

The results of ΔIRI measured with the Walking Profiler at non-dowelled and dowelled traditional JPCPs are in agreement with the findings of previous investigations. In particular at non-dowelled traditional JPCPs, ΔIRI up to 0.6 m/km was obtained. These changes of IRI during the day affect the road users and produce a challenge for the transportation agencies in terms of the representative measurement of the ride quality of JPCPs when they need to decide about the reception of new pavements or maintenance works.

On the contrary, the user of non-dowelled short slabs experiences basically the same ride quality independent on the time of the day, i.e. the IRI measured at any time of the day is the IRI-value representative for the pavement. This stability of the ride quality does not produce the problem for the transportation agency to deal with different IRI-values for the same pavement. This stability provided by (non-dowelled) short slabs is explained for the reduction of the slab length and the restrictions to the change in slab curvature produced by effective aggregate interlock due to the reduction of the crack width under the joints.

7.8.5. Early-age behaviour in relation to the in-service functional performance of JPCPs

7.8.5.1. Crack width (at joints)

The crack width at joints is a result of the early-age behaviour of the JPCP. In particular the reduction of the crack width at joints in short slabs JPCPs produce a radical increase of the LTE of the non-dowelled JPCP, from insufficiency to what is suitable for a good performance of JPCPs. In terms of functional performance, this produces a fundamental contribution to the reduction of JF observed in short slabs JPCPs. Moreover, as the JF is the major contributor of the JPCP roughness, the crack width (being the product of the early-age concrete behaviour) influences the ride quality provided by the non-dowelled JPCPs, in particular the short slabs JPCPs.

The recommendation to develop a model able to predict the joint faulting in short slabs includes the consideration of the LTE in that modelling. As in non-dowelled JPCPs, there is a direct relation between the LTE and the crack width under the joints, the relation LTE-crack width presented in Chapter 6 for structural purposes, can be useful for functional purposes as well.

Furthermore, the reduction of the crack width (at joints) is a factor contributing to the ride quality stability provided by short slabs JPCPs. Indeed, this reduction of crack width produces an effective aggregate interlock that restricts the changes in slab curvature.

7.8.5.2. Uncracked joints

The presence of UnCrJ depends on the early-age behaviour of the JPCP and it is particularly sensitive for short slabs because the postulated benefits of this innovation are valid on the basis that the slabs are effectively shorter. From the functional point of view the presence of UnCrJ affects the ride quality as follows:

- Joint Faulting: the increasing of LTE in short slabs JPCPs is not valid anymore because the reduction of the crack width at joints is not produced
- Slab curvature: as a reduction of it is not produced, no better ride equality can be expected. Furthermore, the short slabs cannot provide better stability of the ride quality, because there is no slab curvature reduction neither effective aggregate interlock (no reduction of crack width at joints)

- Transverse cracking: it is not only affected by the effects of the UnCrJ in the LTE and slab curvature, but also due to the non-existent new traffic load configurations in a thinner non-dowelled JPCP.

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8. INTEGRAL ECONOMIC ANALYSIS OF JPCPs

8.1. PAVEMENT CLIENTS' INTEGRAL COMPARISON

Until now the study of short slabs concrete pavements has been focused especially on their structural behaviour (Roesler et al, 2012; Covarrubias, 2012; Salsilli et al, 2015). Taking into account this structural performance, a reduction of construction costs up to 25% with respect to traditional JPCPs has been established due to the traffic load configuration, the slab curvature reduction, unsealed joints and absence of dowels bars (Covarrubias 2008; Covarrubias 2012). However, an integral analysis also must include the functional pavement because it is priority for the pavement clients (Haas & Hudson, 1996). In this way, the objective of the present chapter is to make an integral economic comparative analysis including the structural and functional performance of non-dowelled short and traditional slabs. In effect, a new design configuration needs to be compared with the traditional one in order to evaluate if effectively it represents an improvement (Montgomery, 2012).

In general, calibrated deteriorations models are used to evaluate the functional performance of pavements and from there to calculate the life cycle the maintenance costs. However, there is not enough data available for the development and calibration of deterioration models for the innovative short slabs. Still it is possible to make integral costs-benefits analyses in an explicit, transparent and well-sustained way using the empirical evidence available. For that, some considerations need to be made to reduce the number of variables simultaneously involved in the economic comparison. In this way, the present evaluation is made first with the same concrete thickness and then with an equivalent concrete thickness. In both cases the results are represented in costs-benefits graphs.

8.2. COSTS- BENEFITS COMPARATIVE ANALYSES

8.2.1. Analysis with same concrete thickness

The empirical evidence available allows a functional evaluation with the same concrete thickness of traditional and short slabs JPCPs using the Analytic Hierarchy Process, AHP (as presented in Chapter 7). The same slab thickness basically means the same construction costs[7] and then the cost difference is occurring after the JPCP construction. For this costs quantification the differences in the relative number of ESALs that both JPCPs can resist with the same concrete thickness until the same level of cracking (Present Serviceability Index, PSI = 2.5) are take into account. The Fig. 8.2.1 shows this relative proportion based on the results of Roesler et al (2012). The specific number of ESALs can depend of the JPCP application and the particularities of every project. However, the relative proportion of ESALs of Fig. 8.1 is common to the different JPCP applications studied in the present thesis as it was presented in the functional evaluation with AHP (Chapter 7). In fact, Roesler et al (2012) found that short slabs can resist 10 times more ESALs (average) than traditional JPCPs with the same concrete thickness, even when the traditional JPCPs are dowelled.

[7] Considering elastomeric seals in the joints of traditional JPCP (Barrera et al, 2008) and 2 times more joints in short slabs, but without seal (Perez, 2012).

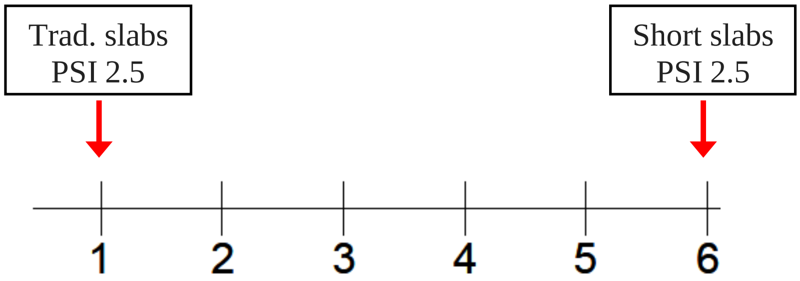

Fig. 8.2.1. Relative number of ESALs resisted by traditional and short slabs JPCPs (based on the results of Roesler et al, 2012).

At the final state of traditional JPCP (PSI=2.5), the short slabs still have a residual structural capacity (Fig. 8.2.2). One way to quantify this is to consider the structural capacity of the concrete overlay necessary to add to the traditional JPCP in its final state (PSI=2.5) to reach the same number of ESALs as short slabs (Fig. 8.2.2). It is important to highlight that independent on the chosen rehabilitation strategy and the timing of its application, always it involves an extra-cost of traditional JPCPs to reach the same number of ESALs as resisted by short slabs. However, as short slabs are patented, it is necessary to include a cost of 1.2 €/m^2 (www.tcpavements.com) for using this technology (section 8.3.1).

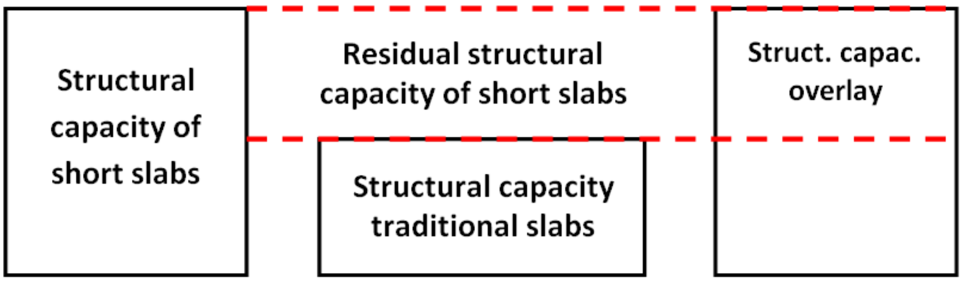

Fig. 8.2.2. Residual structural capacity of short slabs JPCP

As the objective of the rehabilitation is to provide the same structural capacity, the comparison of benefits between both types of JPCPs is based on the functional analysis, in particular the ride quality which also is an overall measure of the pavement´s general condition (Loizos and Plati, 2002, 2008; Thenoux and Gaete, 1995). For that, the scores of the AHP evaluation presented in Chapter 7 are applied.

As the AHP functional evaluation is based on empirical evidence, the slab curvature was not included directly because the evidence available to characterize the interaction with the other deteriorations that affect the ride quality (faulting, spalling and cracking) is not as strong as the evidence available of the interaction between these latter deteriorations. However, in Chapter 7 the slab curvature was considered as an additional criterion for the functional evaluation. In the present section the treatment is similar because the slab curvature not only affects the absolute ride quality but also the changes of it that the users can perceive at different times of the day.

8.2.2. Analysis with equivalent concrete thickness

In the comparisons to resist the same amount of ESALs, the short slabs have been found up to 25% cheaper than traditional JPCPs (Covarrubias, 2008; Covarrubias, 2012) because of the less concrete thickness required. As the structural capacity is the same the comparison is based on the functional performance.

The field evidence (and the engineering principle behind) shows a remarkable functional performance of short slabs even with less concrete thickness. As an example the Fig. 8.2.3 shows the effect of the slab thickness on the joint faulting calculated with the ERES model using the data of short slabs with different thicknesses of a bus lane at Santiago city (Chile) even after 14 million ESALs. According to the ERES model the thinner the slab the higher the Joint Faulting, JF (ERES, 1996). However, in the field no measurable JF was detected in any of the sections with different slab thicknesses (Salsilli et al., 2015). The same situation occurred in short slabs JPCPs in USA, where no measurable JF was detected after 51 million ESALs (Roesler et al, 2012).

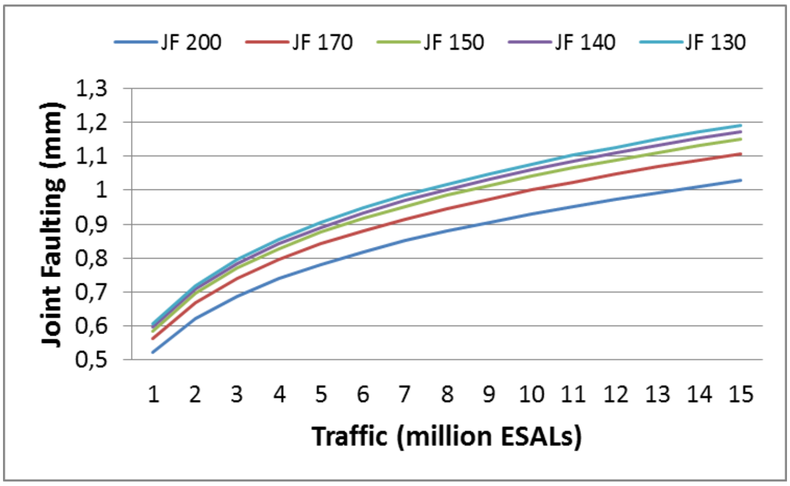

Fig. 8.2.3. Modelling results of JF for different concrete thicknesses of short slabs

The reduction of 50% of the crack width under the joints of short slabs produces a radical increase of the Load Transfer Efficiency (LTE) by aggregate interlock (Pradena and Houben, 2014). The LTE is fundamental in the development of JF (Ioannides et al., 1990; SHRP 2, 2011; NCHRP, 2012) and the JF yields the major contribution to the deterioration of the ride quality (Bustos et al, 2000; Jung et al, 2011). The deterioration models developed for traditional JPCPs are not able to represent the relevance of this effect resulting in the excellent functional performance of real-world short slabs even with less thickness than the traditional ones.

Anyway, using a conservative approach it is considered in the evaluation that the functional and structural performance of both alternatives is the same.

However, for the reasons explained in section 8.2.1 the slab curvature is included as an additional criterion for the functional evaluation because it not only affects the absolute ride quality but also the changes of ride quality that the users can perceive at different times of the day.

8.3. RESULTS OF THE COMPARATIVE ANALYSES

8.3.1. Same concrete thickness

As presented in section 8.2.1, a way to quantify the residual structural capacity of short slabs JPCPs is by means of the rehabilitation costs required for the traditional JPCP in order to resist the same traffic demands that short slabs (Fig. 8.3.1).

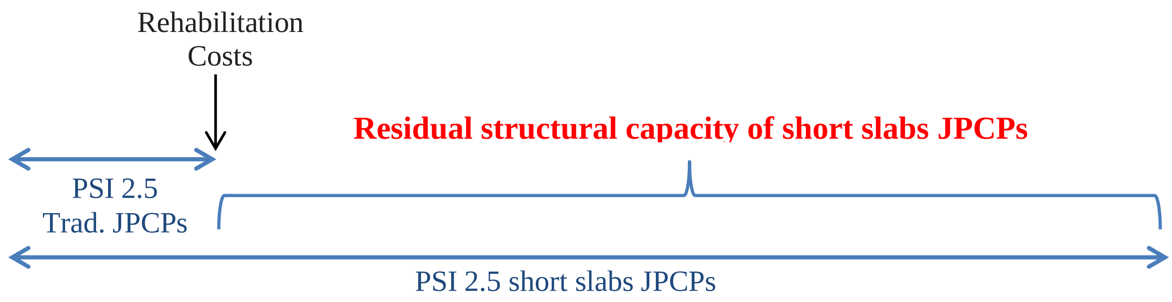

Fig. 8.3.1. Quantification of the residual structural capacity of short slabs JPCPs

The specific rehabilitation strategy and the timing of its application will depend of the particular JPCP application and the characteristics of every project. Although in the present section a numerical example is presented, it is important to highlight that, independent of the chosen rehabilitation strategy (and the timing of its application), always it involves an extra-cost of the traditional JPCP in order to resist the same traffic demands that short slabs (Fig. 8.3.1). Again, in the present section just an example is presented, but the principle behind (extra-cost of traditional JPCP) remains for different applications.

For illustration purposes, a JPCP section of 1 km length, 7 m wide, and a typical rehabilitation strategy and timing of application have been chosen. In fact, a diamond grinding and a bonded overlay JPCP are applied at year 5 (Table 8.3.1). The cost at year 5 (i.e. 2021) of the diamond grinding (Barrera et al, 2003) is projected applying an inflation of 3% (www.ec.europa.eu/smart-regulation). Assuming the same inflation, the cost of the bonded overlay JPCP (MINVU, 2009) is projected at year 5 as well. The Table 8.3.1 presents these costs which in total are €229,600.

Table 8.3.1. Typical rehabilitation strategy and its costs (at year 5)

Rehabilitation works	Cost (€/km)
Diamond grinding	44,250
Bonded Overlay JPCP (120 mm)	185,350

As the rehabilitation works are applied at year 5, their costs need to be discounted until year 0 (i.e. 2016) in order to be compared with the costs of construction and for using the technology short slabs. Assuming a discount rate of 7% (www.ec.europa.eu/smart-regulation) the €229,600 at year 5 is equivalent to €163,702 at year 0.

For the JPCP section analysed, the cost for using the technology short slabs is €8400, and the estimated construction cost is €270,000 (Covarrubias, 2008), i.e. € 342,000 at year 0. Therefore, the addition of these costs at year 0 is the total cost of the short slabs JPCPs

(€350,400) which represent 70% of the costs for traditional JPCPs (€505,700) given by the construction costs and the rehabilitation works at year 0 (see Fig. 8.3.3).

Regarding to the slab curvature, short slabs can have even ¼ of the slab length of traditional JPCPs. Using a conservative approach, a reduction of only 50% of the traditional slab length has been considered, i.e. 50% of the slab curvature (Fig. 8.3.2).

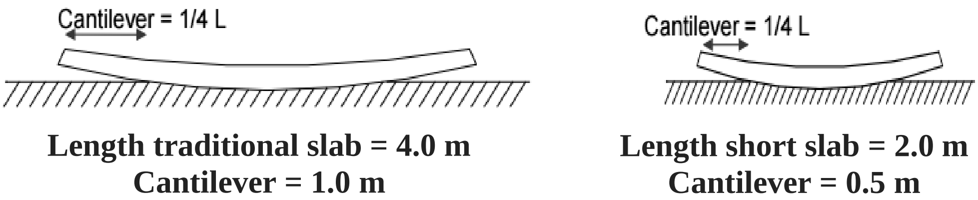

Fig. 8.3.2. Representation of the slab curvature reduction of short slabs.

The Fig. 8.3.3 represents the integral comparison between traditional and short slabs JPCPs for the same concrete thickness. The surface irregularity is expressed in terms of the AHP score (Chapter 7).

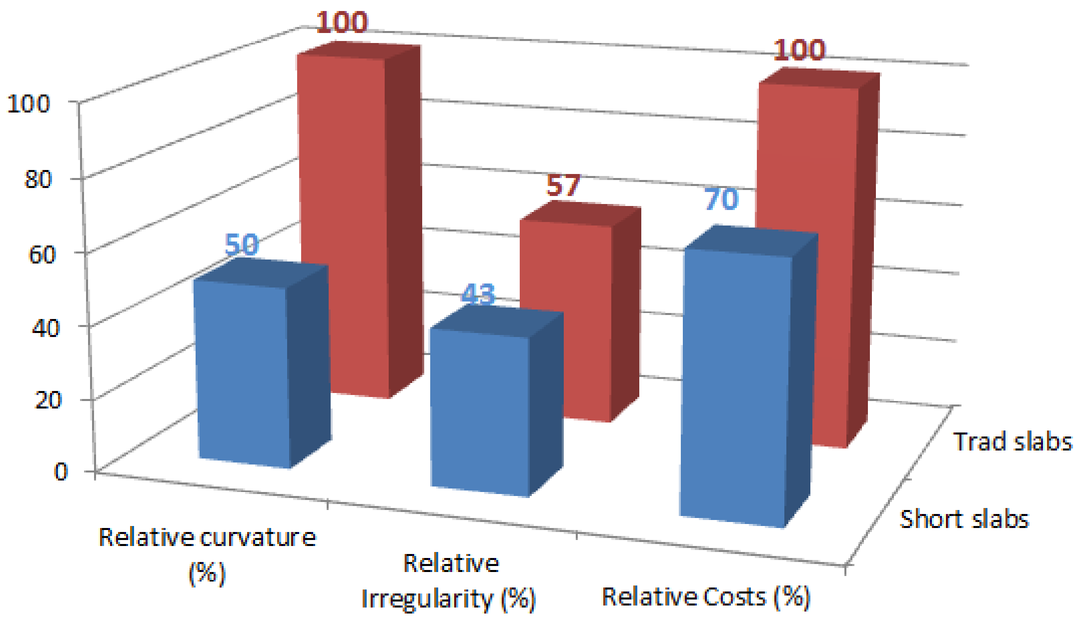

Fig. 8.3.3. Relative comparison slab curvature – surface irregularity – costs for traditional and short slabs for same concrete thickness

8.3.2. Equivalent concrete thickness

Using a conservative approach the construction costs of short slabs are considered 20% less than the ones of traditional JPCPs. Regarding to the slab curvature, short slabs can have even ¼ of the slab length of traditional JPCPs. Again using a conservative approach, a reduction of only 50% of the traditional slab length has been considered, i.e. 50% of the slab curvature.

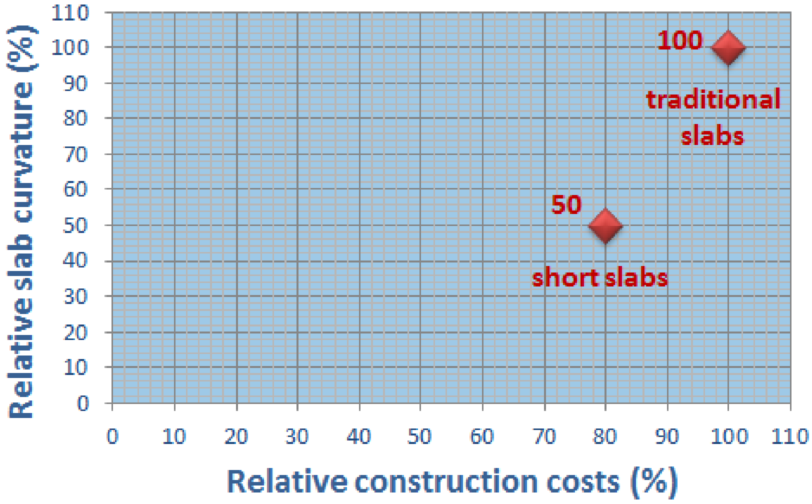

Fig. 8.3.3. Relative comparison slab curvature – construction costs for traditional and short slabs for equivalent concrete thickness

8.4. EFFECT OF UNCRACKED JOINTS IN THE ECONOMIC COMPARISON

The benefits of short slabs are strictly related with the fact that the slabs are effectively shorter than the traditional ones. Otherwise there is not a new traffic load configuration, not a reduction of the slab curvature or crack width at joints (that produce a drastic increase of LTE by aggregate interlock). If the cracks at joints are not produced, the benefits of short slabs presented in this chapter are not valid.

To guarantee that the cracks under the joints are produced, the Relative Joint Depth (RJD) needs to be in the range of 30% to 50%, according to the analyses presented in Chapter 5.

8.5. CONCLUSIONS

An integral costs-benefits analysis of non-dowelled short and traditional slabs has been performed. The economic comparison has included not only the structural performance but also the functional one which is directly related with the pavement clients.

The evaluation has been made using the empirical evidence available and a conservative approach of the benefits of short slabs. Still, short slabs are more cost-effective than traditional JPCPs when the costs and benefits (less detriment) are compared with the same concrete thickness or equivalent thickness. In the comparison for the same concrete thickness, independent on the specific rehabilitation strategy applied and the timing of its application, it always involves an extra-cost of traditional JPCPs with respect to short slabs. And in the case of equivalent thickness, even when short slabs are showing a remarkable functional performance, the same performance than traditional JPCPs was considered. Then, the difference is in the effects of the slab curvature on the ride quality.

As part of the conservative approach applied, despite the favourable functional performance of short slabs, only private costs have been included in the evaluation. Even when social costs, such as the ones of the users, are generally associated to the functional performance of pavements, nowadays there are not yet calibrated relations for the innovative short slabs. And the evaluation of the present section has been made using empirical evidence available.

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9. CONCLUSIONS AND RECOMMENDATIONS

9.1. CONCLUSIONS

9.1.1. Early-age concrete behaviour in JPCPs

The models available for predicting the joint opening in JPCPs ignore the interaction (in time and space) of the group of joints. The system approach applied in this thesis is different. In fact, in the system approach the cracking process of JPCPs is not only time-dependent but also space-dependent, i.e. it takes into account the interaction of the group of joints. Furthermore, the system approach also includes the viscoelastic concrete behaviour (relaxation) and the effect of geometry of the pavement (thickness, slab length), the time of construction of the JPCP and the saw-cutting method, between other variables. In this way it is possible to predict a particular crack width (for instance the one of the 1^{st} series of cracks) or the presence of uncracked joints in the JPCP in order to evaluate the effects of the early-age concrete behaviour on the in-service performance of JPCPs.

In order to obtain realistic values of the relevant results of the early-age concrete behaviour, the model of the cracking process of JPCPs was calibrated with a procedure that considers the intended uses of the model (mainly the $AvCW1^{st}$ for the link with the LTE by aggregate interlock) and the necessity of being practical and useful for pavement clients as public agencies related with different JPCP applications (as urban, interurban, airports). In fact, with this practical procedure it is possible to adjust the calibration constants of the model to the real-world $AvCW1^{st}$. This allows adapting the model to the particular characteristics of the pavements under study, for instance the JPCPs of specific geographical regions and/or particular JPCP applications.

10 test sections were considered to compare the modelling results with the real-world behaviour of the JPCPs. 2 test sections of traditional JPCPs (1 in Belgium and 1 in Chile) were used in the adaptation phase and 8 test sections located in Chile were considered in the post-calibration analyses. Between these 8 test sections, 3 correspond to traditional JPCPs and 5 to short slabs JPCPs. Besides, 3 of these 8 test sections are urban JPCPs, 2 interurban JPCPs, 2 test sections are on an industrial complex and 1 on an airport apron.

Relaxation of the concrete appears to have a huge influence on the development of the cracking process in a JPCP, i.e. the percentage of uncracked joints and the crack widths. A time-dependent formulation of the relaxation has been adopted, originally with default values 1 for the calibration constants ‘a’ and ‘b’. In the calibration process these constants were adjusted to the values ‘a = 1.1’ and ‘b = 2.3’ in order to improve the agreement between the modelled and the real-world $AvCW1^{st}$. Actually this agreement was improved from 0.7 AGREE (average) before the calibration to 1.0 AGREE (average) after the calibration.

The post-calibration analyses were performed comparing the real-world $AvCW1^{st}$ of the 8 test sections previously mentioned with the modelled $AvCW1^{st}$. The model was able to predict correctly the $AvCW1^{st}$ in the 4 main JPCP applications, i.e. interurban, urban, industrial and airports. Furthermore, the model was able to predict correctly the $AvCW1^{st}$ in the cases of restricted length of short urban streets, industrial floors with irregular layout, and an airport apron with restricted length caused by the necessity to continue the airport operations during construction.

Additional post-calibration calculations were made to analyse the influence of the Relative Joint Depth (RJD) and the saw-cutting method (more details in sections 9.2.1.6 and 9.2.2.1)

9.1.2. Structural analysis: the relation LTE- AvCW1st

The present thesis has been focused on the direct cause of the LTE by aggregate interlock (i.e. the crack width) instead of indirect ones. In this way, it is possible to incorporate in mechanistic-empirical design methods for non-dowelled JPCPs the direct cause of the LTE by aggregate interlock. This is especially important for the innovative (non-dowelled) short slabs JPCPs where the LTE relies on the aggregate interlock.

Due to the small crack widths, the non-dowelled short slabs JPCPs are able to provide adequate LTE (≥ 70%) even without dowels bars. Indeed, in this case the LTE relies on aggregate interlock and the results of the Faultimeter measurements have confirmed this interlocking for crack widths ≤ 1.2 mm, which are the typical crack width values of short slabs JPCPs.

9.1.3. Functional analysis

As the analysis of short slabs JPCPs has been focussed on the structural performance, before the incorporation of the early-age concrete behaviour in the functional performance, it was necessary to perform the functional analysis of unsealed joints, to study the applicability of deteriorations models developed originally for traditional JPCPs incorporating as well evaluation methods such as the Analytic Hierarchy Process (AHP) which is useful for the evaluation of innovations. Considering the satisfaction of the pavement clients, the functional evaluation was directed to the ride quality which is a high priority for these clients.

The specific conclusions of the functional analysis are as follows:

9.1.3.1. Joint faulting modelling

The JF modelling is fundamental in the functional analysis because it is the major contributor to the ride quality deterioration of JPCPs. The model of HDM-4 was developed for traditional JPCPs and is not adequate for short slabs JPCPs. Indeed, in non-dowelled traditional JPCPs the changes in the slab length do not represent a change as radical in the LTE as when short slabs are being modelled. In effect, a reduction of 50% of the slab length of a traditional JPCP produces a reduction of 50% of the crack width under the joints of the short slabs JPCPs which results in a radical increase of the LTE by aggregate interlock (the LTE being fundamental in the production of JF). Accordingly, it is necessary to modify the HDM-4 model or to develop a new model that includes the LTE in order to better represent the JF of the non-dowelled short slabs (see also section 9.2.3.3).

9.1.3.2. Ride quality evaluation with AHP

Nowadays not enough data are available for developing and calibrating deterioration models for short slabs JPCPs, but there is empirical evidence for a comparative functional evaluation in an explicit, traceable and well-sustained way with the Analytical Hierarchy Process (AHP) which provides a formal decision-making framework for decisions that the owners or the agencies need to make. Although the AHP matrices were based on empirical evidence, sensitivity analyses were performed in order to test if changes in the original preference were produced, or on the contrary, the original decision was confirmed. The sensitivity analyses confirmed the original preference, i.e. the short slabs are the non-dowelled JPCPs that provide the better in-service ride quality.

9.1.3.3. Variability of the ride quality in JPCPs

The daily variation of the air temperature results in a variation of the temperature and the temperature gradient of the JPCP slab, leading to a variation of the roughness IRI (International Roughness Index). The highest IRI-values are found early in the morning due to the curling of the individual slabs. The results of ΔIRI (IRI in the morning minus IRI in the afternoon) measured with the Walking Profiler at non-dowelled and dowelled traditional JPCPs are in agreement with the findings of previous investigations. In particular at non-dowelled traditional JPCPs, ΔIRI up to 0.6 m/km was obtained. These changes of IRI during the day affect the road users and produce a challenge for the transportation agencies in terms of the representative measurement of the ride quality of JPCPs when they need to decide about rehabilitation or maintenance works.

On the contrary, the user of a non-dowelled short slabs JPCP experiences basically the same ride quality independent of the time of the day, i.e. the IRI measured at any time of the day is the IRI-value representative for the pavement. This stability of the ride quality does not produce the problem for the transportation agency to deal with different IRI-values for the same pavement. This stability provided by (non-dowelled) short slabs can be explained by the reduction of the slab length and the restrictions to the changes of slab curvature produced by effective aggregate interlock due to the reduction of the crack width under the joints.

9.1.3.4. Early-age behaviour in relation to the in-service functional performance of JPCPs

Crack width (at joints)

The reduction of the crack width at joints in short slabs JPCPs produces a radical increase of the LTE. In terms of functional performance, this produces a fundamental contribution to the reduction of JF observed in short slabs JPCPs. In fact, the recommendation to develop a model able to predict the joint faulting in short slabs JPCPs includes the consideration of the LTE (see section 9.2.3.3).

In non-dowelled JPCPs there is a direct relation between the LTE and the crack width under the joints. The relations LTE-crack width, presented in Chapter 6 for structural purposes, can be useful for functional purposes as well. For instance, to determine the LTE to be included in the deterioration model to predict the joint faulting in short slabs JPCPs.

Moreover, as the JF is the major contributor of the JPCP roughness, the crack width resulting from the early-age concrete behaviour influences the ride quality provided by the non-dowelled JPCPs, in particular the short slabs JPCPs.

Furthermore, the reduction of crack width (at joints) is a factor contributing to the stability of the ride quality of short slabs JPCPs. Indeed, this reduction of crack width produces an effective aggregate interlock that restricts the changes of slab curvature.

Uncracked joints

The presence of UnCrJ depends on the early-age behaviour of the JPCP and it is particularly relevant for short slabs because the postulated benefits of this innovation are valid on the basis that the slabs are effectively shorter. From the functional point of view the presence of UnCrJ affects the ride quality as follows:

- Joint Faulting: the increase of LTE in short slabs JPCPs is not valid anymore because there is no reduction of the crack width at joints
- Slab curvature: as a reduction of it is not produced, no better ride equality can be expected. Furthermore, the short slabs cannot provide better stability of the ride quality, because there is no slab curvature reduction neither effective aggregate interlock (no reduction of crack width at joints)
- Transverse (slab) cracking: it is not only affected by the effects of the UnCrJ on the LTE and slab curvature, but also by new traffic load configurations on thinner and larger non-dowelled JPCP slabs.

9.1.4. Costs-benefits comparative analysis

An integral costs-benefits analysis of non-dowelled JPCPs with short and traditional slabs was performed. This economic comparison included not only the structural performance but also the functional one which is directly related with the pavement clients. The evaluation was made using the empirical evidence available and a conservative approach of the benefits of short slabs JPCPs. Still, short slabs are more cost-effective than traditional JPCPs when the costs and benefits are compared with the same concrete thickness or equivalent thickness. In the comparison with the same concrete thickness, independent on the specific rehabilitation strategy applied and the timing of its application, it always involves an extra-cost of traditional JPCPs with respect to short slabs. And, in the case of equivalent thickness, even when short slabs are showing a remarkably better functional performance, the same performance than traditional JPCPs was considered. In this way, the difference was in the effects of the slab curvature on the ride quality and the less construction costs of short slabs JPCPs.

9.2. RECOMMENDATIONS

9.2.1. Recommendations for the design of JPCPs

9.2.1.1. Integral design

Although the structural capacity of the pavement has the highest technical priority, it is important that the pavement solution also incorporates the relevant factors for the clients that, at the end, the pavements need to serve. The pavement clients assign high priority to the functional pavement performance. Hence, taking into account the pavement clients' satisfaction it is important to make an integral analysis of the pavements, that considers not only the structural adequacy but also the functional condition of every pavement alternative

9.2.1.2. Slab length

The conclusions presented in section 9.1 (derived from the analyses performed in the thesis) show the advantages of short slabs JPCPs regarding to traditional JPCPs. In fact, short slabs JPCPs present structural and functional benefits resulting from the reduction of the slab length, the new load traffic configuration, the reduction of slab curvature (and the variation of it), but also reduction of the crack width (at joints) which produces a drastic increase of the LTE by aggregate interlock.

However, the structural and functional benefits of short slabs JPCPs only do occur when the slabs are effectively short. Otherwise, there is no reduction of slab length, or new traffic load configuration, neither reduction of slab curvature, or crack width (at joints). Hence, the activation of all the joints is crucial in short slabs JPCPs. For that, it is necessary to take into account the saw-cut method applied and more specifically the RJD as a result of that method (see sections 9.2.1.4 and 9.2.2.1).

9.2.1.3. Incorporation of the early-age concrete behaviour in the design

As short slabs are non-dowelled JPCPs, the LTE is provided by aggregate interlock which depend directly of the crack width (at joints) resulting from the early-age concrete behaviour. Accordingly, the design of (non-dowelled) short slabs JPCPs must include this fundamental and direct relationship. This is also valid for the design of traditional non-dowelled JPCPs.

In order to develop the relationship LTE-AvCW1st, first it is necessary to correctly predict the AvCW1st. In the present research a system approach to model the early-age concrete behaviour in the JPCP has been presented and calibrated. In comparison with the calibration procedure described in this thesis, a simpler procedure focussing on in-service JPCPs (≥ 1 year after their construction) is proposed particularly for public agencies, with the objective to incorporate the direct cause of the LTE by aggregate interlock in their pavement design manuals. This simpler alternative is proposed considering the good behaviour of the calibrated model, the several successful field corroborations already made at early-ages, and the fact that the intended use of the model is the prediction of the AvCW1st to be linked with the in-service JPCP performance. In this way, the measurements of the AvCW1st can be made on the JPCPs representative for the conditions of interest with a simple fissurometer. If the measured results are similar to the model predictions the process stops there. Otherwise, the calibration constants of the relaxation model can be adjusted according to the real-world AvCW1st.

9.2.1.4. LTE associated to the AvCW1st

It is highly recommended to build the non-dowelled JPCPs with high quality coarse aggregates. Indeed, constructing with such kind of aggregates can even provide adequate LTE in traditional non-dowelled JPCPs.

Although in short slabs JPCPs the adequate provision of LTE is already guaranteed by the small crack widths (≤ 1.2 m), the application of high quality coarse aggregates provides even higher values of LTE.

Regarding the relation LTE-AvCW1st, it is highly recommended to develop this relation directly from field measurements or laboratory tests. When this is not possible, the following recommendations can be useful:

- When high quality aggregates are applied, the presented results of South Africa and Chile (Chapter 6) can be used as a reference with the application of a safety factor (that can be calculated with the finite elements program EverFE)..
- If no high quality aggregates are applied, EverFE is useful to generate specific curves LTE-AvCW1st or set of characteristic curves (for a specific geographical region for instance). A more conservative approach is the direct application of the nonlinear aggregate interlock model of EverFE.

9.2.1.5. Unsealed joints

The joint seals need to be cost-effective enhancing the JPCP performance, including all the effects upon the users. This is a big challenge considering the extra costs of sealing (and re-sealing) the joints, the problems with the joint seals effectiveness and the comparison with the adequate performance of unsealed joints. Therefore, the unsealed joints arise as a cost-effective alternative for JPCPs. In particular, the unsealed joints are highly recommended in JPCPs where the main functional objective is the accessibility (local streets, country roads, etc.) and industrial floors of warehouses where the JPCP is covered by a roof and then protected for rainwater. Furthermore, the unsealed joints are recommended for the specific roadways mentioned in Chapter 7 (section 7.3.5).

9.2.1.6. Relative joint depth

The effectiveness of the Relative Joint Depth (RJD) was evaluated taking advantage that the system approach applied in the present research incorporates the RJD as one of the variables affecting the cracking process of JPCPs.

The (post-calibration) analyses confirmed that the RJD is one of the most influential variables that enables to regulate in a practical, simple and economical way the construction process in order to obtain the desired results of AvCW1st and UnCrJ. For the analysed conditions, the RJD must be between 35% and 40% in order to obtain AvCW1st $\leq$ 1.1 [mm] and 0% UnCrJ when the JPCP construction is made in summer (4 pm). With the same purpose, the construction in winter (10 am) requires deeper RJDs (between 50% and 60%) because the temperature increases shortly after the period of construction. On the contrary, in summer, after the JPCP is built the concrete shrinkage goes together with a temperature decrease.

The presence of UnCrJ is particularly relevant in short slabs JPCPs because their postulated benefits depend on the fact that the slabs are effectively shorter. As in short slabs JPCPs the use of Early-Entry Saw-Cutting (EESC) methods is a common practice, specific recommendations regarding the saw-cutting method applied in the construction of JPCPs are given in section 9.2.2.1.

9.2.2. Recommendations for the construction of JPCPs

9.2.2.1. Saw-cutting method

Although construction in summer produces the most unfavourable condition of AvCW1st, it is a typical season of construction of JPCPs. Considering this reality, the practical recommendation is to regulate the depth of the saw-cut in order to obtain AvCW1st $\leq$ 1.1 [mm] (and 0% UnCrJ). This practical recommendation is given considering the influence of the RJD and the possibilities to regulate it in a simple and economical way in the construction process of JPCPs.

Although favourable from the AvCW1st point of view (narrow cracks), the construction in winter requires deeper saw-cuts to prevent a high percentage of UnCrJ. These deeper saw-cuts are not necessarily difficult to perform in practice when thin blades (< 3 mm) using conventional saw-cutting equipment are applied (as it is the case of the unsealed joints).

Especially in short slabs JPCPs, the EESC is a common practice. Although this saw-cut method is very effective to relieve internal tensions in the early-age concrete, it is able to produce only a shallow cut up to 30 [mm]. In this context, one possibility is to apply EESC in combination with the traditional saw-cutting method, i.e. some joints can be sawn with EESC to relieve the tension at very early-age. Afterwards, the rest of the joints can be saw-cut with conventional equipment and a deeper second saw-cut can be applied with conventional equipment in the joints where the EESC was applied. Another successful alternative observed in the field is the modification of the EESC equipment to realize deeper saw-cuts.

9.2.2.2. Unsealed joints

In order to obtain good results with the unsealed joints it is important to respect the design features of it, i.e. thin saw-cutting (< 3 mm) and a limited amount of fines of the granular base (≤ 8% passing 75 µm). In addition, it is recommended to give attention to the timing of application of the curing compound (i.e. after finishing the pavement surface) and to re-apply it at joints after the saw-cutting, especially when JPCPs are constructed at high temperature or in windy conditions. Furthermore, in these conditions it is also recommended to use an evaporation reducer prior to the application of the curing compound in order to prevent early water evaporation from the concrete.

9.2.3. Recommendations for future research

9.2.3.1. Early-age concrete behaviour in JPCPs

In the present thesis a system approach was applied to analyse the influence of the early-age concrete behaviour on the in-service performance of JPCPs. Although the system approach is different from the existing models, still simplifications of the complex cracking process of JPCPs were made (as explained in the thesis) and the model is perfectible. Actually, a calibration level was established according to the intended uses of the model leaving open the option for continuous improvements.

The system approach considers the effects of multiple variables, as the RJD, in the cracking process of JPCPs. Actually, post-calibration analyses were performed showing the importance of performing saw-cuts deep enough to activate the joints. As EESC produces a shallow cut up to 30 [mm] and it is is a common practice in short slabs JPCPs, the recommendation of the present research is similar to previous ones, i.e. before a general adoption of the EESC, further field investigations are necessary in order to evaluate the real potential of the EESC to activate the joints in different conditions, especially the most unfavourable ones. This recommendation is particularly relevant for short slabs JPCPs where the postulated structural and functional benefits depend on the fact that the slabs are effectively short (i.e. all activated joints).

9.2.3.2. Structural design

The relationships LTE-crack width presented in this thesis are valid for natural aggregates. In the case that recycled aggregates are used in the concrete mix, further specific research is required.

Further research to analyse the effects of dynamic loads over the development in time of LTE by aggregate interlock is recommended. Still, the static case provides an adequate indication (more conservative) of the relation LTE-crack width for the incorporation of the early-age concrete behaviour in the design of non-dowelled JPCPs. This is confirmed by the conclusions of the studies, with dynamic loads, in South Africa (and USA) and presented in Chapter 6 (section 6.4.2.5).

9.2.3.3. Functional design

Unsealed joints
It is recommended to continue the follow-up of the projects with unsealed joints presented in this thesis (evaluation with more accumulated ESALs) and to generate new particular experiences with unsealed joints, for instance in airport aprons. Furthermore, it is recommended to develop deterioration models with the objective to predict the effect of spalled joints on the JPCP roughness.

Joint faulting
As mentioned previously, it is recommended to modify the model of HDM-4 or to develop a new model able to represent better the JF of the short slabs JPCPs. For this, a dedicated study is necessary including comprehensive field measurements of JF at short slabs JPCPs (in different conditions) and considering the LTE in the model. The final objective of the model is to serve to the necessities of the pavement clients. Accordingly, the model must be balanced between the adequate prediction of the JF and the necessity to be practical and useful to pavement clients (as transportation agencies). This includes the variables required by the modelling and the calibration process to evaluate short slabs JPCPs at different conditions.

The development of a deterioration model to predict the joint faulting of short slabs JPCPs should be a priority because the joint faulting is the major contributor to the JPCP roughness.

Appendix A

Cracking process in JPCPs for different locations of the 1st series of cracks

A.1. INTRODUCTION

In the system approach of Houben (2008b, 2008c, 2010b) the value of the crack width is not only the result of the material changes but also of the location of the 1^{st} series of cracks, the 2^{nd} ones and so on until the cracking process is completed. In particular, it depends of the location of the 1^{st} series of cracks, how the calculation of the cracking process exactly develops. In the Chapter 3 the case when the 1^{st} series of cracks occur at the location of every 3^{rd} joint was presented as an example. In the present appendix the cases when the 1^{st} series of cracks occur at the location of every 4^{th}, 5^{th}, 6^{th} and 7^{th} joint are presented based on the work of Houben (2008b, 2008c, 2010b). Other cases develop in a similar way and can be found in Houben (2008b, 2008c, 2010b).

A.2. 1^{st} SERIES OF CRACKS AT THE LOCATION OF EVERY 4^{th} JOINT

The Fig. A.2.1 shows that when the 1^{st} series of transverse cracks occur every 4^{th} joint (so at the joints nrs. 1, 5, 9, etc.), the possible 2^{nd} series of cracks then occur by definition at the joints nrs. 3, 7, 11, etc. midway 2 already present cracks of the 1^{st} series. These 2^{nd} series of cracks do occur when the tensile stress σj(t) exceeds the present tensile strength $f_{ctm}(t)$ (Equations 3.2.21 and 3.4.1 in Chapter 3 respectively).

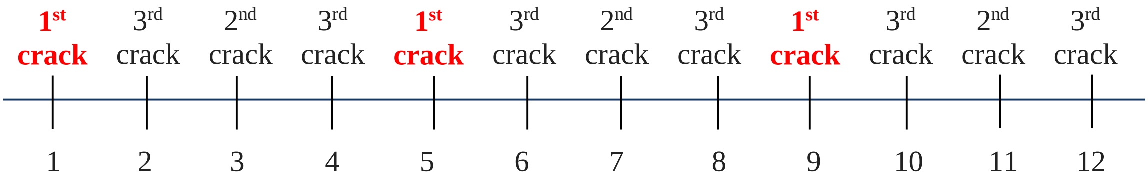

Fig. A.2.1. Development of the cracking process when the 1^{st} series of cracks occur every 4^{th} joint (Houben, 2008b, 2008c, 2010b).

After the occurrence of the 1^{st} series of cracks the spacing between the cracks L_{w1st} is 4 times the slab length and the breathing length L_{a1st} 2 times the slab length.

After the possible occurrence of the 2^{nd} series of cracks the spacing between the cracks L_{w2nd} is 2 times the slab length and the breathing length L_{a2nd} is equal to the slab length.

The (possible) 3^{rd} series of cracks occur by definition at the joints nrs. 2, 4, 6, 8, etc. midway the already present cracks; these 3^{rd} series of cracks again occur when the tensile stress σj(t) exceeds the present tensile strength $f_{ctm}(t)$ (Equations 3.2.21 and 3.4.1 in Chapter 3 respectively).

After the occurrence of the 3^{rd} series of cracks (all the joints then thus are cracked through) it should be checked whether or not a transversal crack occurs in the middle of the individual slabs. This calculation goes according to the procedures explained in Houben (2008a, 2010a) for non-weakened plain concrete pavements.

A.3. 1^{st} SERIES OF CRACKS AT THE LOCATION OF EVERY 5^{th} JOINT

The Fig. A.3.1 shows that when the 1^{st} series of transverse cracks occur every 5^{th} joint (i.e. at the joints nrs. 1, 6, 11, etc.), for reasons of symmetry the possible 2^{nd} series of cracks then occur together in the 2 joints lying in between (nrs. 3 and 4, nrs. 8 and 9, etc.).

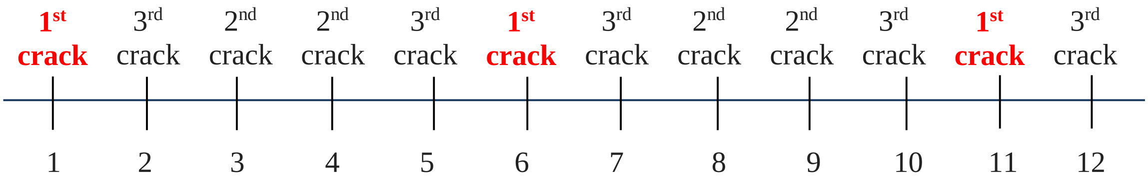

Fig. A.3.1. Development of the cracking process when the 1st series of cracks occur every 5th joint (Houben, 2008b, 2008c, 2010b).

Taking into account the geometry presented in Fig. A.3.1, Houben (2008b, 2008c, 2010b) developed the following equations for the development of the cracking process of the JPCP.

The stress reduction at the location of the 2nd series of cracks due to the occurrence of the 1st series of cracks becomes (Eq. A.3.1):

$$\Delta\sigma_{12zs} = 1.2 * 0.5 * \sigma(t) * \left(1 + \frac{w_{12i}}{(1000 * L_{a12})}\right) \quad \text{(MPa)} \qquad \text{(A.3.1)}$$

The breathing length L_{a12} is now equal to 2.5 times the slab length.

The tensile stress at the location of the 2nd series of cracks due to the occurrence of the 1st series of cracks becomes (Eq. A.3.2):

$$\sigma_{2zs}(t) = \left(4/5\right) * \sigma_{zs}(t) - \Delta\sigma_{12zs} + \left(1/5\right) * \sigma_{zs(t=occurrence1^{st}\,cracks)} \quad \text{(MPa)} \qquad \text{(A.3.2)}$$

The Eq. A.3.3 describes the maximum tensile strain midway between the 1st series of cracks, so not at the location of a 2nd series of cracks but halfway between them.

$$\varepsilon_2(t) = \left(\frac{5}{4}\right) * \left(\frac{\sigma_2(t)}{g * E_{cm}(t)}\right) \qquad \text{(A.3.3)}$$

In the case that the 2nd series of cracks indeed occur, then the initial crack width of every of these cracks can be represented by the Eq. A.3.4.

$$w_{2^{nd}i} = \frac{0.5 * 1000000 * E_{cm}(t) * \varepsilon_2(t)^2}{\gamma * f} \quad \text{(mm)} \qquad \text{(A.3.4)}$$

After the possible occurrence of the 2nd series of cracks the greatest (and thus most critical) distance between 2 cracks is between the joints nrs. 1 and 3 and also between nrs. 4 and 6. Hence, L_{w2nd} is 2 times the slab length and the breathing length L_{a2nd} is equal to the slab length.

The (possible) 3rd series of cracks occur by definition at the joints nrs. 2 and 5 midway the cracks that are already present. This 3rd series of cracks occur if the tensile stress σj(t) exceeds the present tensile strength $f_{ctm}(t)$ (Equations 3.2.21 and 3.4.1 in Chapter 3 respectively).

After the occurrence of the 3rd series of cracks (all the joints then thus are cracked through) it should be checked whether or not a transversal crack occurs in the middle of the individual slabs. This calculation goes according to the procedures explained in Houben (2008a, 2010a) for non-weakened plain concrete pavements.

A.4. 1st SERIES OF CRACKS AT THE LOCATION OF EVERY 6th JOINT

The Fig. A.4.1 shows that when the 1st series of transverse cracks occur every 6th joint (so at the joints nrs. 1 and 7), the possible 2nd series of cracks then occur by definition at the joint nr. 4 midway 2 already present cracks of the 1st series. These 2nd series of cracks do occur when the tensile stress σj(t) exceeds the present tensile strength $f_{ctm}(t)$ (Equations 3.2.21 and 3.4.1 in Chapter 3 respectively).

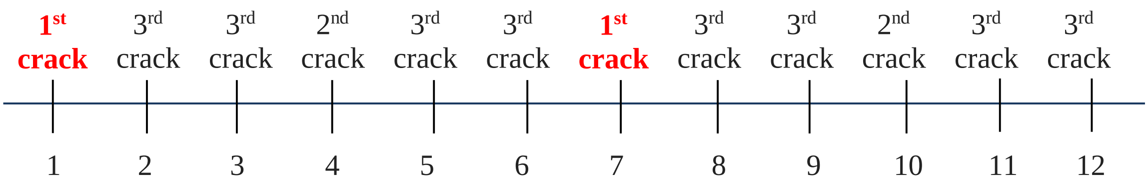

Fig. A.4.1. Development of the cracking process when the 1st series of cracks occur every 6th joint (Houben, 2008b, 2008c, 2010b).

After the occurrence of the 1st series of cracks the spacing between the cracks L_{w1st} is 6 times the slab length and the breathing length L_{a1st} 3 times the slab length.

After the possible occurrence of the 2nd series of cracks the spacing between the cracks L_{w2nd} is 3 times the slab length and the breathing length L_{a2nd} 1.5 times the slab length.

The (possible) 3rd series of cracks occur at the same time in the symmetrically situated joints nrs. 2, 3, 5 and 6 and then the calculations has to be done according to what was presented in Chapter 3, i.e. when the 1st series of cracks occur at the location of every 3rd joint.

A.5. 1st SERIES OF CRACKS AT THE LOCATION OF EVERY 7th JOINT

The Fig. A.5.1 shows that when the 1st series of transverse cracks occur every 7th joint (so at the joints nrs. 1, 8, etc.), for reasons of symmetry the possible 2nd series of cracks then occur together in the symmetrically situated joints nrs. 4 and 5, 11 and 12, etc.

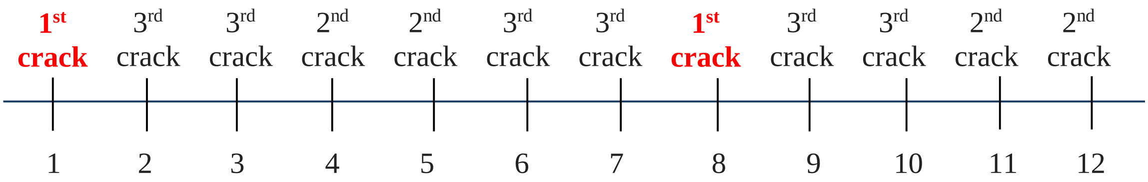

Fig. A.5.1. Development of the cracking process when the 1st series of cracks occur every 7th joint (Houben, 2008b, 2008c, 2010b).

Taking into account the geometry presented in Fig. A.5.1, Houben (2008b, 2008c, 2010b) developed the following equations for the development of the cracking process of the JPCP.

The stress reduction at the location of the 2nd series of cracks due to the occurrence of the 1st series of cracks becomes (Eq. A.5.1):

$$\Delta\sigma_{12zs} = 1.143*0.5*\sigma(t)*\left(1+\frac{w_{12i}}{(1000*L_{a12})}\right) \quad \text{(MPa)} \qquad \text{(A.5.1)}$$

The breathing length L_{a12} is now equal to 3.5 times the slab length.

The tensile stress at the location of the 2nd series of cracks due to the occurrence of the 1st series of cracks becomes (Eq. A.5.2):

$$\sigma_{2zs}(t) = \left(6/7\right)*\sigma_{zs}(t) - \Delta\sigma_{12zs} + \left(1/7\right)*\sigma_{zs(t=ocurrence1^{st}\,cracks)} \quad \text{(MPa)} \qquad \text{(A.5.2)}$$

The Eq. A.5.3 describes the maximum tensile strain midway between the 1st series of cracks, so not at the location of a 2nd series of cracks but halfway between them.

$$\varepsilon_2(t) = \left(\frac{7}{6}\right)*\left(\frac{\sigma_2(t)}{g*E_{cm}(t)}\right) \qquad \text{(A.5.3)}$$

In the case that the 2nd series of cracks indeed occur, then the initial crack width of every of these cracks can be represented by the Eq. A.5.4.

$$w_{2^{nd}i} = \frac{0.5*1000000*E_{cm}(t)*\varepsilon_2(t)^2}{\gamma*f} \quad \text{(mm)} \qquad \text{(A.5.4)}$$

The (possible) 3rd series of cracks occur at the same time in the symmetrically situated joints nrs. 2, 3, 6 and 7 and then the calculations has to be done according to what was presented in Chapter 3, i.e. when the 1st series of cracks occur at the location of every 3rd joint.

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Appendix B

Results of the factorial design simulations

B.1. INTRODUCTION

In Chapter 4 were presented the results of the AvCW1st 1 year after the pavement construction and the percentage of UnCrJ for the case of slab length L = 5 [m] of traditional JPCPs (i.e. short slabs of length 0.5L = 2.5 m). In the present Appendix the results for L = 4.0 [m] and L = 4.5 [m] are presented. As mentioned in Chapter 4 the trends are similar that for L = 5.0 [m].

Similar to what was presented in Section 4.7.3 of the Chapter 4, in this appendix the graphs of the crack width development of the 1st series of cracks for JPCPs built at Summer (4 pm) are presented for L = 4.0 [m], L = 4.5 [m], L =5.0 [m]. In all cases there is not presence of UnCrJ.

B.2. AvCW1st 1 YEAR AFTER THE JPCPs CONSTRUCTION

Table B.2.1. AvCW1st (mm) for traditional JPCPs (L = 4.0 m) and short slabs JPCPs (0.5L = 2.0 m).

Saw-cutting (RJD %)	Friction	Concrete grade	Season and time of construction	
			Summer 4 pm	Winter 10 am
25	1.0	C28/35	3.1/1.6	1.5/0.8
		C35/45	3.9/2.0	1.9/1.0
30	1.0	C28/35	2.7/1.4	1.5/0.8
		C35/45	3.7/1.9	1.8/0.9
35	1.0	C28/35	2.5/1.3	1.5/0.8
		C35/45	2.8/1.4	1.7/0.9

Table B.2.2. AvCW1st (mm) for traditional JPCPs (L = 4.5 m) and short slabs JPCPs (0.5L = 2.25 m).

Saw-cutting (RJD %)	Friction	Concrete grade	Season and time of construction	
			Summer 4 pm	Winter 10 am
25	1.0	C28/35	3.0/1.5	1.5/0.8
		C35/45	3.8/1.9	1.9/1.0
30	1.0	C28/35	2.2/1.1	1.4/0.7
		C35/45	3.6/1.8	1.9/1.0
35	1.0	C28/35	2.1/1.1	1.4/0.7
		C35/45	2.3/1.2	1.9/1.0

B.3. RESULTS OF THE UnCrJ

Table B.3.1. UnCrJ (%) for traditional JPCPs (L = 4.0 m) and short slabs JPCPs (0.5L = 2.0 m).

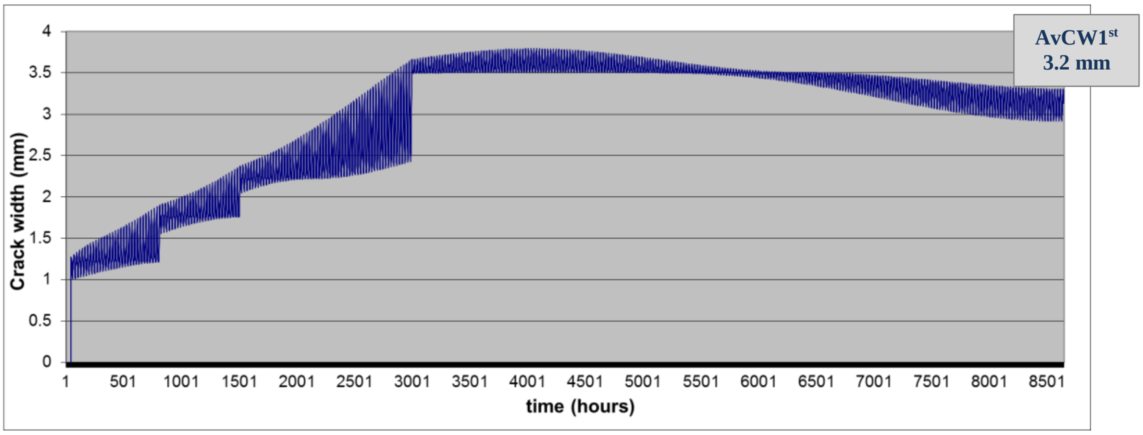
Saw-cutting (RJD %)	Friction	Concrete grade	Season and time of construction	
			Summer 4 pm	Winter 10 am
25	1.0	C28/35	0/48	85/93
		C35/45	0/48	88/93
30	1.0	C28/35	0/0	77/88
		C35/45	0/48	86/92
35	1.0	C28/35	0/0	77/88
		C35/45	0/48	86/92

Table B.3.1. UnCrJ (%) for traditional JPCPs (L = 4.5 m) and short slabs JPCPs (0.5L = 2.25 m).

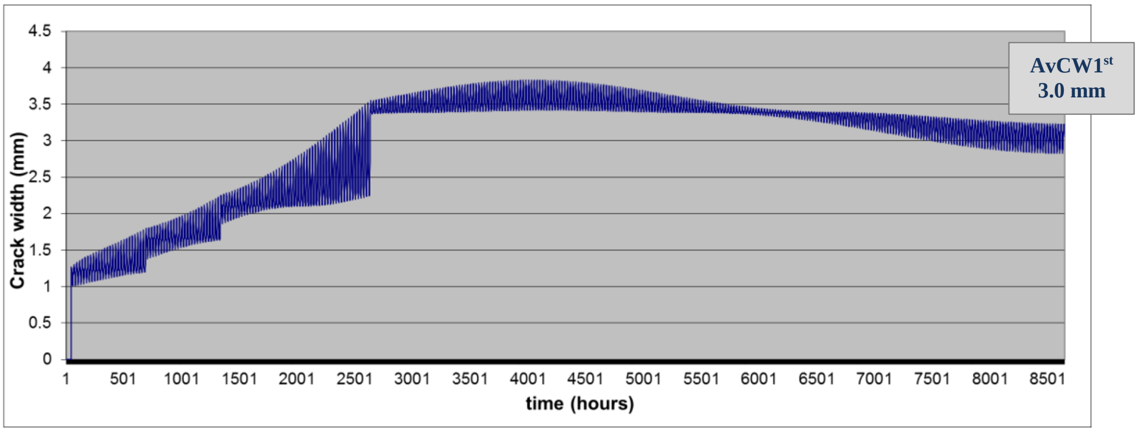
Saw-cutting (RJD %)	Friction	Concrete grade	Season and time of construction	
			Summer 4 pm	Winter 10 am
25	1.0	C28/35	0/48	85/92
		C35/45	0/48	86/93
30	1.0	C28/35	0/48	83/91
		C35/45	0/48	86/93
35	1.0	C28/35	0/0	83/91
		C35/45	0/0	77/88

B.4. CRACK WIDTH DEVELOPMENT OF THE 1st SERIES OF CRACKS UNTIL 1 YEAR AFTER THE JPCP CONSTRUCTION

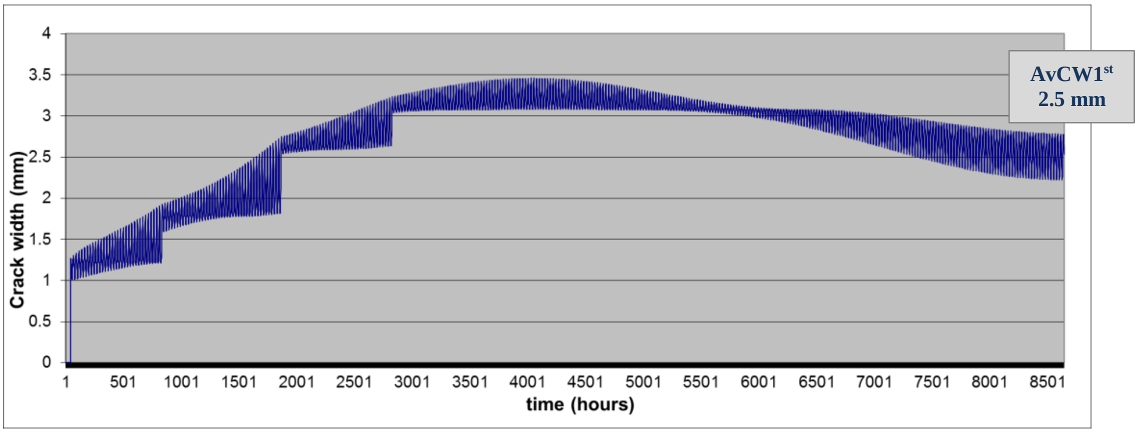
B.4.1. JPCP built in summer (4 pm), concrete grade C28/35 and RJD 25%



(a)



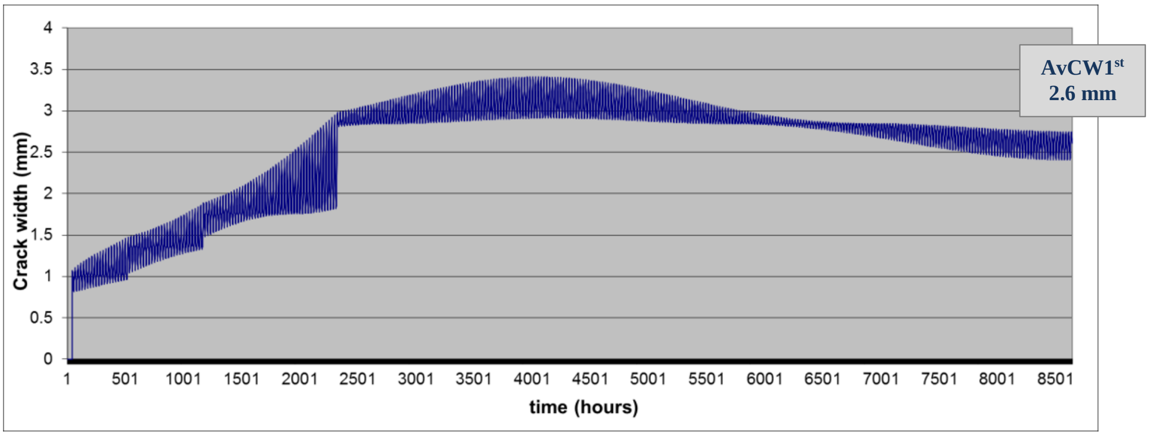
(b)



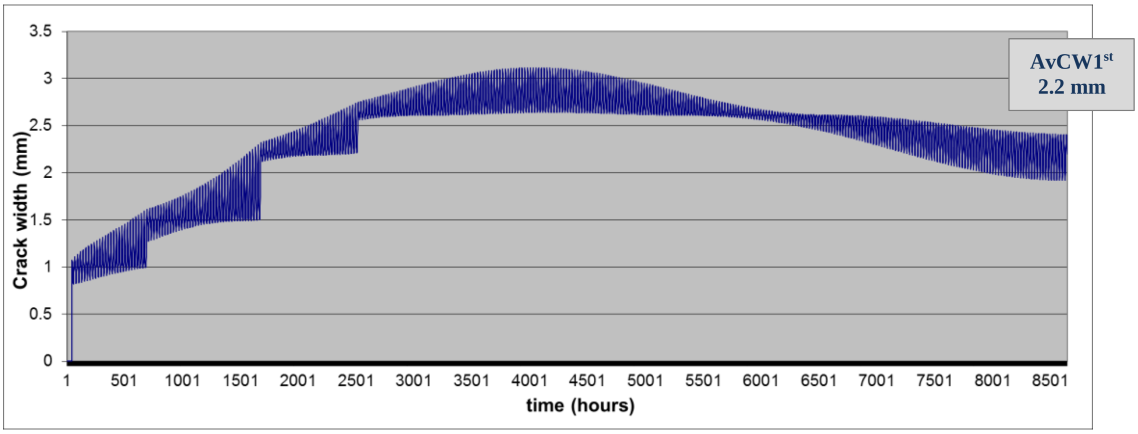
(c)

Fig. B.4.1. Crack width development for JPCPs built in summer (4 pm), RJD 25%, concrete grade C28/35, slab lengths 4 [m] (a), 4.5 [m] (b) and 5 [m] (c)

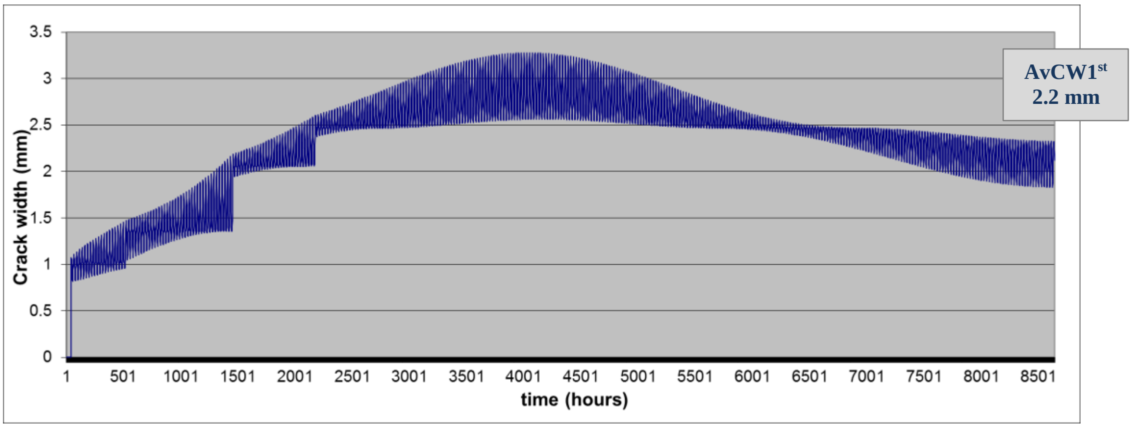
B.4.2. JPCP built in summer (4 pm), concrete grade C28/35 and RJD 30%



(a)



(b)



(c)

Fig. B.4.2. Crack width development for JPCPs built in summer (4 pm), RJD 30%, concrete grade C28/35, slab lengths 4 [m] (a), 4.5 [m] (b) and 5 [m] (c)

B.4.3. JPCP built in summer (4 pm), concrete grade C28/35 and RJD 35%

The cases for slab lengths 4 [m] and 5 [m] were presented in Section 4.7.3 of Chapter 4. The Fig. B.4.3 presents the case for slab length 4.5 [m].

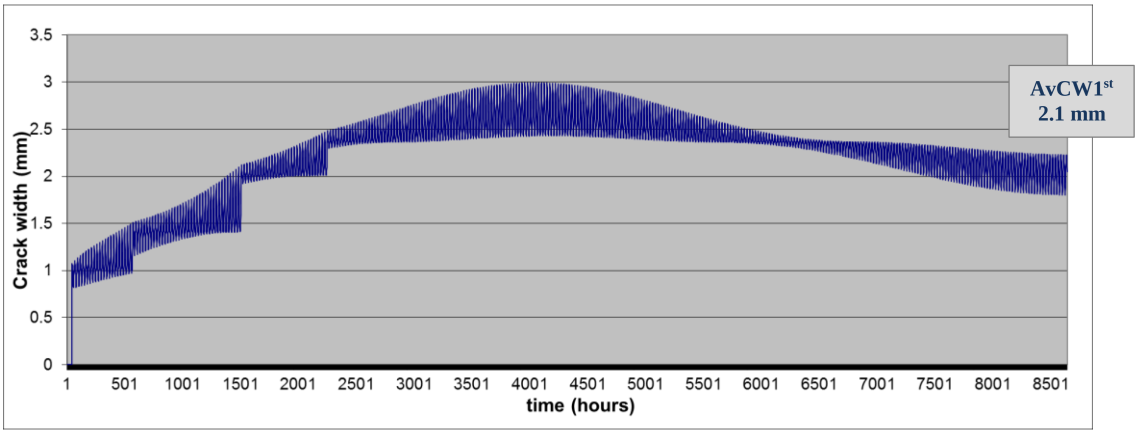
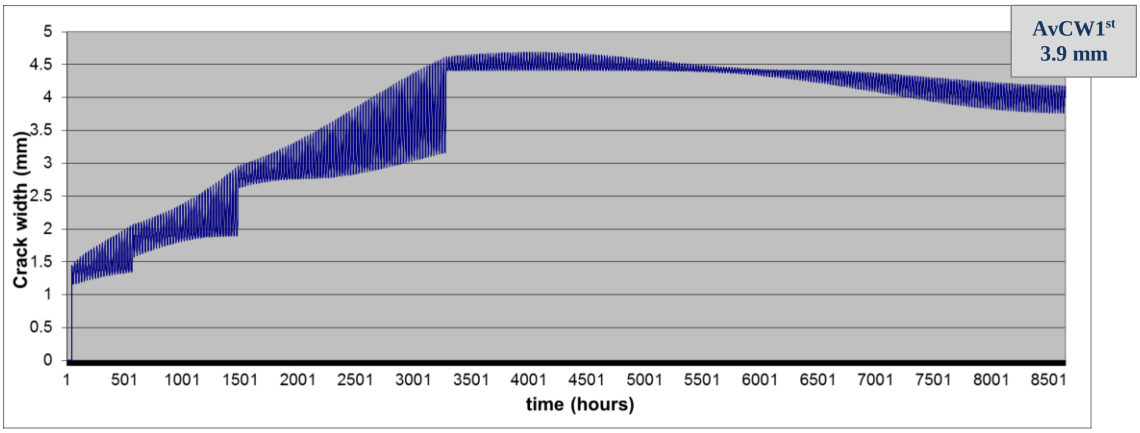
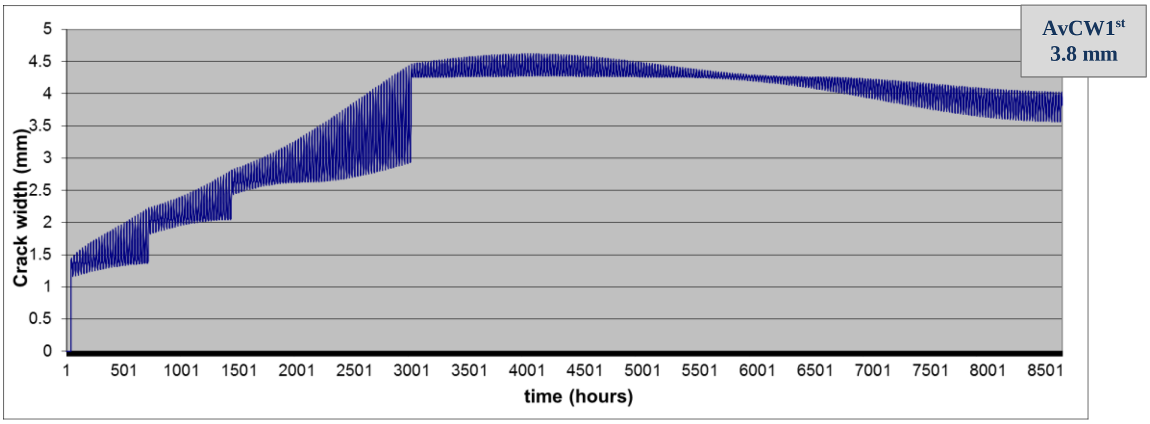


Fig. B.4.3. Crack width development for JPCPs built in summer (4 pm), RJD 35%, concrete grade C28/35, slab length 4.5 [m]

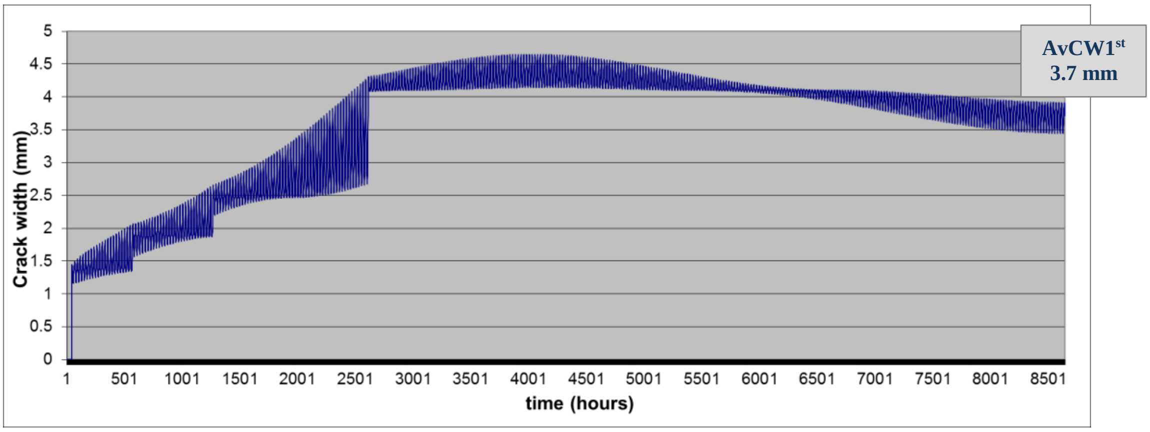
B.4.4. JPCP built in summer (4 pm), concrete grade C35/45 and RJD 25%



(a)



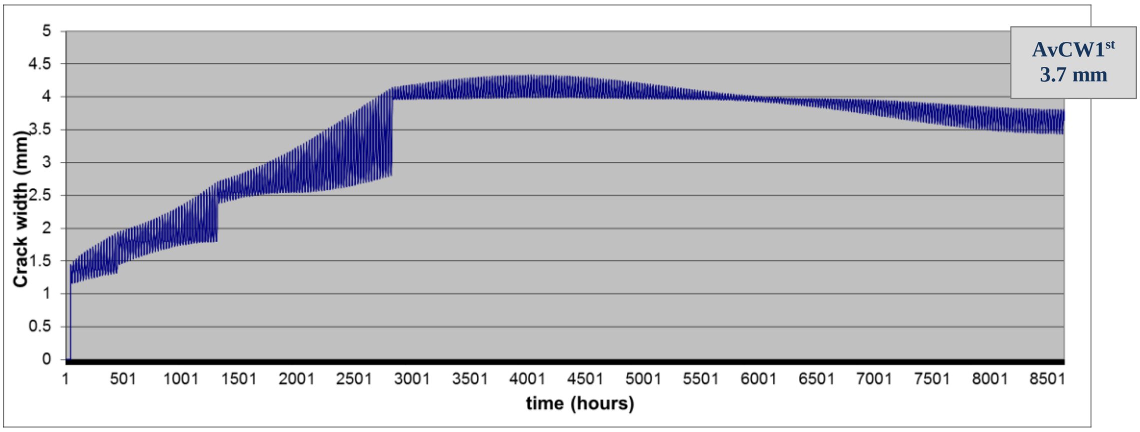
(b)



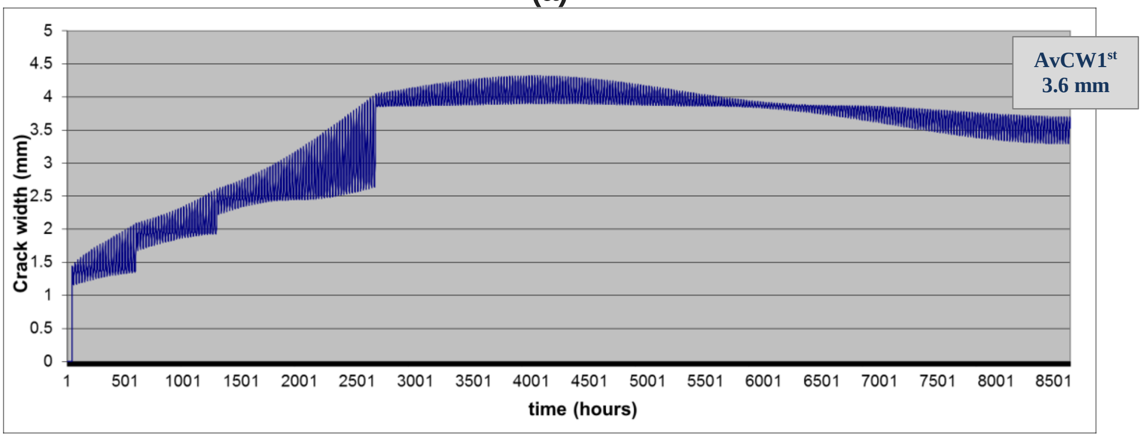
(c)

Fig. B.4.4. Crack width development for JPCPs built in summer (4 pm), RJD 25%, concrete grade C35/45, slab lengths 4 [m] (a), 4.5 [m] (b) and 5 [m] (c)

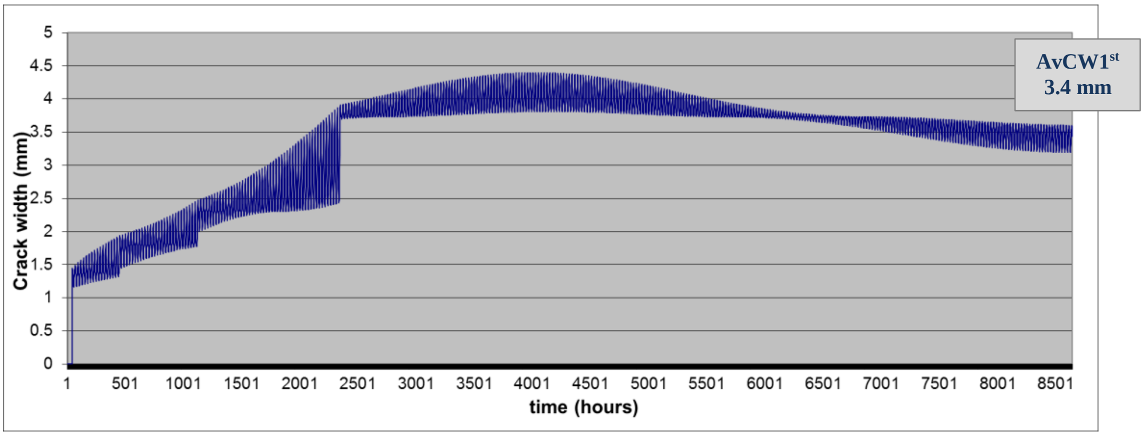
B.4.5. JPCP built in summer (4 pm), concrete grade C35/45 and RJD 30%



(a)



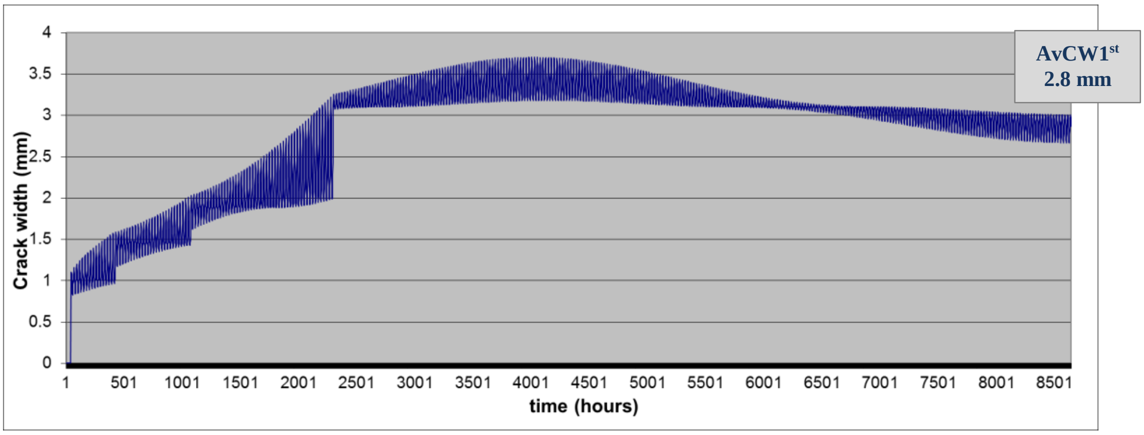
(b)



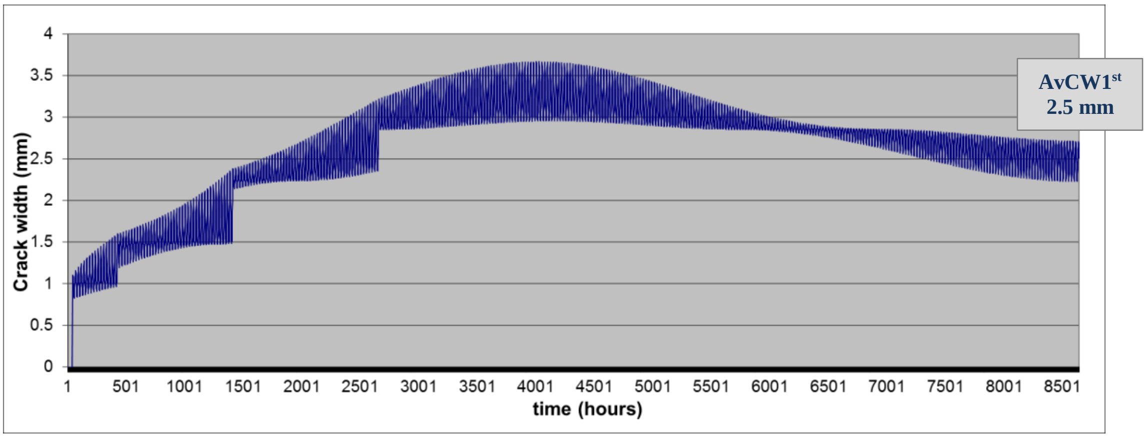
(c)

Fig. B.4.5. Crack width development for JPCPs built in summer (4 pm), RJD 30%, concrete grade C35/45, slab lengths 4 [m] (a), 4.5 [m] (b) and 5 [m] (c)

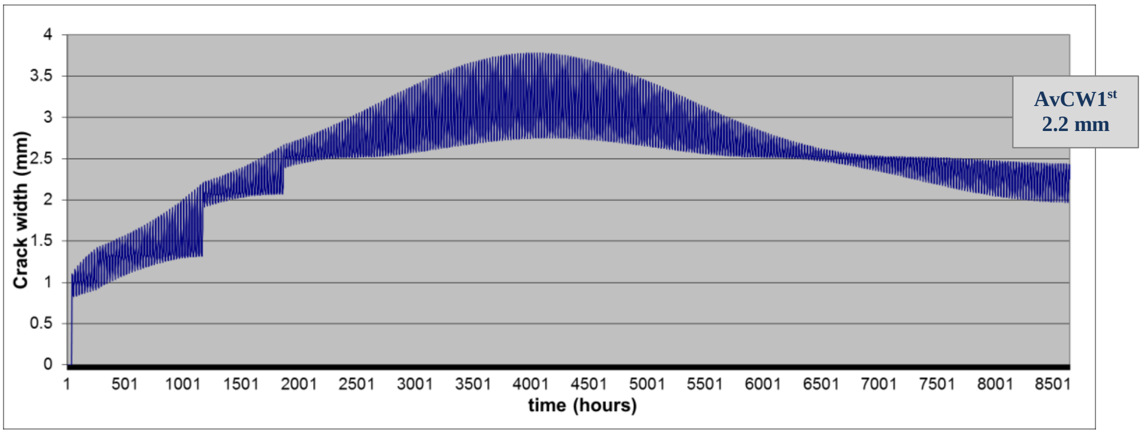
B.4.6. JPCP built in summer (4 pm), concrete grade C35/45 and RJD 35%



(a)



(b)



(c)

Fig. B.4.6. Crack width development for JPCPs built in summer (4 pm), RJD 35%, concrete grade C35/45, slab lengths 4 [m] (a), 4.5 [m] (b) and 5 [m] (c)

B.5. CONCLUSION

The results presented in this appendix confirmed the trends of Chapter 4 and the necessity of the calibration process due to the high values of AvCW1st 1 year after the JPCP construction (> 3 mm) and the fact that smaller AvCW1st are obtained for longer slabs.

Appendix C

Methodology of the
Analytic Hierarchy Process (AHP)

C.1. INTRODUCTION

The analyses of the ride quality presented in Chapter 7 were made using the Analytic Hierarchy Process (AHP). The present appendix includes the mathematical methodology of AHP based on the work of Saaty (1980) as described by the notes of Mocceni (http://www.dii.unisi.it/~mocenni/Note_AHP.pdf).

The AHP considers a set of evaluation criteria, and a set of alternative options among which the best decision is to be made. The AHP generates a weight for each evaluation criterion. The higher the weight, the more important the corresponding criterion. Next, for a fixed criterion, the AHP assigns a score to each option according to the decision maker's pairwise comparisons of the options based on that criterion. The higher the score, the better the performance of the option with respect to the considered criterion. Finally, the AHP combines the criteria weights and the options scores, thus determining a global score for each option, and a consequent ranking. The global score for a given option is a weighted sum of the scores it obtained with respect to all the criteria.

The AHP can be implemented in three simple consecutive steps:

1) Computing the vector of criteria weights.
2) Computing the matrix of option scores.
3) Ranking the options.

Each step will be described in detail in the following. It is assumed that m evaluation criteria are considered, and n options are to be evaluated.

C.2. COMPUTING THE VECTOR OF CRITERIA WEIGHTS

In order to compute the weights for the different criteria, the AHP starts creating a pairwise comparison matrix A. The matrix A is a $m \times m$ real matrix, where m is the number of evaluation criteria considered. Each entry a_{jk} of the matrix A represents the importance of the jth criterion relative to the kth criterion. If $a_{jk} > 1$, then the jth criterion is more important than the kth criterion, while if $a_{jk} < 1$, then the jth criterion is less important than the kth criterion. If two criteria have the same importance, then the entry a_{jk} is 1. The entries a_{jk} and a_{kj} satisfy the following constraint:

$$a_{jk} * a_{kj} = 1 \tag{C.2.1}$$

Obviously, $a_{jj} = 1$ for all j. The relative importance between two criteria is measured according to a numerical scale from 1 to 9, as shown in Table C.2.1, where it is assumed that the jth criterion is equally or more important than the kth criterion. The phrases in the "Interpretation" column of Table 1 are only suggestive, and may be used to translate the decision maker's qualitative evaluations of the relative importance between two criteria into numbers (it is also possible to assign intermediate values which do not correspond to a precise interpretation). However, the scale can be also used to represent empirical evidence, i.e. quantitative evaluation (instead of qualitative) as it was presented in Chapter 7 of this thesis.

Table C.2.1. Table of relative scores (Saaty, 1980)

Value of a_{jk}	**Interpretation**
1	*j* and *k* are equally important
3	*j* is slightly more important than *k*
5	*j* is more important than *k*
7	*j* is strongly more important than *k*
9	*j* is absolutely more important than *k*

Once the matrix A is built, it is possible to derive from A the normalized pairwise comparison matrix A_{norm} by making equal to 1 the sum of the entries on each column, i.e. each entry a_{jk} of the matrix A_{norm} is computed as:

$$\bar{a}_{jk} = \frac{a_{jk}}{\sum_{i=1}^{m} a_{lk}} \tag{C.2.2}$$

Finally, the criteria weight vector *w* (that is a *m*-dimensional column vector) is built by averaging the entries on each row of A_{norm}, i.e.:

$$w_j = \frac{\sum_{i=1}^{n} \bar{a}_{jl}}{m} \tag{C.2.3}$$

C.3. COMPUTING THE MATRIX OF OPTION SCORES

The matrix of option scores is a $n \times m$ real matrix S. Each entry s_{ij} of S represents the score of the *i*th option with respect to the *j*th criterion. In order to derive such scores, a pairwise comparison matrix $B^{(j)}$ is first built for each of the *m* criteria, $j=1,...,m$. The matrix $B^{(j)}$ is a $n \times n$ real matrix, where *n* is the number of options evaluated. Each entry $b_{ih}^{(j)}$ of the matrix $B^{(j)}$ represents the evaluation of the *i*th option compared to the *h*th option with respect to the *j*th criterion. If $b_{ih}^{(j)} > 1$, then the *i*th option is better than the *h*th option, while if $b_{ih}^{(j)} < 1$, then the *i*th option is worse than the *h*th option. If two options are evaluated as equivalent with respect to the *j*th criterion, then the entry $b_{ih}^{(j)}$ is 1. The entries $b_{ih}^{(j)}$ and $b_{hi}^{(j)}$ satisfy the following constraint:

$$b_{ih}^{(j)} * b_{hi}^{(j)} = 1 \tag{C.3.1}$$

And $b_{ii}^{(j)} = 1$ for all *i*. In order to translate the pairwise evaluations (qualitative or quantitative) into the AHP scale numbers, the Table C.2.1 may be used.

Second, the AHP applies to each matrix $B^{(j)}$ the same two-step procedure described for the pairwise comparison matrix A, i.e. it divides each entry by the sum of the entries in the same column, and then it averages the entries on each row, thus obtaining the score vectors $s^{(j)}$, $j=1,...,m$. The vector $s^{(j)}$contains the scores of the evaluated options with respect to the *j*th criterion.

Finally, the score matrix S is obtained as:

$$S = [s^{(1)} .... s^{(m)}] \quad \text{(C.3.2)}$$

i.e. the *j*th column of S corresponds to $s^{(j)}$.

C.4. RANKING THE OPTIONS

Once the weight vector *w* and the score matrix S have been computed, the AHP obtains a vector *v* of global scores by multiplying S and *w*, i.e.:

$$v = S * w \quad \text{(C.4.1)}$$

The *i*th entry v_i of *v* represents the global score assigned by the AHP to the *i*th option. As the final step, the option ranking is accomplished by ordering the global scores in decreasing order.

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ABOUT THE AUTHOR

Mauricio Pradena was born on 30 April 1976 in Chile. In 2001, he received his title of Civil Engineer from University of Concepción (Chile). After his graduation he worked as Lecturer in the Department of Topography at the University of Concepción. In 2008 he obtained his Master of Construction degree from Catholic University (Chile) with the first place outstanding. At his master thesis, he analysed the threshold values of the International Roughness Index (IRI) of asphalt concrete pavements in Chile. Since 2008, Mauricio Pradena is Assistant Professor of the Civil Engineering Department of the University of Concepción, Chile. Nowadays, he is a PhD Candidate in the section Pavement Engineering at Delft University of Technology, supervised by Prof.dr.ir. Sandra Erkens and Assoc.Prof.ir. Lambert Houben. His PhD research is about the effects of the early-age concrete behaviour on the in-service performance of jointed plain concrete pavements.

RELEVANT PUBLICATIONS

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Pradena, M.A. & Diaz, M. (2016). Unsealed joints in urban concrete pavements for buses. Municipal Engineer, Proceedings of the Institution of Civil Engineers. A head to print.

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