Study of 76 bridges potentially affected by DEF

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Abstract

With the rapid construction of new roads and motorways in the 1980s and 1990s in France and the use of better concrete with a faster hydration of cement, some parts of bridges are now potentially affected by delayed ettringite formation (DEF).

In this context, a study was performed to identify and prioritize the most impacted bridges by DEF within a family of 76 bridges; noting that no AAR is involved in the pathology of these structures.

From the analyses of bridge files and especially the concrete compositions, the initial theoretical temperatures reached in the concrete during the bridge construction were estimated. This estimation was done for each part of the bridges. Then, with the results of materials analyses and the pathologies observed during the different visual inspections, a classification of all bridges was done.

Meanwhile, concrete swelling calculations were carried out with the software CESAR-LCPC©. The calculations allowed to analyse the deterioration state of a part of an abutment wall and a part of a prestressed concrete slab bridge at the end of the DEF reaction. Calculations allowed to conclude on the risks presented by this bridge.

Finally, preventive and curative measures were given to the owner of the bridges in order to limit their degradations and to repair some of them.

Keywords: *CESAR-LCPC© software; delayed ettringite formation; finite element model*

1. INTRODUCTION

Within the management of its assets, the motorway company APRR is faced with the problem of managing 76 concrete structures located on the A71 motorway and damaged by delayed ettringite formation (DEF) in concrete, with a more or less extent of this latter reaction and without AAR in concrete.

In this context, a general project management assignment was launched to for the follow-up of the A71 motorway structures affected by DEF. This mission also includes the piloting of repair works and possible reinforcement of the structures.

These bridges were built between 1988 and 1989 and most of them were the subject of a special supervision since 2002, when the disease was diagnosed.

They are composed of 48 overpasses (PSDP, i.e. prestressed slab deck), 25 underpasses (PICF, i.e. frame bridge or PRAD, i.e. deck made of precast prestressed beams), and 3 steel and concrete composite viaducts. They were monitored and diagnosed between 2002 and 2016, based on a Cracking Index monitoring and distancemeter measurements, as well as analysis of concrete with a scanning electron microscope (SEM) and residual expansion tests on concrete.

In order to define a maintenance and investment strategy for these bridges, the motorway company APRR carried out its own initial synthesis. Nevertheless, as this first study could not evaluate the real financial needs to ensure the long-term durability of these structures, a second study was realized with the following purposes:

- to refine the summary and the first maintenance scenarios on which APRR works, by integrating in particular the real operating constraints of the A71 motorway, as well as the budgetary constraints in terms of investment and work costs, in order to define the optimal scenario for prioritizing the structures to be treated,

- to conduct a crossover study of all existing input data from the different diagnosis in order to define the additional investigations to be implemented for the coming years,
- to ensure the preparation, awarding and management of works contracts for the first 4 years of the program (from 2018 to 2021) and to set priorities among the works,
- to restart the calculation of the load-bearing capacity of an abutment crossbeam and an end of a slab desk, both belonging to a particularly affected PSDP type structure, this calculation being based on the assumptions made on materials and resulting from existing investigations aimed at defining the behavior of the concrete affected by DEF.

2. METHODS

The first study carried out by the highway company APRR resulted in a table summarizing the condition of each structure as well as an initial prioritization according to several parameters, namely the parts of the bridge, its category, the characteristics of the concrete and the estimated internal heating temperature at construction.

The analysis of this first study revealed some gaps and inconsistencies in the approach, which led to a significant update of the treatment of incomings in order to obtain a reliable ranking.

In the second study, the proposed ranking of the bridges was carried out according to the method detailed in the following parts and based on different factors which are:

- The threshold temperature which could trigger a DEF,
- The parts of bridges concerned,
- The categories to which the bridges belong to,
- The characteristics of the concrete,
- The results of the cracking index measurements,
- The theoretical heating temperatures of the concrete during its setting,
- The results of the SEM tests carried out on some structures,
- The results of the expansion tests,
- The analysis of existing expertise reports on some structures,
- And finally, the analysis of inspection reports supplemented by specific visits on sites.

Given the analysis made relying on the available input data, it turns out that additional investigations would be necessary to remove some doubts about the effective occurrence or not of DEF. Different investigations were recommended as part of this study and they are described in the following parts.

2.1 Structure's members

Given the pathology condition observed on some on the bridges located on the A71 motorway, it was possible to determine the parts of the structure most at risk, associated with the different types of bridges.

For the overpasses, the selected parts are the abutment crossbeams and the slab ends most prone to the arrival of water laden with deicing salts through non-watertight expansion joints.

For underpasses, preferably made of frames, the selected parts are the walls, the sole plates and the crossbeams.

Finally, for the 3 viaducts, these are the abutments, piers (columns) and their footings as well as the reinforced concrete slabs.

2.2 Structure's categories

Given the multifactorial nature of this type of internal pathology linked to the exothermic nature of the cement, the pouring conditions of concrete, the environment in which it evolves, etc., it is necessary to define different risk categories.

The classification used in this study is described below:

- **No DEF**, no disorder observed and/or estimated internal heating temperature at construction below a threshold temperature.
- **Suspected DEF**, suspected DEF according to the disorders found during the detailed inspections and suggesting an internal expansion of the concrete (mesh cracking in particular) and/or estimated theoretical internal heating temperature higher than a threshold temperature.
- **Confirmed**, DEF confirmed in the SEM test as part of the additional investigations undertaken on the structures.

In order not to lose information and to make the prioritization of the structures more reliable, once the DEF was visually confirmed, which corresponds to the "suspected DEF" classification, the structures were sub-classified according to the evolution of the latter:

- **Non-triggered**, temperature reached above the different thresholds, but absence of visible disorders (with different levels of criticality depending on the estimated temperature of the concrete at the time of setting),
- **In the initiation phase**, presence of incipient pathologies that may correspond to DEF, with threshold temperature exceeded, but no confirmation of the pathology by SEM,
- **In the expansion phase**, presence of frank pathologies that may correspond to DEF and with threshold temperature exceeded and eventually determination of the cracking index and distance measurements, but no confirmation of the pathology by SEM,
- **Completed**, a somewhat peculiar class that is still assumed since all parts of the structure could possibly present a risk of triggering DEF.

Table 2.1: Examples of suspected DEF and DEF confirmed by SEM

PS n°746 at KP 283+607	PS n°586 at KP 267+924	PS n°658 at KP 274+892
		
Suspected DEF and occurrence of disorders at initiation stage	Suspected DEF and occurrence of disorders at expansion stage	DEF confirmed by SEM test

2.3 Characteristics of the concrete

The cement used in the concretes comes from two cement factories: namely the AIRVAULT and BEFFES plants, respectively located in the Deux-Sèvres department (west of France) and in the Centre Val de Loire region (central-western region of France). As part of the investigations achieved on the studied bridges, the presence of DEF was confirmed on bridges made of concrete from both plants. In that sense, the origin of the concrete is not a discriminating criterion for the final ranking and thus it is not selected as a prioritization parameter.

According to the analysis of the concrete compositions of all bridges, the elements that were judged critical and studied with regards to DEF are all elements made of a Q400 concrete (cement content of 400 kg/m^3).

2.4 Limit temperature

The comparison between the theoretical internal heating temperature and a threshold temperature was selected as being the most relevant criteria to judge its susceptibility towards a DEF triggering.

The study relies on the 2017 IFSTTAR guide [1] so as to judge on the threshold temperatures to be retained for each part of a structure. This threshold temperature has several levels depending on the class of bridge and the exposition class of the part of structure being concerned.

Starting from the elements of this guide [1], a summary of the selected thresholds was realized:

- For the most common bridges, with parts of structure that could be affected by stagnation of water, the recommended threshold temperature is 70°C. These are for instance every abutment crossbeams. It is also possible to assume a 75°C threshold temperature for the parts of structure not affected by stagnation of water and that are common as well, such as the desk ends. It has been nonetheless decided to keep the 70°C threshold in the initial approach, which is more unfavorable given the non-redundant nature of the structure elements.
- For the viaducts considered as exceptional structures and for those being affected by stagnation of water, the recommended threshold temperature is 65°C. Likewise, it is allowed to assume a 70°C threshold temperature for the parts of bridges not affected by stagnation of water but the 65°C threshold was eventually kept.

2.5 Estimated internal heating temperature

The internal heating temperature estimated during the pouring of concrete comes from the computation of the temperature rise in a surrounding environment. The internal heating temperature depends on the temperature of fresh concrete the day of pouring and the temperature rise that is the quotient between the concrete hydration heat (ΔQ) and the concrete thermal capacity (C).

2.5.1 Determination of the temperature rise (ΔT)

The values of the hydration heat ΔQ and the concrete thermal capacity C are data that were collected from the BEFFES and AIRVAULT cement factories.

The values of the temperature rise ΔT determined from the hydration heat of the concrete provided by the cement factories are respectively 60°C and 55°C for the concrete from BEFFES and AIRVAULT.

In opposition to the approach selected during the initial study, it was decided to recalculate the temperature rise while retaining the value ΔQ computed from the chemical constituents of the concretes and corrected by taking into account the thermal losses due to the element’s dimensions (abutment crossheads, slab ends) which can lead to a reduction of the theoretical value ΔT being computed.

From the 2017 IFSTTAR guide [1], we were able to determine the coefficient of reduction R from the thicknesses of the different parts of the bridges.

$$R\ (Th, Q41) = min\left(1\ ;\left(\frac{1}{1+\left(\frac{(max(0.3\ ;\ -0.0057 \times Q41 + 2.0558))}{Th}\right)^{1.5}}\right)\right) \tag{1}$$

If *Th* (*Thickness*) ≥ 5 m, then R = 1

For each structure, the theoretical heating value ΔT was determined (Table 2.2). Given a reasonable scattering of the results achieved, a temperature for each part of bridge was defined with a relative safety.

Table 2.2: Extract from the computations of ΔT per bridge and per parts of bridges

Bridges		Crossbeams				Slab ends			
PS	PR	Cement	Thickness (m)	R	ΔT	Cement	Thickness (m)	R	ΔT
859	294+947	Airvault	1,3	0,88	64,15°C	Airvault	1,1	0,87	57,95°C
658	274+892	Beffes	1,1	0,87	63,21°C	Beffes	0,85	0,83	60,59°C

The computation was made for 48 overpasses. The average temperature variations as well as the average standard deviations are provided in Table 2.3.

Table 2.3: Computation of total ΔT

ΔT average abutments crossbeams	ΔT average slab ends	ΔT average total	average abutments standard deviation	average slab ends standard deviation	average total standard deviation
63,06°C	58,50°C	60,78°C	1,39°C	3,93°C	3,72°C

In order to keep safe, the value ΔT total = 64,50°C was retained.

2.5.2 Determination of temperature of the fresh concrete

The computation of the theoretical heating temperature depends on the temperature of the fresh concrete the day of pouring and on the temperature increase due to the hardening of the concrete.

Given the absence of available data in the bridge documents in terms of fresh concrete temperature at the moment of pouring, the values were then determined from the external surrounding average temperature the day of pouring T_{ext}.

Corrections should be taken on the external temperatures in order to have a better estimation of the fresh concrete temperature during the pouring. Indeed, it can neither be negative nor close to zero as it is influenced by the temperature of the different constituents and of the heating related to their mixing. These constituents are usually stored outside, thus at a temperature close to the external one. Furthermore, the water used for the concrete production has globally a constant temperature whenever the period of the year but can be heated when too cold or cooled down when too warm.

Eventually, as part of this study, the correction was made to T_{ext}. according to the following procedure:

- Minimal concrete temperature is 8°C,
- For an outside temperature θ between] 5° ; 20°], increase of T_{ext} by [3 - 0.2 x (θ – 5)]
- For an outside temperature between] 20° ; 30 °], the value of T_{ext} is kept.

This correction of the outside temperature seems arbitrary but nevertheless allows to compensate the lack of input data.

2.6 SEM analysis

A SEM analysis is realized for the bridges suspected to be affected by DEF. The category of bridges for this type of investigation corresponds to the bridges for which the disorders studied in the inspection reports are relatively more significant or on more extended areas. From these tests, no AAR was detected.

Only the SEM analysis allows us to know if the concrete is affected by delayed ettringite and is thus affected by DEF.

2.7 Crack index

A cracking index was implemented and monitored for the bridges that show a slight suspicion of DEF. The category of bridges for this type of investigation corresponds to the bridges for which the disorders mentioned in the inspection reports are minor or located on a given unique part of structure.

The computation as well as the monitoring of the cracking index is a crucial investigation to take into account for the folder's ranking, because a zero point allows to quantify at which point the bridge or the part of bridge is cracked and to know about its progression rate, whether it is going fast or slow or if there is simply none. The results are presented in the Figure 2.1.c.

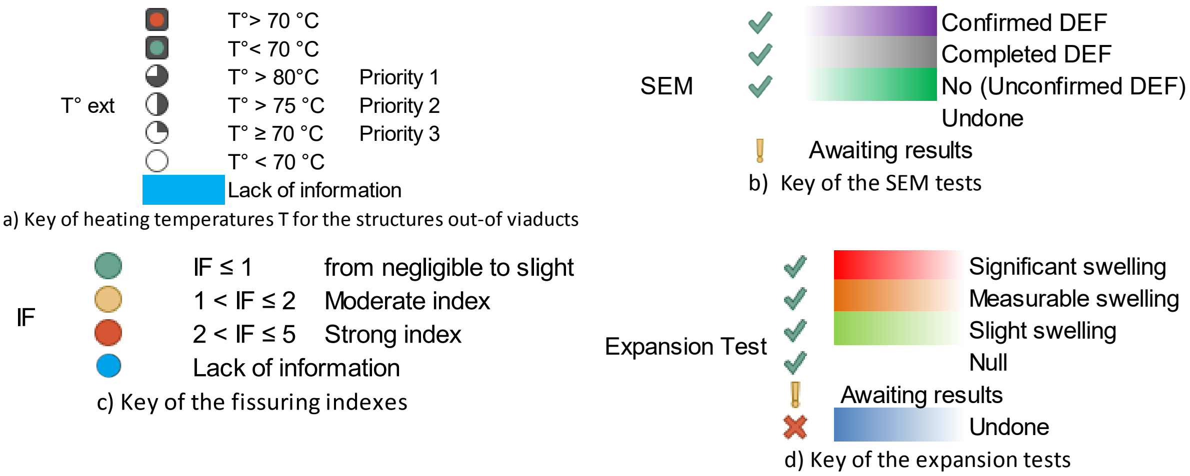

Figure 2.1: Key for classification

2.8 Expansion test

As part of the researches done on the bridges since the discovery of the DEF, expansion tests were performed on 7 overpasses. This kind of test enables to know whether the part of bridge being tested is affected by swelling and if this swelling is significant or not. The results of the expansion tests made on these structures are presented in Figure 2.1.d. Measures of expansion are realized on site by another company, with measures of ambient temperature next to the bridges.

2.9 Synthesis of classification

The whole set of the computed, estimated and /or found information in the different diagnostic reports was summarised as a pictogram banner related to each structure as shown below in Figure 2.2:

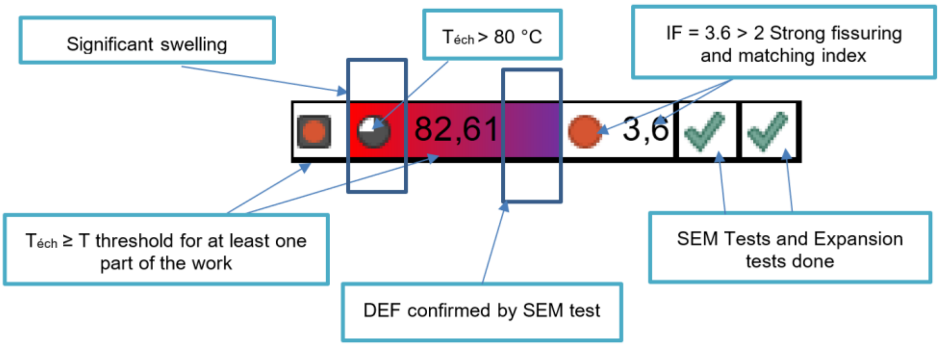

Figure 2.2: Classification overview

Figure 2.3 provides an extract of the synthesis table for a part of the overpasses for illustration purposes.

The values shown in the synthesis table correspond to the most unfavourable cases for each test and to each result for all the parts of bridges being studied. We note that almost all the bridges show excessive heating temperatures, exceeding the 70°C threshold.

A71		Supporting beam										Slab fallouts									
		West					East					West					East				
Overpass	KP	Tech°C	MEB	Expansion	Priority	IF	Tech°C	MEB	Expansion	Priority	IF	Tech°C	MEB	Expansion	Priority	IF	Tech°C	MEB	Expansion	Priority	IF
859	294+947	85,55	2	1	86					83					77					77	
857	294+742	86,75		0	87					83					88					88	
824	291+520	81,41		0	81					82					83					83	
787	287+727	81,89		0	82					75					81					81	
746	283+607	81,25		0	81					81					87					87	
734	282+477	90,05		0	90					90					77					77	
714	280+427	86,75		0	87					87											
712	280+235	75,09		0	75					77					80					80	
683	277+357	77,89	0	2	78					81					81					81	
666	275+687	88,55		0	89					83					81					84	
658	274+892	82,85	2	0	83					84					83					83	
645	273+844	82,69		0	83					84					79					79	
623	271+609	86,35		2	86					87					79					79	
617	271+49	78,37		0	78					81					78					78	
611	270+404	84,37		0	84					83					70					70	
599	269+249	80,69	2	0	81					88					82					82	
586	267+924	85,05		0	85					86					77					77	
559	265+227	72,45		0	72					72					70					70	
531	262+369	79,97		0	80					79					70					70	
515	260+796	72,45		0	72					78					70					70	
503	259+569	82,61	2	2	83					82					77					77	
467	256+39	72,45		0	72					75					70					70	
443	253+664	72,45		0	72					78					78					78	

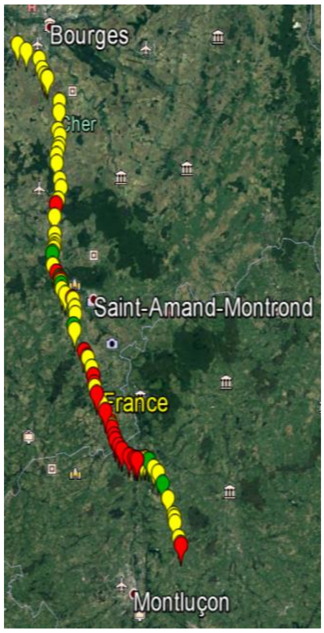

Figure 2.3: Extract from the summary table and distribution of bridges

On Figure 2.3, the GIS (Geographic information system) extract indicates three kind of dots. The red dots correspond to the bridges on which the DEF was diagnosed, and the yellow ones to the bridges potentially affected. The green dots then refer to the bridges that are not at risk. This GIS shows that the bridges on which the DEF was confirmed are precisely located on a distance of roughly 35 kilometres for 20 structures out of the 26 having been analyzed by SEM.

3. RE-COMPUTATION WORK

3.1 Presentation of the re-computation (overpass 683)

Following APRR's request, a bridge among the 76 affected by DEF was selected to model the swelling caused by the DEF. The PS 683 on A71 was chosen because it is one of the structures most affected by DEF among the 76 being studied.

Indeed, according to the special detailed inspection carried out in 2013, the structure shows significant cracking on the west side of the south abutment base with a significant evolution (Figure 3.1) and cracks on the slab end on the southwestern side (Figure 3.2). In addition, these two parts of the structure are monitored by extensometer instrumentation for the abutment crossbeams and by stud deformeters (400 mm base) for the slab end. They were also analyzed by SEM and were the subject of residual expansion test only for the abutment.

Figure 3.1: Base of the South abutment

Figure 3.2: Slab end

The two parts of the structure involved by the recalculation are:

- a 5.5 m long abutment crossbeam, 1.30 high and 1.1 m wide, based on two piles 0.80 in diameter and 3.40 high, supported by a foundation footing,
- a 52.20 m long prestressed slab, 0.85 cm high and 6.30 m wide, prestressed by 7 tendons each composed of 12 strands of 15.7 mm diameter, from a class of 1,770 MPa.

3.2 Purposes

This modelling is carried out using CESAR-LCPC© software. This software allows a three-dimensional mesh to be created. It has several modules, including the TEXO module, which calculates the heating temperatures at the heart of the part under study, and the RGIB module, which determines the swelling induced by the DEF and then calculates the stresses in the steels a posteriori.

For the TEXO module, the temperature rise in the elements of the model was studied over the first 3 days (i.e. 72 hours) from the pouring of the concrete, and during the time the concrete sets. For the RGIB module, the behaviour of the concrete was studied over a period of 20 years with time steps of 180 days. Note that the RGIB model is based on a swelling law based on accelerated expansion tests. These tests last 1 year in the laboratory but actually represent on site a duration 5 to 10 times greater. Therefore, the time steps indicated in the figures 3.3, 3.4 and 3.7 are to be multiplied by 5 or 10 to have an equivalence with the reality.

The aim of this modelling is to determine the heating temperature deep inside of the studied elements according to their manufacturing history, and then to evaluate the swellings due to DEF from the results of expansion tests on cores taken directly from the structure under consideration. In post-processing, the results of this modelling should make it possible to deduce the stresses reached in reinforcement steels.

These calculations make it possible to study the impact of DEF pathologies on the structural capacity of the elements and thus target the repair, reinforcement and/or replacement work to be carried out on the structure.

3.3 Hypothesis

3.3.1 Geometry

In order to reduce calculation times and to better visualize the results of thermal reactions at the core of the part, the CESAR modelling was carried out on only half of the abutment crossbeam and a quarter of the slab. In addition, the chosen mesh was composed of square meshes of 5 cm by 5 cm allowing the steel reinforcements to coincide with the meshes of the model.

3.3.2 TEXO Module

Several thermal parameters must be defined, such as:

- **The thermal conductivity** which translates the displacement of thermal energy from the hot parts of the concrete piece to the formwork used and to the outside environment which represent the cold parts. The system is considered homogeneous and isotropic (diagonal matrix). The values in this matrix are generally taken around **1.67 $W.m^{-1}.K^{-1}$** in the literature. ([2] Tailhan &al. (2010)).
- **The heat capacity** which quantifies the ability of the concrete to absorb energy through heat exchange during chemical reactions during the curing of the concrete. For concrete, this parameter is generally equal to **$2.4.10^6$ $J.m^{-3}.K^{-1}$**.
- **The conditions of exchange** by convection between the surfaces of the model and the external environment which depend on the presence of a formwork element or not. For surfaces with formwork the exchange coefficient is taken at **7,2 $W.m^{-2}.K^{-1}$**, while for surfaces without formwork it is **9,7 $W.m^{-2}.K^{-1}$**. These values are given in the literature. On the symmetry faces of the model, since there is a perfect exchange equilibrium in these sections, the exchange coefficient is **zero**.
- **The temperature of the fresh concrete** on which the heating temperature directly depends on. In the case of this study, it was taken at **17°C.**
- **The outside temperature** that was collected from Météo France over the 3 days following the pouring of the piece. For the sake of precision, the temperatures were given every hour, i.e. a total of 72 temperature values to be entered in the model.
- **The results of the calorimetric tests**, which are heat release tests carried on a concrete sample in semi-adiabatic conditions. The study focused on typical concrete tests already carried out in the literature and that were most close to the concrete used on the structure.

3.3.3 Module RGIB

Several physical parameters must be defined, such as:

- **The swelling curve** to be derived from accelerated expansion tests on concrete cores. This makes it possible to establish a swelling law based on Larive's law. The latter then makes it possible to derive the characteristic time and the latency time of the swelling. For our study, a characteristic time of **62 days** and a latency time of **2500 days** could be deduced from the expansion tests (Figure 3.4). The total swelling is generally taken at **1%** (Figure 3.3).
- **The amplitude of swelling**, which is a coefficient that relates the thermal history to the swelling from the expansion tests. It was taken at **$1,44.10^{-6}$ s^{-1}** for our study.
- **The threshold temperature** which is the temperature at which DEF is taken into account. According to the literature and in particular the tests of Baghdadi's thesis [3], this temperature is **50°C**.
- **The activation energy** for ettringite decomposition which is taken as **408 J/mol** according to the tests in Baghdadi's thesis.
- **The chemical damage** taken into account in our study. This means that we integrate the variation of the Young's modulus of the concrete over time in a part affected by DEF. According to the tests of Brunetaud [4], the maximum damage is taken equal to 0.7, the damage evolution parameter to 3 and the maximum threshold for the chemical expansion of which cracks occur to 0.01%.
- **The creep of the concrete** under compression induced by the restrained swelling of the concrete is also taken into account for the study. This creep has a significant impact on the working rate of reinforcement steels.

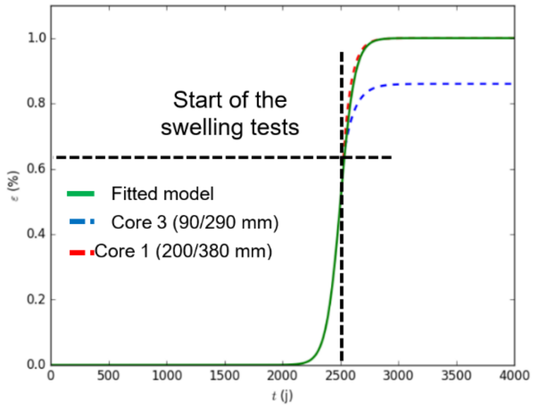

Figure 3.3: Calibration of the swelling model from the core

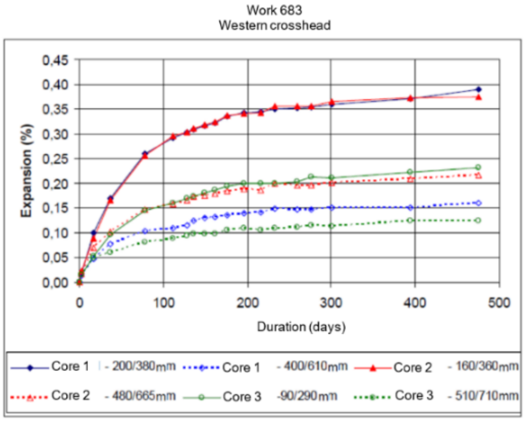

Figure 3.4: Evolution of the longitudinal expansion of 2 samples per concrete core

3.4 Results

3.4.1 TEXO Module

On the basis of the different parameters and calculation hypotheses, the temperatures in the different concrete parts could be determined from the TEXO module (Figure 3.5) and these values were compared to the theoretical values determined by the first macro approach carried out on the 76 structures.

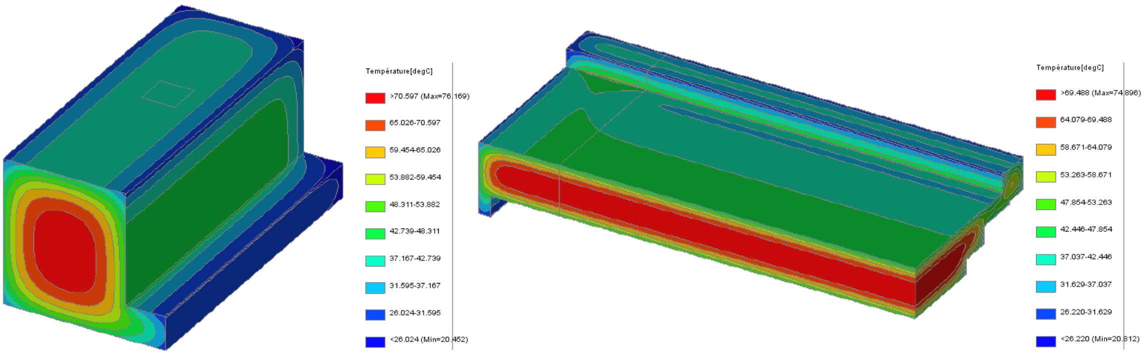

Figure 3.5: Illustration of Temperatures inside the abutment half-crossbeam and inside the quarter-slab

For the abutment crossbeam, a temperature of 76,2°C was found, to be compared with the 78°C calculated in the theoretical study.

Concerning the slab deck, a maximal temperature of 75°C was recorded, that is to be compared to the 79°C provided by the theoretical study, that means a slightly more favourable temperature but still within the order of magnitude. However, these differences remain negligible considering the approximations made during the first approach explained at the beginning of this paper, in particular at the level of the definition of the temperature rise values per sub-part of the structure.

3.4.2 RGIB Module

After making the calculation with the TEXO module, the calculation under the RGIB module can start with the parameters defined above, with only DEF that occurred on this structure. The Figure 3.6 represents the swellings under CESAR-LCPC.

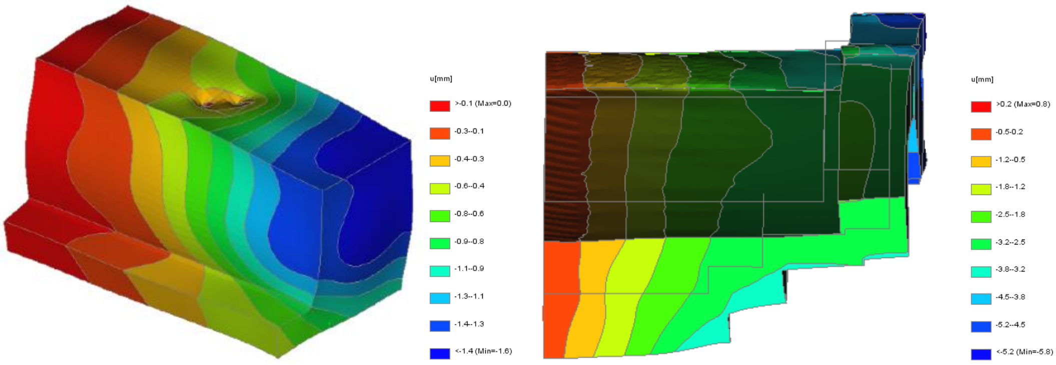

Figure 3.6: Representation of the swellings of the half-crossbeam (left) and quarter-slab (right)

In a first calculation, the two parts of the structure have been studied separately. The swellings due to DEF were found to be of the order of 1.6 mm in relation to the X-axis (parallel to the length of the crossbeam) for a half crossbeam, which means a deformation over the whole crossbeam of 3.2 mm. For the slab, it is about 5.8 mm, i.e. a total transverse deformation of 11.6 mm.

The monitoring located on the crossbeam gave an elongation of 0.12 mm per year in 2010. By positioning the value given by the distancemeter on the calculation model, it is possible to locate the DEF level at the time of measurement in 2010 (Figure 3.7).

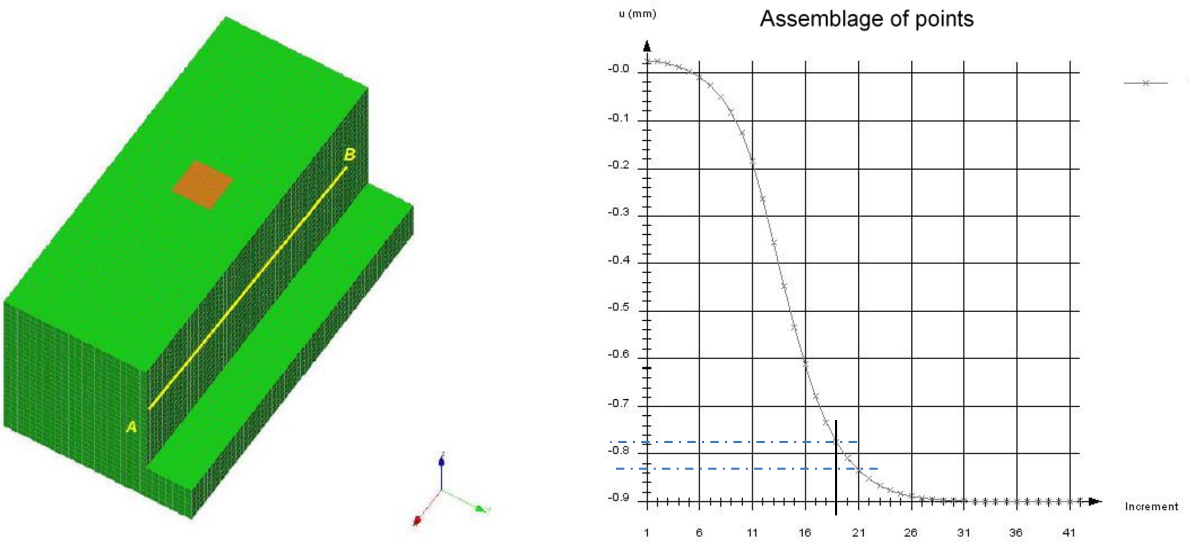

Figure 3.7: Illustration of the distancemeter and of the evolution of the deformation at point B

The elongation of 0.12 mm/year is found between increments 19 and 21 (1 year = 2 increments, so 10 years in the model, 50 to 100 years in reality) with a deformation of 0.77 mm and 0.83 mm respectively, i.e. 0.06 mm/year for the half-crossbeam or 0.12 mm/year for the full crossbeam. It was therefore deduced that the crossbeam still had some swelling potential in 2010. Today, it is therefore possible that its reaction is ending.

In a post-treatment, stresses in the reinforcement steels could be determined according to the deformations induced by the DEF. These stresses were calculated at characteristic points (Figure 3.8).

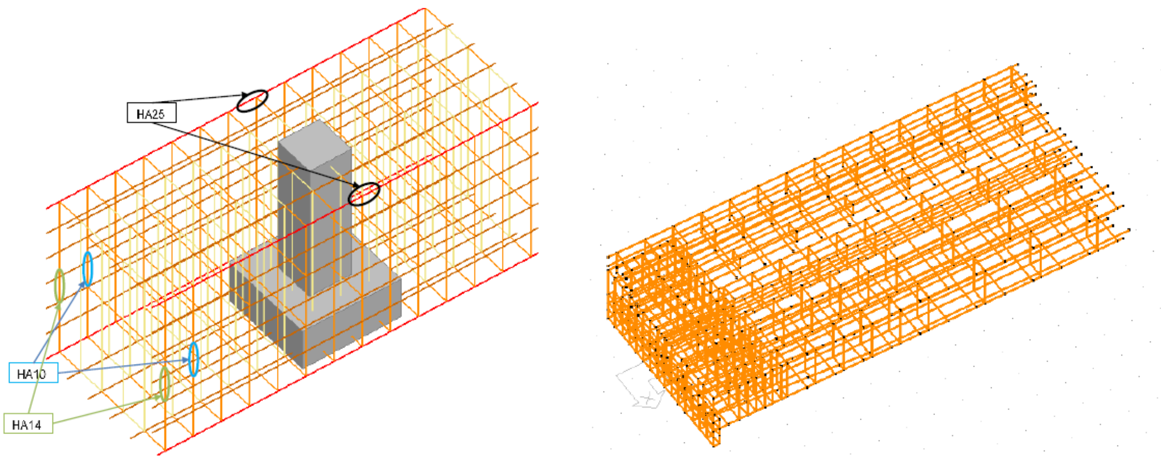

Figure 3.8: Illustration of the reinforcement inside the crossbeam (left) and inside the slab (right)

For the crossbeam, the maximum working stress in HA14 and HA10 reached 380 MPa and 360 MPa respectively at the end of the swelling reaction, while HA25 reached a maximum of 165 MPa. As a result, the vertical steels are highly stressed exceeding the regulatory limit (240 MPa) but not the ultimate one (400 MPa).

For the slab, the horizontal steels perpendicular to the direction of the roadway in the slab end are the most stressed. In fact, their working stress was estimated at 480 MPa exceeding the fracture limit. However, it turns out that this stress is only found very locally in the steel bars.

The other most stressed steels are also horizontal steels but in the current part of the slab. They reach on average 290 MPa exceeding the regulatory limit but not the breaking one. As far as prestressing is concerned, the behaviour of the tendon is shown in the Figure 3.9.

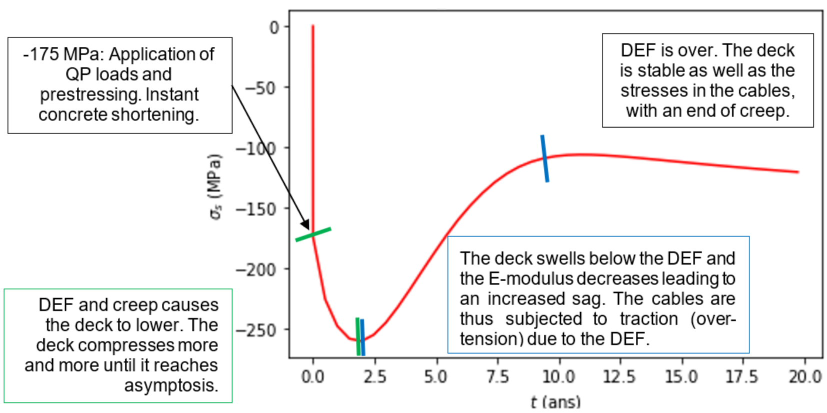

Figure 3.9: Stress evolution in the tendon.

According to the original calculation note, the tendon tension after losses is 1065 MPa. In the graph above, the maximum additional stress is 260 MPa which gives a total stress of 1325 MPa (less than the ultimate stress). The tendons are therefore capable of withstanding the DEF phenomenon.

4. REMEDIAL MEASURES

4.1 Practicable treatments of affected bridges

Some works are being considered depending on the type of structure, on the severity of the pathology but also according to its level of development. The main works concern the bridge' tightness as well as its potential structural reinforcement.

In the first case, the bridge's tightness aims at forestalling any form of triggering and/or development of DEF.

The second case aim at implementing a curative treatment of the pathologies caused by DEF and other pathologies it triggers.

4.1.1 Treatment of underpasses' tightness

As part of this study, it was proposed for the underpasses -in addition to usual works of degraded concrete repair- the following waterproofing works recommendations:

- Implementation of a liquid waterproofing system (SEL) on the facing walls and on the wing or side walls.
- Rehabilitation of the water drainage system of the side walls or wing walls. This work is depending on the bridge configuration. If it displays a suspicion of DEF and if it is equipped with barbicans, then a high-water pressure cleaning system is advised. If the bridge shows DEF presence confirmed by SEM test, then the use of weep holes in addition to high water pressure system is recommended.
- Establishment of metal protections on sharp edges affected by bad weather, notably on wing or side walls.

4.1.2 Treatment of overpasses' tightness

As part of this study, – and in addition to usual repair works of degraded concretes – several recommendations of waterproofing works on abutment crossbeams and on slab ends were proposed, these elements being the most pathological, with various levels of treatments:

- Implementation of a liquid waterproofing system (SEL) on the crossbeam of the abutments and of the slab fallouts. This treatment was recommended once the structure shows a suspicion of DEF.
- Hydrodemolition and reconstruction of the front wall of the abutment and/or of the slab ends. This treatment is then recommended whenever the seeable abutment faces display a relatively significant cracking. This step aims at demolishing every unstable element in order to obtain an edge that would allow the implementation of an appropriate waterproof protection.
- Treatment of the buried face of abutments through the implementation of a transversal and longitudinal drainage. This kind of works may lead to a too significant discomfort for the users, and they were therefore not retained for the rest of the study.

4.2 Prioritization of repair works

According to the considered treatments, it is then possible to look for maintenance scenarios on the basis of a financial optimization process, or of the exploitation of the supported and crossed roads, or even on both approaches.

The research for a financial optimisation may lead to the research of a prioritisation per classes of works and thus optimize the posts over the same year. Indirectly, this also leads to works of the same scale the same year, and therefor also seeking to work according to the priority of intervention of the works.

The investigation of an optimisation of the discomfort of exploitation may lead to maximise the typologies of works, and thus the required competences and thereby decrease the efficiency of the firm's organisation, or even to mix the prioritisations of works. In that framework, when the exploitation discomfort on the highway is minimised, the discomfort on the secondary road network could be maximised.

The research of an optimisation of both visions represents an intermediate solution that tends to maximise the sections of works, minimise the typologies of works and finally respect the transparency of secondary roads.

Furthermore, this approach takes into account the aspects of precautionary treatment for the bridges affected without pathologies showing evolutive trends and that may wait a little before curative works.

An investment plan was realised according to the types of works recommended for each bridge. In 2018, waterproofing works and structural reinforcement on a selected overpass as a test bridge was planned; In 2019, waterproofing works on underpasses were realized; In 2020: waterproofing works on overpasses; And in 2021 to 2022: waterproofing works and structural reinforcement on two overpasses selected according to their levels of pathologies are intended.

Upon realization of this article, reinforcement and waterproofing works have been made for the selected overpasses and some other works are currently being realised for the waterproofing of 16 underpasses.

5. CONCLUSIONS

From this study, it has been possible to analyse all data of the 76 bridges to set the priorities of a global maintenance strategy for the owner. In a second part, repair and reinforcement works have been defined and the first reinforcement works done.

In parallel, EF calculation has been made for one particular bridge to estimate its condition at the end of the DEF. This calculation are unfavourable but give an adequate representation of the maximum risk of damage for this bridge.

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