Lessons learned from structures damaged by delayed ettringite formation and the French prevention strategy

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The Development of DEF

The expansive sulphate internal reaction due to Delayed Ettringite Formation (DEF) can damage concrete structures severely. The primary ettringite (a hydrous calcium trisulphoaluminate) is a normal reaction product formed from the reaction of C_3A and C_4AF with gypsum during the plastic stage of the hydration of Portland cement. However, when peak temperatures in concrete are over about 65°C, the sulphates may be incorporated in other cement phases. After concrete hardening, the very slow formation of higher volume secondary ettringite may occur as water is taken into the crystal structure which can lead to potentially disruptive expansion. DEF is defined as the formation of ettringite in a concrete after setting, and without any external sulphate supply, but with a water supply. DEF appears in concretes exposed to frequent humidity or contact to water, and subjected to a relatively high thermal treatment (> 65°C) or having reached equivalent temperatures for other reasons (massive cast-in-place concrete, concrete casting during summer, etc).

The first reported cases of DEF occurred in some precast concrete elements subjected to a heat treatment unsuited to the composition and the environment of the concrete. International examples of DEF include railway sleepers (Tepponen 1987, Heinz 1989, Vitouva 1991, Shayan 1992, Oberholster 1992, Mielenz 1995, Sahu 2004, Santos Silva 2008), and massive cast-in-place concrete components (Collepardi 1999, Hobbs 2001, Thomas 2008, Ingham 2012). DEF was first observed in France, in 1997 (Divet 1998), on bridges whose concrete had been cast on site. The bridge parts damaged by DEF were primarily massive structural elements (piers, crossbeams on piers or abutments, etc.) in contact with water or subjected to high moisture. DEF is distinct from the more traditional sulphate reactions where the sulphates attack the concrete from outside and create a progressive degradation from the surface towards the interior of the element. DEF affects the interior concrete without any ingress of external sulphates. It leads to a concrete swelling and the cracking of the structure.

Diagnosis of DEF

Laboratory studies were conducted using petrographic and mineralogical analysis (Deloye, 1977, AFPC-AFREM 1997). They included the following investigations: porosity, chemical analysis, X-ray diffraction, thermal analysis and scanning electron microscopy (on polished sections and broken surfaces). Several core samples were taken from the structures, with varying depths. The specimens were taken from both degraded zones and un-degraded zones to allow comparison between the two.

It is important to note that no alkali-silica reaction product was observed in the following investigated structures cases. The microscopic analyses showed that the studied concretes have been subjected to a major expansive internal sulphate reaction as ettringite formed after hardening. This check was conducted with the objective of avoiding any confusion with damages caused by the alkali-silica reaction. This is important as when both causes of deterioration are present, it is difficult to know which reaction is mainly responsible for the damage.

The internal sulphate reaction results in the presence of ettringite. This latter was present throughout the material and on all the examined surfaces. The ettringite that is generally observed in the concrete has a wide range of textures, but it is mainly poorly crystallized and massive. Furthermore this form is principally present at the cement paste/aggregate interfaces. Figure 1 is a back-scattered electron (BSE) image showing ettringite-filled gap around aggregate particles.

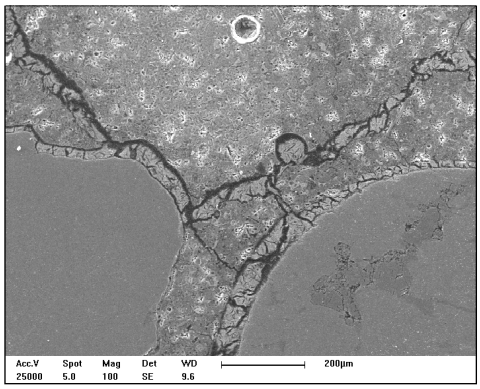

Figure 1: BSE image of a polished sample showing ettringite-filled gap around aggregate particles.

Temperature is a fundamental parameter for the formation of delayed ettringite. For this reason it is necessary to evaluate the temperature within concrete during setting. Our starting hypothesis was that the temperature rise in a massive concrete element can be estimated on the basis of the temperature rise under adiabatic conditions. The temperature field T(X,t) was predicted at all points of coordinates X in the structure and at each time t. This prediction is based on a finite element calculation which requires the knowledge of the exothermic properties of the concrete, the geometry and the parameters that regulate exchanges between the environment and the concrete element. Modelling was conducted as a diagnostic aid to understand the damage observed in the structure after several years. Numerical simulation of the thermal effects was undertaken with the TEXO module which is part of the CESAR-LCPC finite element program of IFSTTAR.

Lessons learned from investigations of structures

Most of the following cases are presented in detail in the report of L. Divet (Divet 2001).

The Ondes bridge

This bridge, located near Toulouse, was completed and opened to traffic in 1955. It consists of four pre-tensioned beams which are joined together by a top slab and crossbeams (Fig.2). It has a total length of 202 m and consists of five independent spans. The average height of the piers is 6.8 m. They are constructed on mass concrete which is set in the molassic substratum and they consist of two circular columns, 2 m in diameter, which are connected at the top by a

cap beam (see Fig.2). These latter are either hollow, at piers P1 and P3 supporting the moving bridge bearings, or solid, at piers P2 and P4 to which the fixed bridge bearings are fitted.

The damage only affects the solid cap beam of pier P2 (Fig. 3), the concrete of which was cast in August 1954. This is a solid parallelepiped reinforced concrete cap beam, measuring 1.5 m high, 8.2m long and 2.7 m wide. The first visible defects were observed during a detailed inspection conducted in 1982. The inspection report mentions the presence of cracking and the growth of vegetation at the ends of the pier cap beam. The previous inspection reports did not mention these defects. We can consequently assume that the damage first occurred in the early 1980s.

Figure 2: Overall view of the Ondes bridge

Figure 3: View of the end of the cap beam in 2005, after the treatment applied in 1995

The damage appeared as a network of widely-spaced multidirectional cracks. Cracking sometimes has a preferential orientation that depends on how the reinforcement is distributed. The crack widths vary from a few tenths of millimetres to a few millimetres according to the zones. The cracks are frequently emphasized by moisture and, occasionally, a whitish or greyish substance has been observed to exude from the cracks. The horizontal cracks appear to be more severe and are occasionally as much as 7 mm wide. Lastly, the damage is mainly present at the cap beam ends, on the facings that are directly exposed to bad weather and water dripping from the footpaths. No damage was found on the central part of the cap beam.

The concrete of the cap beams has a specified cement content of 350 kg/m^3 of Ordinary Portland Cement (CEM I type), with a water/cement ratio of 0.5 in mass. The aggregates are alluvial. The investigations conducted in the laboratory showed that the deteriorated concrete of pier P2 did not conform to the specifications. Analysis showed that the concrete contains a cement overdose of about 100 kg/m^3 (cement content of 430 kg/m^3). The mix is well composed of sand and gravels 5/15, but it incorporates gravels 25/50 that were not specified initially (see table 1).

The heating cycle of the concrete during its setting was then estimated by a numerical simulation using the TEXO module. For this, information was required on: meteorological data during the setting period of the concrete, geometry of the structural element, type and duration of use of the formwork, construction phases, exothermic properties of the concrete. A three-dimensional thermal modelling was performed using this data (fig. 4). Figure 5 shows a plot of temperature versus time. The temperature increase was important for the cap beam of pier P2: the maximum temperature reached in the centre of the material was close to 80 °C, while for the cap beam of pier P4, the temperature did not exceed 70 °C. Moreover, the temperature in the concrete of pier P2 remained over 70°C during about 70 hours.

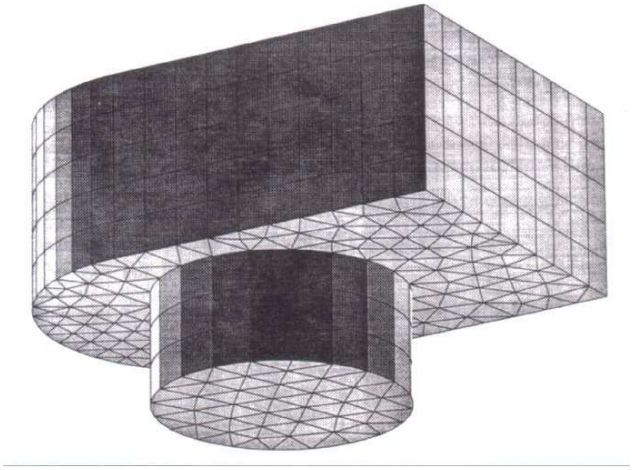

Figure 4: 3D modeling of the cap beam of pier P2 with CESAR-LCPC.

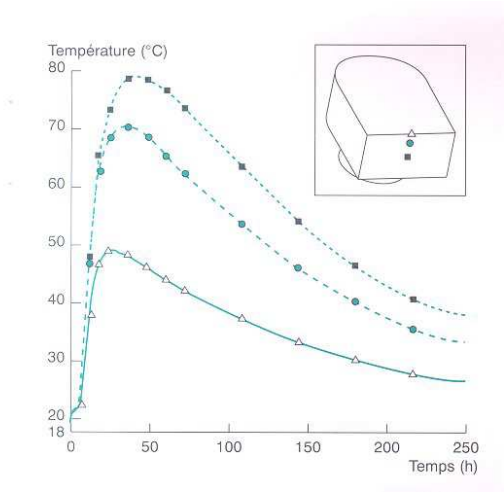

Figure 5: Computed temperatures in the cap beam of pier P2

In consequence the deterioration due to the DEF is limited to that in the cap beam of pier P2. This is the consequence of the initial heating of the concrete from the combination of several causes: use of an exothermic OPC for a massive concrete part, pouring of the concrete during summer time without any particular precautions and an overdose of the cement relative to the initial specified concrete composition.

As part of the remedial programme on the Ondes bridge, the waterproofing membrane was repaired and the expansion joints of the roadway and sidewalk were reformed in June 1995. On pier P2, the extremities of the cap beam were treated by an injection of the cracks and then protected by a surface coating. In July 1996, a discontinuous horizontal crack of width 0.2 mm was observed. Water inflows are also observed on the extremities, despite the repair made in 1995. The evolution of the phenomenon is confirmed in 1998 with the occurrence of new cracks and by the results of the monitoring of the global deformations that has been installed. This attests of the difficulty to implement durable repair solutions for this type of pathology, and justifies the interest in enforcing preventive measures during construction to avoid DEF.

Table 1 : Specified concrete composition of the different bridges (in kg/m^3).

Material	Cement	Water	Filler	Sand	Gravel	Gravel	Gravel
Ondes	350	175		750 (0/5)	1100 (5/15)		
Bourgogne	400	180		730 (0/4)	376 (4/12)	700 (12/20)	
Lodeve	400	188	140* (0/3)	555 (0/4)	395 (4/10)	310 (10/20)	
Bellevue	380	205		800	290 (6/10)	130 (10/14)	640 (14/20)
Beynost	350	170		750 (0/5)	250 (5/10)	930 (10/20)	
Cheviré	385	185	75	730	285 (6/10)	130 (10/14)	630 (14/20)

* calcareous sand

The Bourgogne bridge

The Bourgogne bridge is a cable-stayed bridge built during the years 1989-1992 over the Saône River in Chalon/Saône (Fig. 6). It has a total length of 351 m and comprises 8 spans. The main span is 151.80 m long, and the pylons are 46 m high. The deck is made of a prestressed concrete slab. The lower part of each pylon is composed of a footing, a base and two legs. The dimensions (in metres) with respect to length, width and height are respectively for the footing: 15.20 x 6 x 2, for the base: 11.90 x 3 x 6.5, and for the legs: 2.20 x 3 x 6.

The footing is always immersed, the base is partially immersed in water but may be sometimes fully immersed during floods, and the legs are always above the water level of the river. The immersed parts of the base of the pylons P3 and P4 are affected by a multidirectional cracking like in the aerial parts (fig. 7). The width of the cracks reaches several millimetres on P3, and is lower than one millimetre on P4. A high cracking density is observed on the faces of the base, with a maximum density in the middle of the faces. The most important crack is observed on the top face of the base with a width of 1 cm. Some cracks also show an offset of the crack lips which is typical of a differential expansion inside the concrete; the most important offset is 2 to 3 mm on the top face of the P3 base. The lateral faces of the legs of pylons P3 and P4 present some cracks that are oriented horizontal, with widths varying between 0.2 and 0.4 mm.

Figure 6: Overall view of the Bourgogne bridge

Figure 7: Cracking of the aerial part of the pylon base

For the footing, the base and the legs, the cement content is 400 kg/m3, the water/cement ratio is 0.45 and the cement used is a CPA 55 R (CEMI 52.5 N type).

A finite element calculation of the temperature reached in the base and in the legs was conducted with the module TEXO, and with the help of adiabatic test results obtained on a reconstituted concrete. The results show that the temperature in the footing, the base and the two legs exceeds the 65 °C threshold, with respectively 79 °C for the footing and the base, 71°C for the lower part of the legs and 61 °C for the upper part of the legs. They show that the temperature of the base remains over 70 °C during 6 days, while those of the footing remains over 70 °C during 3 days. The laboratory investigations (particularly with the Scanning Electron Microscope) confirmed that the concrete of the basement was affected by DEF and that the concrete of the footing and legs was unaffected.

The Lodève Bridge

This bridge was constructed during the years 1980-1981. It is a statically determinate pre-stressed beam structure with a three span deck which is 120 m long and 13 m wide. The deck is supported by two abutments and two piers constructed on footings. The damage is confined to the cap beams of the two piers. Each pier (Fig. 8) consists of a pillar whose cross-section fits within a rectangle measuring 3.1 x 7.5 m, with a height of 14.5 m in the case of the northern pier and 9.3 m in the case of the southern pier. Each pillar is topped by a cap beam which has a length of 14 m, a width of 3.5 m, and whose height varies between 1.1 m and 2.0 m. The extremities of the cap beams are covered with architectural facings consisting of six prefabricated reinforced concrete units. The two cap beams were built in August and September 1980.

The cap beam on the North pier has closely-spaced vertical cracks on both sides with a maximum crack width varying between 0.2 mm and 1.2 mm (fig. 9). The extremities of the cap beam are covered by the architectural facings which prevent their observation. Some cracks contain streaks of calcite, which indicates a circulation of water inside the concrete. Several horizontal cracks are also present and appear to be located at construction joints. The results of the 1997 crack survey for the cap beam of the north pier shows a considerable increase in cracking as compared with the 1989 inspection.

Figure 8: Overall view of a Lodève Bridge pier

Figure 9: Network of cracks on the cap beam of the North pier

The cap beam of the South pier suffers from the same type of damage, but seems less severely affected. The maximum crack width is only 0.3 mm, except on the east extremity where widths attain 1.7 mm. Furthermore, the network of cracks is denser and efflorescence can be observed in this zone. Lastly, the detailed inspection conducted in 1997 showed a lack of drainage from the supports (abutments and piers). Therefore, some parts of the supports are abnormally exposed to a humid environment.

The damage rating of a concrete structure may be determined with the help of a cracking index which corresponds to the summation of cracks intercepted on axes of variable lengths and normalized to a length of one metre (LCPC 1997). One of the cap beam of the Lodève bridge has been monitored with the help of cracking indexes and measurements of overall movement (LCPC 2009) since 2000. The cracking index measured in the central part of the cap beam is 0.55 mm/m and is stabilised. According to the classification (LCPC 1997), this value corresponds to a low cracking. For the extremity of the cap beam, the cracking index

reached 1.64 in 2002 and 2.0 in 2007; it indicated an evolution of the phenomenon and the cracking is rated moderate to high. This was confirmed by the overall movement monitoring which gives a global deformation rate for the cap beam extremity of 0.2 mm/m/year.

The concrete of the piles and cap beams was made with an OPC cement (CEM I type) having a rapid hardening; the cement content was 400 kg/m^3 and the W/C ratio was 0.47. The aggregates are silico-calcareous.

A finite element calculation of the temperature reached in the cap beam and the pillar (module TEXO) was carried out to explain the difference of behaviour between the damaged cap beam and the non damaged pillar. Laboratory investigations (particularly with the Scanning Electron Microscope) confirmed that the concrete of the pillar was unaffected by DEF. In the cap beam, the calculated temperature increase was close to 80 °C in the centre and the temperature remained over 70 °C for nearly 5 days. In the pillar, which is less massive and has a particular geometry which favours the heat losses because of a larger surface area, the maximum calculated temperature reached 70 °C in a relatively small zone in the centre. Most of the concrete of the pillar reached a temperature below 65 °C. This was the main reason for the absence of DEF in the pillar but it is also less exposed to water ingress and humidity..

The Bellevue Bridge

This bridge was constructed in Nantes in 1988–1989 and opened in 1990 (Fig. 10). It is a continuous box girder bridge with 6 spans and a total length of 385 m, and has 7 supports (2 abutments and 5 piers). The 5 reinforced concrete piers are massive and are 6.3 m long by 3.0m wide. The damage is mostly confined to pier P6 on which multidirectional cracking typical of that caused by an internal swelling reaction is apparent (Fig. 11). In addition the map cracking is mainly localised within the tidal zone, where the average tidal range is between 3 and 4 metres. The four sides of the pier are affected and cracking seems to be progressing in the dry zone. The concrete of this pier was cast in August and September 1989. Pier P4 also exhibits map cracking, but this is less severe. Lastly, piers P3 and P5 have vertical cracks with widths reaching 0.8 mm. However, this cracking was observed immediately after construction and was attributed to thermal shrinkage of the concrete.

The cement content is 380 kg/m^3, the W/C ratio is 0.54, and the aggregates are predominately from siliceous and silicate rocks. The cement is a Portland cement (CEM II/A type) composed of about 10 % of calcareous fillers (see table 1).

Figure 10: Overall view of The Bellevue Bridge

Figure 11: Pier 6 of the Bellevue Bridge with map cracking in the tidal zone.

The evolution of the temperature inside the concrete has not been estimated by a finite element calculation. The maximum temperature was estimated by the simplified method of calculation presented in the annex 4 of the LCPC guide (LCPC 2007). With the hypothesis that the temperature of the fresh concrete at the moment of pouring was equal to 20 °C, the maximum temperature at the centre of the pier was estimated to have reached 75 °C.

The Saint-Maurice de Beynost bridge

This bridge, built in 1992, has a deck made of reinforced concrete beams and is composed of three independent spans made continuous on piers by the slab over the beams. The spans are skewed and have different length (14.92 m – 17.44 m – 9.47 m). The deck is resting on two abutments and two intermediary piers (fig. 12). Each abutment is composed of 4 rectangular columns topped by a cap beam. Each pier is composed of 5 columns of 5.4 m height and also topped by a cap beam.

The damage has occurred only on the extremity of the cap beam of the pier P2 which is exposed to the elements (fig. 13). A map cracking was observed during the first detailed inspection of the bridge in 1992. This cracking is evolving since the width of the cracks was below 0.3 mm in 1992 and reached up to 2 mm in the detailed inspection of 1997.

Figure 12: Partial view of the Saint Maurice de Beynost bridge

Figure 13: Map cracking at the end of the cap beam exposed to rain

From the documentation of the bridge, it was possible to find the dates of casting of the concrete in the different parts of the bridge. The cap beam of pier P2 was built in August 1982. The cement is an OPC (CEM I type), the W/C ratio is 0.49, and the concrete composition is close to the specified value of 350kg/m^3 given in table 1.

The calculation of the temperature increase with time in the bridge parts (pier and cap beam) was conducted with the TEXO. It was based on the meteorological data for the construction period and data obtained from adiabatic tests carried on the concrete composition reconstituted in laboratory. No damage has occurred in the piers, and the calculation shows that the maximum temperature reached at the centre of the piers was 61 °C. For the cap beam of the pier P2, laboratory investigations identified characteristic features of DEF, and the maximum temperature reached at the centre of this cap beam was 69 °C. The calculation shows also that the cooling of the cap beam was slow and that its concrete temperature remains greater than 60 °C for 44 hours.

The Cheviré Bridge

The Cheviré bridge was opened to traffic in May 1991 and is classified as an exceptional bridge (Fig. 14). It comprises two access viaducts composed of continuous box girders made

of prestressed concrete having respectively a length of 603 m for the north viaduct and 798 m for the south. The central span has a length of 242 m and includes an independent steel box girder 164 m long, supported by cantilever prestressed concrete box girder. The total length of the bridge is 1562 m. The North viaduct is supporting by one abutment and nine piers, and the South one is supporting by one abutment and twelve piers. The hollow piers have a variable height (from 5.2 to 43.5 m) and comprise concrete bases at the bottom. The dimensions of these bases are: 3.15 x 3.90 x 4.00 m.

In 1998, map cracking was observed on the facings of some bases of the South access viaduct (Fig. 15). DEF was identified by laboratory investigations. The rate of deterioration of the bases is variable: some bases presented only some vertical cracks with a width lower than 0.3mm while others exhibited more or less developed map cracking (up to a width of 0.6 mm). The phenomenon is globally slow and the rate of opening of the cracks was 0.1 mm/m/year, from 2003 to 2006.

Figure 14: General view of the Cheviré Bridge. (Courtesy G. Forquet)

Figure 15: Cracking is more important in the bottom of the base because of the capillary suction (Cheviré bridge)

The searches in the records showed that the damaged bases were cast during the summer 1988. The cracked faces are exposed to rain and capillary rises. The concrete was made with a Portland cement incorporating calcareous fillers (cement II/A type), a marine sand and gneiss aggregates.

The strategy for prevention

The French recommendations for the prevention of damage due to DEF (LCPC 2007) were established from data available in the scientific literature and from the detailed studies of damaged structures, like the six cases presented above. From this analysis, a strategy was developed based on categorising the structure according to the level of risk of occurrence of cracking that can be accepted, and related to the environmental conditions to which the structure is exposed during its service life. This strategy (Godart 2012) defines four levels of prevention with precautions that have to be applied related to risk and environmental categories. The precautions are mainly based on the limitation of the maximum temperature reached within the centre of the concrete pour during hardening of the concrete, and on the choice of an appropriate composition of the concrete.

The structures (or parts of them) are classified in 3 categories that are representative of the level of risk of cracking with respect to DEF that are acceptable for a given structure (or a part of it). The choice of the structure category is a function of the nature of the structure, its

purpose, the consequences of the cracking in relation with the desired safety level, and its future maintenance: Category I refers to the structures for which the consequences of the occurrence of cracking are low or acceptable (for example: temporary structures), Category II gathers the structures for which the consequences of the occurrence of cracking would be detrimental (like resisting elements of most buildings and civil engineering structures), and Category III corresponds to structures for which the consequences of the occurrence of cracking would be unacceptable (like nuclear containment, dams or tunnels).

Table 2: Comparative study of factors encountered in the investigated bridges.

	Ondes	**Bourgogne**	**Lodève**	**Bellevue**	**Beynost**	**Cheviré**
Date of construction	1955	1990	1980	1988	1982	1988/89
Casting of concrete	August	August/ September	August/ September	August/ September	August	July/ August
Structural part	Cap beam	Base of pylon	Cap beam	Pier	Cap beam	Base of pier
Delay in occurrence of damage	27 yrs	6 yrs	9 yrs	10 yrs	10 yrs	8 yrs
Environ-ment	Waterproofing Problem	Immersed & variable immersion	Lack of drainage	Immersed & variable immersion	Exposure to rains	Rains and capillarity
T max (°C)	80	79	80	80	69	75
W/C ratio	0.50	0.45	0.47	0.54	0.49	0.48
Nature of cement	CEM I	CEM I	CEM I	CEM II/A	CEM I	CEM II/A
Cement con-tent (kg/m^3)	430	400	400	380	350	385
SO_3 cement (%)	2,5	2,8	2,6	2,5	3,4	2,5
C3A cement (%)	11,2	8,2	9,8	7,0	10,4	7,0

Three exposure classes take into account the fact that water or a high ambient humidity are factors necessary for the development of DEF: the class XH1 concerns a dry or moderate humidity, the class XH2 is related with alternation of humidity and drying or with a high humidity, and the class XH3 considers structures in constant with water (permanent immersion, water stagnation on the surface, tidal zone,...).

Then four levels of prevention are determined according to the risk category of the structure and the exposure class XH to which the considered part of the structure is subjected. The choice of these levels of prevention indicated by the letters As, Bs, Cs and Ds, is the responsibility of the structure owner in setting the design specification. Table 3 is helpful when making this selection.

The principle of prevention relies primarily on the limitation of the heating of the concrete characterised by the maximum temperature, Tmax, likely to be reached within the structure and, also, by the duration of the period for which a high temperature is maintained. The limiting values of Tmax were defined by a group of expert on the basis of a literature review,

a comparative study of factors encountered in damaged and non damaged bridges (Table 2), several French research thesis, and some foreign recommendations such as (BRE, 2001).

Table 3 : Levels of prevention defined by structure risk category with exposure

Risk category of structure	Moisture/water exposure class of the structural part		
	XH1	XH2	XH3
I	As	As	As
II	As	Bs	Cs
III	As	Cs	Ds

The precautions corresponding to the four levels of prevention are the following:

- Level As: Tmax < 85 °C. However in the case of a heat treatment applied on a precast element, it is authorized to go beyond the temperature Tmax = 85 °C up to 90 °C, provided that the length of the period during which the temperature exceeds 85°C is limited to 4 hours.
- Level Bs: Tmax < 75 °C. However if the maximum temperature reached in the concrete cannot remain lower than 75°C, then it must remain lower than 85 °C and at least one supplementary condition must be respected.
- Level Cs: Tmax < 70 °C. However if the maximum temperature reached in the concrete cannot remain lower than 70°C, then it must remain lower than 80 °C and at least one supplementary condition must be respected.
- Level Ds: Tmax < 65 °C. But if Tmax cannot remain lower than 65 °C, then it must remain lower than 75°C with the use of mineral additions and the validation of the concrete composition by an independent laboratory expert in DEF.

The supplementary condition should be selected from the following:

- Equivalent active alkalis of the concrete < 3 kg/m^3
- Use of a cement conforming to the standard NF P 15-319 (ES) for sulphate resisting cements, and some additional conditions (see LCPC 2007)
- Use of cements non conforming to the standard NF P 15-319 of the type CEM II/B-V or CEM II/B-S or CEM II/B-Q or CEM II/B (S-V) or CEM III/A or CEM V, with SO_3 of cement < 3 % and C_3A of the clinker < 8 %
- Use of fly ashes, slags, calcinated natural pozzolans or metakaolin in combination with a CEM I, with additions content > 20 %, and with SO_3 of cement < 3 % and C_3A of the clinker < 8 %
- Application of a performance test on the concrete mix according to (LCPC 2007)
- For precast elements, the concrete mix and heating cycle identical or similar to that having at least 5 references of use in similar conditions without any problem.

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