

TECHNICS AND METHODS

Recommendations for preventing disorders due to Delayed Ettringite Formation



technical guide

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TECHNICS AND METHODS

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Viaduct on the new coast road in La Réunion seen from the sea
Credit: Sébastien Marchal, Viaduc Littoral photo library

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Introduction

Delayed Ettringite Formation (DEF) is a cause of disorders that can cause quite severe damage in concrete structures. While only a few cases have been reported in France, they are still significant enough to justify taking certain precautions for new construction. The recommendations presented in this document are intended to minimise the risk of disorders resulting from this phenomenon.

These recommendations pertain to engineering structures and buildings containing large-sized structural elements that are in contact with water or exposed to a moist environment. They serve to target the desired level of prevention on the basis of both the category of structure (or part of structure) and the environmental conditions affecting the structure. Each of these levels of prevention is associated with precautions to be applied as well as verifications to be performed. These recommendations also set forth a number of detailing procedures to be implemented.

The first cases of delayed ettringite formation appeared outside of France beginning in 1987 in certain pre-cast components that had been subjected to a heat treatment that was not appropriate for the type of concrete mix design and exposure (e.g. railroad ties). Both the mix designs and heat treatment cycles were therefore modified so as to avoid the occurrence of new disorders; since that time, such adaptations have been formalised within the standard relative to precast products NF EN 13369. Disorders caused by delayed ettringite formation were first observed in France in 1997 on bridges whose concrete had been cast in place; more massive structural elements were primarily involved in such observations (e.g. piers, crossbeams on piers or abutments, tower bases), either in contact with water or exposed to a high moisture level. Since then, such disorders have been observed in PRAD type pre-cast bridge beams manufactured before the publication of the LCPC recommendations of 2007 for the prevention of disorders due to DEF.

According to our current state of knowledge, only a few research results have actually been published on the treatment of structures affected by delayed ettringite formation and moreover no treatment solution has been proposed to halt the progression of this reaction within structures. Given this situation, a preventive approach proves essential in seeking to avoid the occurrence and development of this phenomenon in structures to be built.

These recommendations supplement the conventional state-of-the-art guidelines that address both concrete material quality and durability within structures; more specifically, these guidelines are complementary to Standard NF EN 206/CN, pre-cast concrete product standards and the entire series of NF EN 1992 standards. They offer adapted solutions in the event of contradictions raised with some of the currently-applicable rules and regulations; such is especially the case with recommendations relative to hardened concrete frost resistance prescribing the use of high contents of type CEM I cement in order to resist spalling, as this condition may be detrimental as regards durability with regard to delayed ettringite formation. In addition, this set of recommendations reinforces some of the basic notions that, on occasion, tend to be overlooked or even forgotten in the building process.

a) For structures cast in place


- Compressed construction schedules, the attainment of high concrete strength at early age in order to accelerate formwork turnaround or the early introduction of prestressing forces, particularly in the case of massive elements, must not come at the expense of the durability of structures built under these conditions (see Appendix 3).
- It is necessary to comply with the principle of selecting a concrete mix design that is well adapted to the part of the structure in question within its specific environment. This principle extends to adopting a multicriteria optimisation approach when choosing both the cement and the concrete mix, avoiding, for example, the use of high proportions of highly-exothermic CEM I cements within a massive structural element.
- The casting of massive elements must be avoided during very hot periods when no specific conditions have been adopted to limit excessive concrete heating.

b) For structures composed of precast concrete elements

- For these elements, it should be avoided all heat treatment cycles that involve excessive maximum temperatures in conjunction with an excessive constant temperature plateau for concrete mix designs that are sensitive to DEF.

The present recommendations combine a performance-based approach with specification of means, this latter being mandatory when performance-oriented specifications cannot be applied.

Lastly, these recommendations update and replace the previous ones of 2007, on the basis of knowledge that has been gained since then.

The background of the slide is a micrograph showing a material with a central vertical crack. The crack is filled with a dark, layered substance, possibly a repair or a different phase. The surrounding material is light-colored and contains numerous small, dark, angular particles, likely inclusions or precipitates. The overall texture is granular and somewhat porous.

Chapter 1
General remarks
on Delayed Ettringite
Formation

Photo on page 7

A layer of delayed ettringite formed at the aggregate/cement paste interface

Credit: Béatrice Desrues, Ifsttar

1.1 The Delayed Ettringite Formation phenomenon

This internal sulphate reaction may be characterised as the delayed formation of ettringite within a hardened cementitious material several months or even years after the cement has set and without the inflow of any external sulphate. The term delayed (see Appendix 1) used in this context indicates that the ettringite was not able to form during cement hydration, due to a significant heating of the concrete taking place a few hours or days after casting.

A distinction must be drawn between delayed ettringite formation (DEF) and the external sulphate reaction (ESR), which was first discovered in 1887 and detected by Candlot subsequent to observations of mortars in Paris's fortifications when in contact with gypsum water. In the case of ESR, sulphate may be provided by soils and de-icing salts or be conveyed by groundwater, infiltration water, seawater or water effluent from industrial sites. Sulphates penetrate via the concrete capillary network and can cause so-called "secondary" ettringite formation (see Appendix 1), which in turn could lead to expansion phenomena and hence concrete deterioration. As regards ESR therefore, deterioration starts gradually from the surface and progresses towards the core of the concrete element.

In some instances, when concrete undergoes heating at an early age, delayed ettringite formation (DEF) may appear without any sulphate input from an external source. The international term ascribed to this condition is Delayed Ettringite Formation (or DEF for short). Such a phenomenon can arise for two types of concrete: heat-treated concretes, and cast-in-place concretes in critical structural elements, i.e. concrete elements whose heat release is only marginally discharged to the exterior, which leads to a sizeable rise in concrete temperature (see Appendix 2).

A large majority of internally-generated sulphate ions originate in the cement and, under certain conditions, can be dissolved in the concrete pore solution. The sulphate reaction involves these ions present in the pore solution along with the cement aluminates, potentially leading to the formation of ettringite capable of causing expansion inside the hardened concrete. The internal sulphate swelling phenomenon can be detected by the occurrence at the concrete surface of multidirectional cracking with a relatively wide mesh spacing of between 10 and 30 cm.

No specific symptoms indicate this delayed ettringite formation since other structural pathologies, in particular alkali-aggregate reactions, also display the same signs.

Up until now, it has been observed that concrete elements affected by this pathology:

- Are so-called critical elements, i.e. concrete elements whose heat release is only marginally discharged to the outside, leading to a significant concrete temperature rise (see Appendix 2).
- Were cast during summertime and exposed to a temperature estimated to be above 80 °C at the core during the concrete hardening period.
- Were subjected to a moist environment for a number of years.

The elements in question may also be pre-cast elements that had undergone a thermal treatment at very high heat.

1.2 The main parameters associated with delayed ettringite formation

The causes, physicochemical mechanisms and kinetics of the reaction that gives rise to the DEF swelling phenomenon, as well as the impact of the various parameters affecting DEF, are not yet thoroughly understood and continue to be the topic of widespread research. It appears however that a combination of several parameters is essential to initiating and extending the DEF, and this probably explains the low number of structures currently identified as experiencing DEF. The principal parameters involved are water, temperature and its duration of application, the sulphate and aluminate contents of the cement, and lastly the alkali content of the concrete:

Water and humidity: one longstanding observation, both in the laboratory and on actual structures, involves the fundamental role played by water as the reaction develops. Water is a reactive medium essential to producing the reaction; it is as much involved in the transfer process as in the actual formation of reaction products. DEF primarily affects the parts of structures either in contact with water (submerged zone, tidal zone) or subjected to water ingress (exposure to bad weather, waterproofing defects, absence of drainage, etc.), or perhaps exposed to a high moisture level.

Temperature and its duration of application: the maximum temperature reached as well as the amount of time high temperature is maintained both influence the risk of delayed ettringite formation. Laboratory work has shown that if the temperature exceeds 65 °C and if other key parameters are present, a situation generally occurs in which DEF is able to develop. Intense heating of the concrete during setting and hardening is a necessary precondition, yet on its own remains insufficient.

Sulphate and aluminate contents of the cement: sulphates and aluminates are directly involved in the reactive mechanism that serves to form ettringite, which is composed of a hydrated calcium trisulfoaluminate. Consequently, DEF can only arise if the cement used contains a high enough quantity of both tricalcic aluminates ($3\text{CaO Al}_2\text{O}_3$ or C_3A) and sulphates (SO_3).

Alkali content of the concrete: the role of this on ettringite solubility is well documented. Ettringite is more highly soluble at higher alkali rates. As a result of ettringite solubility variation with temperature, a strong interaction exists between these two parameters during the DEF process. All other parameters being the same, a drop in the initial alkali content serves to increase the critical temperature value.

1.3 The current standardization environment

The standard NF EN 206/CN “Béton: spécifications, performances, production et conformité” (Concrete: specifications, performances, production and conformity) establishes a set of compositional specifications for concrete which depend on the level of exposure the structure or part thereof will sustain during its period of use. These specifications are intended to guarantee structural durability with respect to certain types of physicochemical aggression, in particular the external sulphate reaction. This standard refers to these recommendations and provides no other specific response concerning the prevention of DEF.

On the other hand, the Standard EN 13670/CN, entitled “Execution of concrete structures” indicates that the peak temperature of the concrete within an element must not exceed 70 °C, unless data prove that with the particular combination of materials used, greater temperatures would exert no significant adverse effect on the concrete’s in-service performance. This standard also refers to these recommendations.

In the area of pre-casting, the Standard NF EN 13369 “Règles communes pour les produits préfabriqués en béton” (Common rules for precast products) acknowledges DEF by referring to these recommendations and provides a set of environment-based recommendations (i.e. dry vs. wet) that focus on:

- either the maximum temperature of heat treatment
- or a concrete durability experiment
- or maximum contents of both cement sulphate and concrete alkali

Delayed ettringite formation is also taken into account in a number of standards specific to pre-cast products such as NF EN 13230-1 “Applications ferroviaires – Voie – Traverses et supports en béton” (Railway applications - Track - Concrete sleepers and bearers).

1.4 Influence of concrete constituents and composition

Each constituent of the concrete must be rigorously selected. In particular, for some levels of prevention the equivalent active alkali content of the concrete must be measured as detailed in Chapter 3. This alkali evaluation should be made as specified in the documentation fascicle FD P 18-464.

1.4.1 The cements

The choice of cement is important. For example, the cement used should be covered by the standard NF EN 197-1. Moreover, some intrinsic characteristics must be considered, as detailed in the precautions set out in Chapter 3.

In addition, the heat of hydration measured by means of Langavant calorimetry (the Standard NF EN 196-9) must be characterised by a temperature rise kinetic and the hydration heat release at infinite time.

Some types of cement generate less hydration heat, for example CEM II type compound Portland cements, CEM III type blast furnace cement, CEM IV type pozzolanic cements, CEM V type compound cements or supersulphated cements (SSC) (NF EN 15743).

1.4.2 Mineral additions

Mineral additions can also be used in addition to or as substitutes for CEM I and CEM II type cements in order to make the concrete less exothermic. Moreover, the resistance of concrete to DEF improves when constituents with latent hydraulic or pozzolanic properties are used (blast furnace slag or fly ash, natural or artificial pozzolans). This is due in particular to a relative decrease in the quantity of aluminates from clinker and modifications to hydrate type and texture.

1.4.3 Aggregates

It is advisable to verify the alkali content of the aggregate as mentioned in the documentation fascicle FD P18-464 “Concrete – Measures to prevent the alkali-aggregate reaction”. In addition, the petrographic nature and volumic mass of the aggregate influence heat transfers.

1.4.4 Admixtures

Admixtures can also play a role in the heat release kinetic.

1.4.5 Mix design

Mix design work remains essential in order to limit delayed ettringite formation.

Chapter 2

Determination of the required level of prevention



Photo on page 13

Piers undergoing construction for the Puento del Atlantico in Panama

Credit: the photo library of Vinci and its subsidiaries

The purpose of these recommendations is to provide precautionary measures for the placement and design of a concrete, so as to better contain the risks of disorders due to delayed ettringite formation (DEF) throughout the lifetime of the structure.

The approach adopted consists of identifying those parts of structures most at risk of developing disorders due to DEF. These are mainly what are deemed to be critical elements (according to the interpretation offered in Appendix 2), as well as pre-cast concrete products subjected to heat treatment. To proceed with this approach, cross-referencing is performed between the category describing the structure (or part thereof), as differentiated by the level of risk deemed to be acceptable, and environmental actions affecting the structure (or part thereof) over the life cycle of the structure.

This cross-referencing step serves to establish a level of prevention for these structural parts that in turn determines the set of precautionary measures to be implemented. Such measures depend to a large extent on the limitation of the maximum temperature reached at the core of the structural elements during concrete hardening and on the choice of a satisfactory concrete mix design.

2.1 Category of the given structure or structural part

The structures (or parts of structures) are classified into 3 categories representing the acceptable level of risk relative to delayed ettringite formation for that structure (or part of structure). The choice of structural category falls under the responsibility of the project owner; this decision depends on: the type of structure, its intended purpose, the consequences of disorders on the desired level of safety, and lastly on its future maintenance.

Category I includes structures (or parts of structures) for which the consequences of a potential disorder would be minor or acceptable. The majority of pre-cast concrete products fall into this category, with the notable exception of pre-cast structural elements, railway sleepers, and products intended for use in more aggressive environments (acoustic screens, bridge cornices, some types of drainage pipes, etc.).

Category II comprises structures (or parts of structures) for which the consequences of a potential disorder are little tolerable. The load-bearing elements of most buildings and civil engineering structures (including standard bridges) are placed in this category, as are pre-cast structural components.

Category III consists of structures (or parts of structures) for which the consequences of a potential disorder would be totally or almost totally unacceptable. Typically, this includes exceptional structures that may require a complete absence of disorders for safety or aesthetic reasons, or because their repair or replacement is not feasible. Additional precautions may be necessary for some of these structures.

Table 1
Examples of structures or structural elements classified by category

Category	Examples of structures or structural elements
Category I Minor or acceptable consequences	Structures made of concrete rated in a strength class of less than C 16/20 (including plugging concrete) Non load-bearing components of buildings Easily-replaced elements Temporary structures The majority of pre-cast non-structural products
Category II Little tolerable consequences	The load-bearing components of most buildings and civil engineering structures (including standard bridges) The majority of pre-cast structural products (including pressurised pipes)
Category III Unacceptable or almost unacceptable consequences	Buildings housing nuclear power plant reactors and cooling towers Dams Tunnels Exceptional bridges and viaducts Monuments and landmark buildings Railway sleepers

2.2 DEF exposure classes

Since Standard NF EN-206/CN does not define a specific exposure class for delayed ettringite formation, three complementary classes XH1, XH2 and XH3 will be introduced into the present document to address this case. These additional classes acknowledge the fact that the presence of water or high ambient relative humidity is essential for the development of DEF. Inflows of alkalis and sulphates from the surrounding environment also exacerbate disorders, however, these are considered to be part of a surface degradation process and are governed by preventive procedures covered elsewhere (e.g. in the Standard NF EN 206/CN).

The three exposure classes XH1, XH2 and XH3 are defined in accordance with the indications listed in table 2, which also displays for information purposes a number of examples of parts of structures classified within their appropriate ambient environments.

Exposure classes XH1 through XH3 serve to supplement the 18 classes already * This percentage is calculated in relation to the combination of cement and mineral addition established in the Standard NF EN 206/CN and must be mentioned in the Specific Technical Clauses (of the Contract) for each part of a structure. In principle, no direct correlation is possible between the 3 classes XH1-XH3 and the 18 exposure classes set forth in the Standard NF EN-206/CN; it can still be considered however that parts of structures classified as XC4 fall into class XH2. The guide published by the EFB (École française du béton) contains a table that assists the choice of an XH class for all types of structure.

For a given structure (or structural part), the specifications listed in the present document must be incorporated as a complement to those imposed by the Standard NF EN 206/CN.

Table 2
Exposure classes for a part of a structure with respect to DEF

Exposure class	Environment	Examples to illustrate the choice of exposure class
XH1	Dry or moderately moist	Part of a concrete structure located inside buildings where the humidity rate of the ambient air is either low or medium Part of a concrete structure located outside and protected from rain
XH2	Alternately moist and dry, high moisture level	Part of a concrete structure located inside buildings where the humidity rate of the ambient air is high Part of a concrete structure unprotected by a coating and exposed to inclement weather, without any water stagnation at the surface Part of a concrete structure unprotected by a coating and frequently subjected to condensation
XH3	In long-lasting contact with water: state of permanent water stagnation at the surface, tidal zone	Part of a concrete structure permanently submerged in water Maritime structural elements A large number of foundations Part of a concrete structure regularly exposed to water projections

2.3 Levels of prevention

Four levels of prevention have been established, as designated respectively by the letter codes As, Bs, Cs and Ds. A level of prevention is determined on the basis of both the structure category and XH exposure class applicable to the particular part of the structure. The level of prevention can be identified by considering the entire structure, yet it is advised to assess each part of the structure individually in order to derive a specifically adapted level of prevention. Responsibility for selecting these various levels of prevention lies with the project owner, for whom table 3 provides some guidance.

For purposes of illustration, let us consider a bridge rated in category II: both the piles and foundation footings qualify for a level of prevention Cs, whereas the piers and deck require Bs level protection; as for the crossbeams on piers and abutments, the choice of level of prevention will be made by evaluating the conditions adopted to ensure water drainage on these structural parts: the level of prevention will thus be set at Bs or Cs depending on the risk of water stagnation.

Table 3
Choice of level of prevention

Structure category	Exposure class of the part of the structure		
	XH1	XH2	XH3
I	As	As	As
II	As	Bs	Cs
III	As	Cs	Ds



Chapter 3

Precautions adopted based on level of prevention



Photo on page 19

Sinking the pier of the viaduct on the new coastal road in La Réunion

Credit: Viaduc Littoral photo library, Sébastien Marchal

Each of the four levels of prevention As, Bs, Cs and Ds corresponds with types of precaution to be implemented. The prevention principle basically relies on limiting concrete heating, as characterised by the maximum temperature (T_{\max}) that may be reached inside the concrete of the part of the structure and, if applicable, by the length of time a high temperature is maintained.

To be able to estimate the maximum temperature that may be reached in the core of an element, Appendix 3 presents a simple estimation method that uses only a limited number of basic data, such as element thickness (over its smallest dimension) and a few data regarding concrete composition (proportioning, 2 days and 28 days compressive strength of the cement, hydration heat of the cement at 41 hours, or preferably 120 hours, etc.) and the initial temperature of the fresh concrete. A more accurate method for estimating maximum temperature consists of running a finite element computation code and including the heat released by concrete determined by a specific test.

In order to avoid any excessive and uncontrolled increase in heat in the concrete, all potential approaches should be implemented (choice of mix design and concrete constituents, choice of the period for concreting, cooling of fresh concrete, appropriate construction procedures, etc.) in order to lower and control the temperature of the concrete during placement and then over the first few days thereafter.

In all cases, a concrete mix design must meet the specifications contained in the standards and recommendations currently in effect. However, it may be extremely difficult, or even impossible, to reconcile the requirements imposed due to durability * This percentage is calculated in relation to the combination of cement and mineral addition. considerations with the present set of recommendations, especially as regards the type and minimum proportion of the binder. As an example, the “G + S” specification from LCPC's “recommendations for the durability of hardened concrete subjected to frost” stipulates a minimum cement content of 385 kg/m^3 (for a D_{\max} value of 20 mm) and significantly limits the possibility of incorporating additions or using composite cements, which could lead to considerable heating in the case of massive structural elements. Since superimposing the various requirements does not always prove appropriate, it is necessary to conduct a special study at the project design stage in order to lay down realistic requirements to be included in the specifications. Such a study might lead to the adoption of a performance-based approach to durability geared to the targeted exposure class and based on a series of recognised tests, the modification of building procedures, or a revision of the project.

Regarding frost and de-icing salts, these performance-oriented tests consist of the tests detailed in the standards NF P18-424 or NF P18-425 (for “severe” or “moderate” frost, respectively) and/or the spalling test described in XP P 18-420. The reader's attention is drawn to the time periods necessary to produce such justifications (at least 3 months).

3.1 Level of prevention: As

For this level of prevention, the risk of disorders due to delayed ettringite formation must be taken into account by means of one of the two following precautions, referred to as 1 and 2:

- 1) The maximum temperature reached within the concrete must remain below $85 \text{ }^\circ\text{C}$.
- 2) If the maximum temperature in the concrete cannot remain below $85 \text{ }^\circ\text{C}$, it must remain below $90 \text{ }^\circ\text{C}$ and the following condition must be met:
 - The heat treatment must be controlled (note 1) and the temperature of the concrete must not exceed $85 \text{ }^\circ\text{C}$ for more than 4 hours.

Note 1: Controlled heat treatment may be applied in a pre-casting plant or in appropriate facilities available at the worksite. It must be accompanied by periodic monitoring involving temperature measurements and testing as part of a quality plan.

3.2 Level of prevention: Bs

For this level of prevention, the risk of disorders due to delayed ettringite formation must be taken into account by means of one of the two following precautions, referred to as 1 and 2:

- 1) The maximum temperature within the concrete must remain below 75 °C.
- 2) If the maximum temperature within the concrete cannot be kept below 75 °C, it must never exceed 85 °C and at least one of the six following conditions must be satisfied:
 - If the heat treatment is controlled (note 1), the temperature of the concrete must not remain above 75 °C for more than 4 hours, and the equivalent active alkali content of the concrete must be less than 3 kg/m³.
 - The use of cement which complies with the standard NF P15-319 (ES) or an SR cement as specified in the standard NF EN 197-1 except for CEM I SR3 and SR5 cements, for concretes which are expected to be subjected to a temperature in excess of 75 °C for more than 10 hours. Use of SR3 and SR5 cements is permitted for concretes when the temperature will remain above 75 °C for less than 10 hours, on condition that the equivalent active alkali content of the concrete is below 3 kg/m³.
 - The use of a CEM I SR3 or SR5 cement which complies with the NF Liants hydrauliques (French Standard Hydraulic Binder) marking and which has been characterised using the methodology set out in Appendix 5, in the case when the temperature remains above 75 °C for more than 10 hours.
 - The use of cements which do not comply with the standard NF P 15-319 (ES) of the types CEM II/B-V, CEM II/B-Q, CEM II/B-M (S-V) on condition they contain more than 20 % of fly ash, CEM III/A or CEM V cements, all of which should have an SO₃ content which does not exceed 3 %, and the clinker introduced during cement manufacture must not contain more than 8 % of C₃A.
 - The use, in combination with CEM I or CEM II/A, of fly ash compliant with Standard NF EN 450-1, ground blast-furnace slag compliant with Standard NF EN 15167-1, silica fume compliant with NF EN 13263-1 or metakaolins compliant with NF P18-513. The proportions of these in the binder (the cement and additive combination) must not be less than 20 % for fly ash, 35 % for blast furnace slag, 10 % for silica fume (note 2) and 20 % for metakaolin. The binder employed must meet the following requirements: C₃A (as a proportion of the clinker) ≤ 8 % and SO₃ (as a proportion of the binder) ≤ 3 %.
 - Verification of concrete durability with respect to DEF by relying upon performance testing and by satisfying a number of decision-making criteria.

Note 1: Controlled heat treatment may be applied in a pre-casting plant or in appropriate facilities available at the worksite. It must be accompanied by periodic monitoring involving temperature measurements and testing as part of a quality plan.

Note 2: In the case of silica fume this limit may be reduced to 5 % as long as the binder contains at least 15 % of fly ash.

3.3 Level of prevention: Cs

For this level of prevention, the risk of disorders due to delayed ettringite formation must be taken into account by means of one of the two following precautions, referred to as 1 and 2:

- 1) The maximum temperature within the concrete must remain below 70 °C.
- 2) If the maximum temperature within the concrete cannot be kept below 70 °C, it must never exceed 80 °C and at least one of the six following conditions must be satisfied:
 - If the heat treatment is controlled (note 1), the temperature of the concrete must not remain above 70 °C for more than 4 hours, and the equivalent active alkali content of the concrete must be less than 3 kg/m³.
 - The use of cement which complies with the standard NF P15-319 (ES) or an SR cement as specified in the standard NF EN 197-1 except for CEM I SR3 and SR5 cements, for concretes which are expected to be subjected to a temperature in excess of 70 °C for more than 10 hours. Use of SR3 and SR5 cements is permitted for concretes when the temperature will remain above 70 °C for less than 10 hours, on condition that the equivalent active alkali content of the concrete is below 3 kg/m³.
 - The use of a CEM I SR3 or SR5 cement which complies with the NF Liants hydrauliques (French Standard Hydraulic Binder) marking and which has been characterised using the methodology set out in Appendix 5, in the case when the temperature remains above 70 °C for more than 10 hours.
 - The use of cements which do not comply with the standard NF P 15-319 (ES) of the types CEM II/B-V, CEM II/B-Q, CEM II/B-M (S-V) on condition they contain more than 20 % of fly ash, CEM III/A or CEM V cements, all of which should have an SO₃ content which does not exceed 3 %, and the clinker introduced during cement manufacture must not contain more than 8 % of C₃A.
 - The use, in combination with CEM I or CEM II/A, of fly ash compliant with Standard NF EN 450-1, ground blast-furnace slag compliant with Standard NF EN 15167-1, silica fume compliant with NF EN 13263-1 or metakaolins compliant with NF P18-513. The proportions of these in the binder (the cement and additive combination) must not be less than 20 % for fly ash, 35 % for blast furnace slag, 10 % for silica fume (note 2) and 20 % for metakaolin. The binder employed must meet the following requirements: C₃A (as a proportion of the clinker) ≤ 8 % and SO₃ (as a proportion of the binder) ≤ 3 %.
 - Verification of concrete durability with respect to DEF by relying upon performance testing and by satisfying a number of decision-making criteria.

Note 1: Controlled heat treatment may be applied in a pre-casting plant or in appropriate facilities available at the worksite. It must be accompanied by periodic monitoring involving temperature measurements and testing as part of a quality plan.

Note 2: In the case of silica fume this limit may be reduced to 5 % as long as the binder contains at least 15 % of fly ash.

3.4 Level of prevention: Ds

For this level of prevention, the risk of disorders due to delayed ettringite formation must be taken into account by means of one of the two following precautions, referred to as 1 and 2, with the first being recommended as the priority precaution:

- 1) The maximum temperature within the concrete must remain below 65 °C.

2) If the maximum temperature within the concrete is unable to remain below 65 °C, then it must never exceed 75 °C and at least one of the five following conditions must be satisfied:

- The use of cement which complies with the standard NF P15-319 (ES) or an SR cement as specified in the standard NF EN 197-1 except for CEM I SR3 and SR5 cements, which are expected to be subjected to a temperature in excess of 65 °C for more than 10 hours. Use of SR3 and SR5 cements is permitted for concretes when the temperature will remain above 65 °C for less than 10 hours, on condition that the equivalent active alkali content of the concrete is below 3 kg/m³.
- The use of a CEM I SR3 or SR5 cement which complies with the NF Liants hydrauliques (French Standard Hydraulic Binder) marking and which has been characterised using the methodology set out in Appendix 5, in the case when the temperature remains above 65 °C for more than 10 hours.
- The use of cements which do not comply with the standard NF P 15-319 (ES) of the types CEM II/B-V, CEM II/B-Q, CEM II/B-M (S-V) on condition they contain more than 20 % of fly ash, CEM III/A or CEM V cements, all of which should have an SO₃ content which does not exceed 3 %, and the clinker introduced during cement manufacture must not contain more than 8 % of C₃A.
- The use, in combination with CEM I or CEM II/A, of fly ash compliant with Standard NF EN 450-1, ground blast-furnace slag compliant with Standard NF EN 15167-1, silica fume compliant with NF EN 13263-1 or metakaolins compliant with NF P18-513. The proportions of these in the binder (the cement and additive combination) must not be less than 20 % for fly ash, 35 % for blast furnace slag, 10 % for silica fume (note 1) and 20 % for metakaolin. The binder employed must meet the following requirements: C₃A (as a proportion of the clinker) ≤ 8 % and SO₃ (as a proportion of the binder) ≤ 3 %.
- Verification of concrete durability with respect to DEF based on performance testing and compliance with a number of decision-making criteria.

Note 1: In the case of silica fume this limit may be reduced to 5 % as long as the binder contains at least 15 % of fly ash.

For long duration construction works, irrespective of the level of prevention, every effort should be made to check that the conditions that ensure the non-reactivity of the concrete with respect to DEF are maintained.

3.5 Some comments on combinations of cement and mineral additions

3.5.1 Some comments on pozzolanic materials

Pozzolanic materials (fly ash, silica fume and metakaolin) have a beneficial effect by limiting, or even inhibiting, the development of DEF. Several hypotheses are available to explain the role played by such mineral additions: they dilute the proportions of C₃A and SO₃, reduce the amount of portlandite, reduce the alkalinity of the interstitial solution and modify the microstructure of the hydrates.

3.5.2 Comments about blast furnace slags

Slag substitution rates may be higher than those specified in the standard NF EN 206/CN without any need to apply the engineering concrete methodology. Under these conditions the amount of slag which exceeds the limit set by the standard NF EN 206/CN is simply not counted when calculating the concrete's equivalent binder.

3.5.3 Examples of combinations of cement and mineral additions

In order to make clear the different mixtures of cements and mineral additions that can be used as a means of inducing relaxation, six examples are given below, the last two of which involve combinations of CEM II/A + addition for which the CEM II/A contains only one main constituent apart from clinker, the main constituent being of the same nature as the addition used. In this last case, it is possible to subtract from the minimum recommended proportion of addition the amount of this constituent which is already present in the cement. This applies to the following combinations:

- **CEM II/A-S + blast furnace slag** according to the following calculation: percentage of slag to be added* = $0.35 - (0.65 \times x)/(1-x)$ where x is the percentage of slag in the CEM II/A-S.
- **CEM II/A-D + silica fume** according to the following calculation: percentage of silica fume to be added* = $0.1 - (0.9 \times x)/(1-x)$ where x is the percentage of silica fume in the CEM II/A-D.
- **CEM II/A-Q + metakaolin** according to the following calculation: percentage of metakaolin to be added* = $0.2 - (0.8 \times x)/(1-x)$ where x is the percentage of calcined clay in the CEM II/A-Q.
- **CEM II/A-V + fly ash** according to the following calculation: percentage of fly ash to be added* = $0.2 - (0.8 \times x)/(1-x)$ where x is the percentage of fly ash in the CEM II/A-V.

Example 1: 80 % of CEM I 52.5 R CE CP2 NF and 20 % of fly ash.

Example 2: 90 % CEM I 52.5 N SR3 CE PM-CP2 NF and 10 % of silica fume.

Example 3: 65 % of CEM II/A-LL 42.5 R CE CP2 NF and 35 % of blast furnace slag. For a C30/37 with a minimum equivalent binder content of 330 kg (exposure class XS1).

According to the table NA.F.1 in the standard NF EN 206/CN, $A/A+C \leq 0.2$ the equivalent binder content must be as follows:

- 269 kg of CEM II/A-LL 42.5 R CE CP2 NF and 68 kg of blast furnace slag (with $k = 0.9$).
- To which 77 kg of blast furnace slag must be added to attain the minimum of 35 % in the DEF recommendations but not considered in the equivalent binder.
- The binder therefore consists of 269 kg of CEM II/A-LL 42.5 R CE CP2 NF and 145 kg of blast furnace slag.

According to table NA.F.3 in the standard NF EN 206/CN (engineering concrete), $A/(A+C) \leq 0.35$ (if the limestone content of the CEM II/A-LL is below 15 %) the equivalent binder content must be as follows:

- 222 kg of CEM II/A-LL 42.5 R CE CP2 NF and 120 kg of blast furnace slag (if $k = 0.9$).

* This percentage is calculated in relation to the combination of cement and mineral addition.

Example 4: 80 % of CEM II/A-LL 42.5 R CE CP2 NF and 20 % of fly ash
For a C30/37 with a minimum equivalent binder content of 330 kg.

According to table NA.F.1 in the standard NF EN 206/CN, $A/(A+C) \leq 0.2$.

The equivalent binder content must be as follows:

- 287 kg of CEM II/A-LL 42.5 R CE CP2 NF and 72 kg of fly ash (if $k = 0.6$).

Example 5: CEM II/A-S 52.5 N CE PM-CP2 NF (with 15 % of slag) and blast furnace slag.

The (cement + slag) combination must contain at least 35 % of slag; as the CEM II/A-S is deemed to already contain $x = 15$ % of slag, the mixture must include as a minimum: percentage of slag to be added = $0.35 - (0.65 \times x)/(1-x) = 0.35 - (0.65 \times 0.15)/(1 - 0.15) = 0.235$ i.e. 23.5 %.

It must be added 24 % of blast furnace slag as addition, i.e. for example 76 % of CEM II/A-S 52.5 N CE PM-CP2 NF (with 15 % of slag) and 24 % of blast furnace slag.

For a C30/37 with a minimum equivalent binder content of 330 kg ; according to the table NAF.1 in the standard NF EN 206/CN, $A/(A + C) \leq 0.2$.

The equivalent binder content must be as follows:

- 269 kg of CEM II/A-S 52.5 N CE PM-CP2 NF and 68 kg of blast furnace slag (where $k = 0.9$).
- To which must be added $((24 \times 269) / 76) - 68 = 17$ kg of blast furnace slag to comply with the 35 % minimum amount to attain the minimum of 35 % in the DEF recommendations but not considered in the equivalent binder.
- The binder is therefore made up of 269 kg of CEM II/A-S 52.5 N CE PM-CP2 NF and 85 kg of blast furnace slag.

According to table NA.F.3 in the standard NF EN 206/CN (engineering concrete), $A/(A+C) \leq 0.35$.

The following equivalent binder content must be obtained:

- 259 kg of CEM II/A-S 52.5 N CE PM-CP2 NF and 80 kg of blast furnace slag (where $k = 0.9$).

Example 6: CEM II/A-V 42.5 N CE CP2 NF (with 13 % of fly ash) and fly ash.

The (cement + fly ash) combination must contain at least 20 % of fly ash; as the CEM II/A-V is deemed to already contain $x = 13$ % of fly ash, the minimum percentage of fly ash to be added to the mixture = $0.20 - (0.80 \times x)/(1-x) = 0.20 - (0.80 \times 0.13)/(1 - 0.13) = 0.08$ i.e. 8 %.

It must be added 8 % of fly ash as an addition, i.e. for example 92 % of CEM II/A-V 42.5 N CE CP2 NF (with 13 % of fly ash) and 8 % of fly ash.

For a C30/37 with a minimum equivalent binder content of 330 kg.

According to table NA.F.1 in the standard NF EN 206/CN, $A/(A+C) \leq 0.2$.

The equivalent binder content must be as follows if 8 % of fly ash is added:

- 314 kg of CEM II/A-V 42.5 N CE CP2 NF and 27 kg of fly ash (where $k = 0.6$).

Chapter 4

Provisions related
to design and calculation
of structures,
concrete mix design
and fabrication,
as well as placement

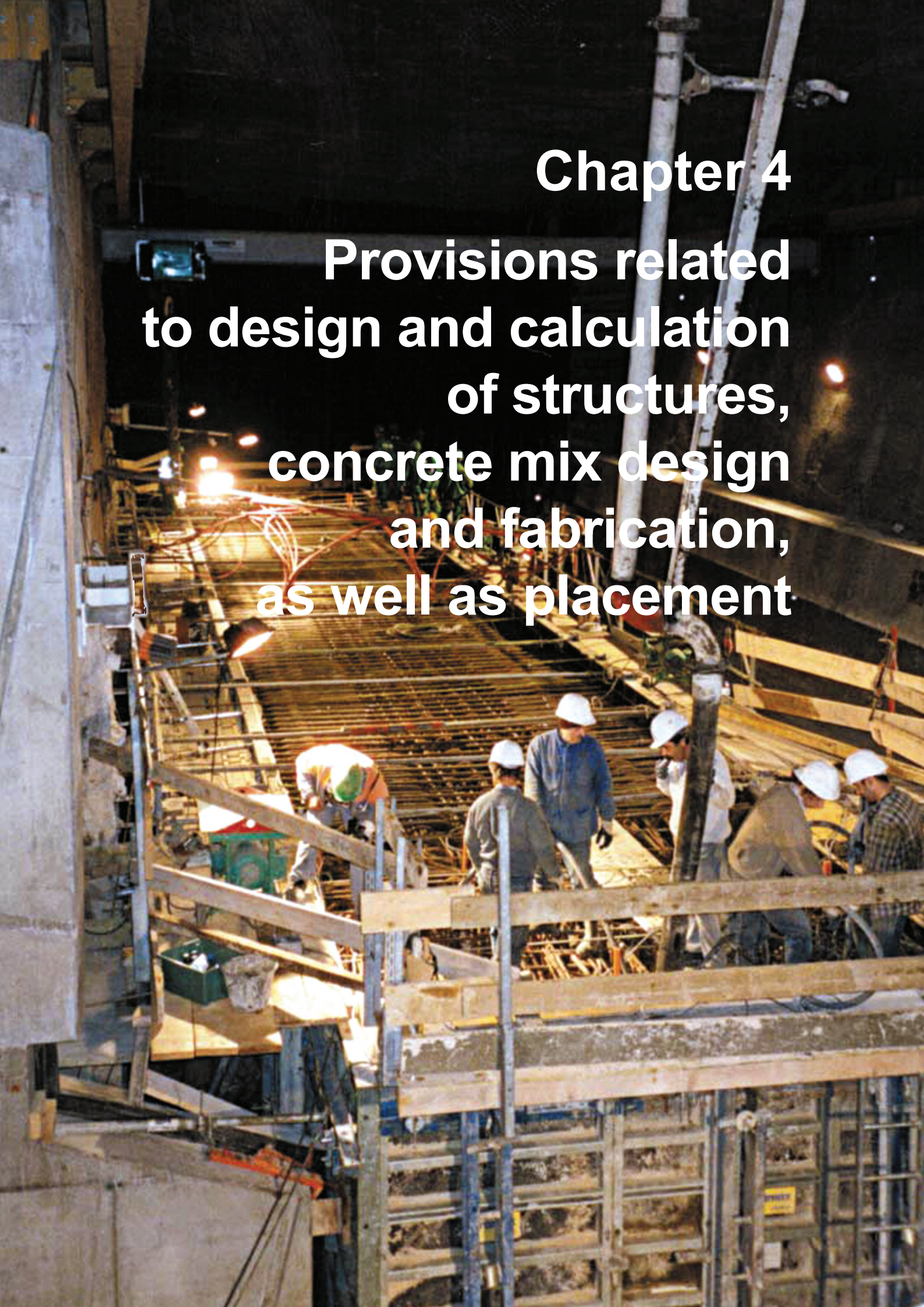


Photo on page 27

Night-time concreting with cooling of a beam on the Pont d'Aquitaine

Credit: VINCI Construction

4.1 General remarks

The detailed provisions set forth in the present chapter are intended to:

- avoid extended contact of the critical part with water throughout the period of use of the structure
- limit the maximum temperature reached inside the concrete contained in critical parts
- control the heat treatments applied to pre-cast elements

4.2 Provisions related to the design and calculation of structures

4.2.1 Avoiding extended contact with water

The structure must be designed so as to avoid, as far as possible, water accumulation and stagnation zones as well as preferential flow paths due to runoff; this means it is necessary to provide slope profiles and shapes that allow for rapid water drainage.

It is also possible to implement the measures required to avoid the penetration of water and humidity into concrete structures either:

- by ensuring waterproofing of the critical part
- or by waterproofing the structural components that serve to protect the critical part and by implementing the appropriate drainage measures

This is particularly the case with bridge decks where it is necessary to install a waterproofing membrane* and water drainage systems that are effective and maintained on a regular basis. Installing an appropriate waterproofing system (in the form of a membrane) may make it possible to classify the structure or structural part in category XH1, however it must be born in mind that the durability of this waterproofing system imposes a regular replacement schedule.

Among the other surfacing/coating materials capable of limiting humidity and/or water penetration, the most widespread are protective coatings (paints, thin coatings, impregnation techniques, etc.) (See the LCPC Guide entitled “Protection of concretes”, the standard NF P 95-103 and the application guide GA P 18-902). The application of paint provides little protection from the effects of DEF and is thus not recommended. The placement of a thicker concrete protective coating (up to several millimetres) offers one means of protection, provided that the systems employed are sufficiently sealed (to water vapour as well). This type of coating however only remains effective for a limited period (on the order of ten years), which means it will need to be replaced several times over the service life of the structure, which directs the choice of protection towards the kinds of prevention solutions presented in Chapter 3.

Applying a protective coating to the concrete thus provides a way of avoiding DEF by means of minimising water ingress. Such a solution may be employed to accompany a more reliable prevention solution, while not definitively ensuring DEF prevention.

* For the horizontal parts of road traffic-bearing bridge decks, the reference document is Fascicle 67 of the General Technical Clauses, Part I, to be complemented by the Technical Guideline issued by the S etra (now Cerema). Concerning the technical provisions to be implemented, the STER 81 Guide, also published by S etra, and its two most recent updates constitute the applicable documentation.

Note: The application of cladding can be used alongside a more reliable solution, yet does not in itself constitute an adequate means of prevention.

Moreover, within the scope of surveillance of structures, it is necessary to inspect the parts deemed to be critical so as to detect any cracks that appear and allow water to penetrate into the concrete. It is thus essential to treat these cracks with a flexible sealant that is able to still plug cracks even if they widen in the future. Various techniques are available, including bridging and injection.

For the other parts, the reader is advised to consult the LCPC Guide on concrete protection, the Standard NF P 95-103 and GA P 18-902, as well as a Sétra information note (CTOA No. 25): “Do not mistake a bridge deck waterproofing and a concrete protection”.

In the case of an underground critical part, the surrounding embankments must be fitted with drainage systems capable of channelling runoff water.

4.2.2 Limiting the temperature rise in concrete

It is recommended to avoid the existence of critical structural parts by optimising jointly the material used in the element and its design. Thus, the use of High Performance Concrete (HPC) can make it possible to produce slender structures with less sensitivity to the risk of DEF. From a general standpoint, it is advised to design structures that include hollow parts or, whenever possible, voided parts.

Millau Viaduct, pier P1, level 24, preparation for concreting

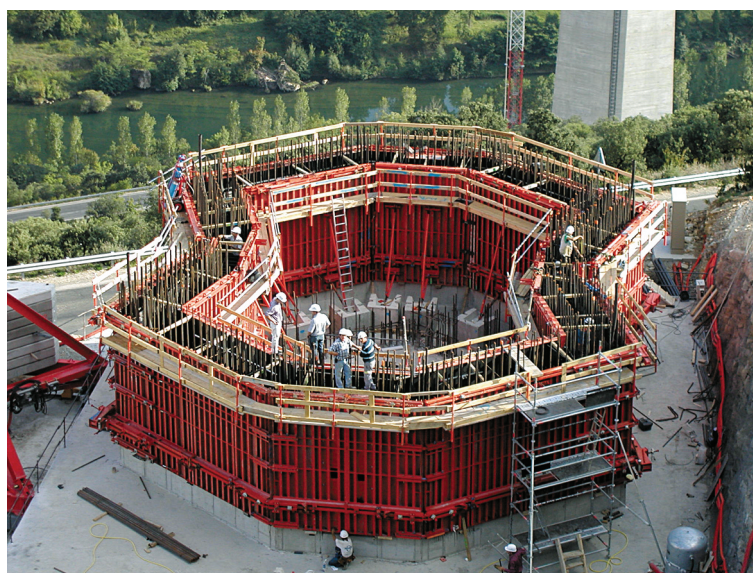


Photo credit: Eiffage Construction

As an example, the order of magnitude of the potential impact of using a hollow element on the maximum temperature reached is as follows:

- For the addition of an internal formwork to a pier (construction of a hollow pier instead of a solid pier, yielding an apparent thickness of 0.5 m instead of 3 m, with a concrete containing 350 kg/m³ of CEM I 52.5 N), the typical reduction in the maximum temperature is approximately 15 °C.

Piers of the Bridge over the Saint-Étienne river in La Réunion



Photo credit: Photo library of VINCI and its subsidiaries, Thierry Roux

4.3 Provisions related to concrete mix design

The choice of type of cement and a possible addition by the prescriber must incorporate the influence of these materials on concrete heating.

The selected binder must in fact be the least exothermic possible while remaining compatible both with specifications relating to the exposure classes and with the early-age concrete strength requirements (it is preferable to define early-age strength specifications with as few constraints as possible, and even to define no specification at all for potentially difficult concreting, yet such a decision must naturally be integrated into the overall project schedule). Similarly, binder content may be minimised while still meeting the set of workability, strength and durability requirements. The use of composite cements and/or incorporation of additions offer adapted solutions for minimising concrete exothermic reactions.

For purposes of illustration, the replacement of a concrete with CEM I 52.5 N cement by one containing CEM III 42.5 N cement in the mix design (on an element with an apparent thickness of 1 metre) is reflected by a drop in maximum temperature of approximately 15 °C.

4.4 Provisions related to concrete fabrication and transport

Before proceeding to an examination of the provisions for limiting concrete heating temperature, it would be worthwhile to review a key parameter involved in this type of heating: concrete heat capacity.

The heat capacity of concrete is defined as the quantity of heat needed to raise the temperature of a unit mass of concrete by 1 °C. This capacity depends on the heat capacity of each of the individual concrete constituents. The following table provides an order of magnitude of the heat capacities experienced when producing one cubic metre of concrete.

This table shows that due to their presence in large quantities, aggregates (coarse + fine) exert a considerable influence on the heat capacity of the mix. This means that a change in aggregate temperature will cause the most significant temperature change at the level of the entire mix. The above table also demonstrates that mixing water plays a major role in the ultimate mix temperature and that substituting this water by ice could trigger a sizeable decrease in the heat of the mix (in which case, it becomes necessary to consider the heat of fusion of ice, which is equal to 334 kJ/kg).

Table 4
Heat capacity of various concrete components

Constituent	Mass (kg)	Mass heat capacity (kJ / (kg . K))*	Constituent heat capacity (kJ / K)
Coarse limestone aggregate (dry)	1050	0.84	882
Siliceous limestone sand (dry)	750	0.80	600
Cement	400	0.75	300
Water	200	3.76	752
Total			2534

Cooling concrete with liquid nitrogen at the Puento del Atlantico worksite in Panama



Photo credit: Photo library of VINCI and its subsidiaries

* According to L. Divet - ERLPC OA 40 [1] – The heat capacity of water undergoing hydration in massive elements is 3.76 rather than 4.18 kJ / (kg . K) see NF EN 196-9.

The temperature of the concrete during laying must be controlled and can be lowered by several methods:

- using cold or refrigerated mixing water
- cooling the aggregate (spraying water on the coarse aggregates)
- protecting aggregate stockpiles from exposure to the sun
- substituting ice for a portion of the mixing water

The first two methods are relatively simple to employ even though they both require special customised equipment that is not typically available at ready-mix concrete plants. The use of ice is more complex and necessitates specific large-scale plant, which in particular demands extended mixing time to ensure complete melting.

The technique of injecting liquid nitrogen into the concrete mixing plant or mobile mixer may prove advantageous; it does not however enjoy widespread use due to its relatively high cost and technical sophistication.

As an example, the orders of magnitude for the potential impact of the various parameters cited above on the maximum temperature reached are:

- With cold mixing water (temperature of 4 °C instead of 20 °C), the reduction in the maximum temperature reached is approximately 3 °C.
- For spraying coarse aggregates with cold water (resulting in a 10 °C temperature drop), the reduction is also approximately 3 °C.

Note: It is necessary to take into account the temperature of the cement when it is delivered to the site. Cement that has just been manufactured in the factory can reach a high temperature (above 50 °C in some instances). As an example, a 10 °C temperature rise in the cement leads to a 1 °C temperature increase in the concrete.

The impacts of both material transport and mixer truck delays also need to be considered, for the purpose of minimising waiting times. Every effort must obviously also be made to limit the amount of time mixer trucks are parked in full sunlight.

4.5 Provisions related to concrete placement

4.5.1 Jobsite organisation

Ambient temperature conditions are difficult to control; more specifically, it is generally impossible to select the concreting season given the jobsite's overall scheduling constraints. It would still be preferable however to choose a time of day that helps minimise the temperature of fresh concrete (i.e. end of the day or at night).

Night-time concreting for a girder of the Pont d'Aquitaine with a water cooling circuit



Photo credit: VINCI Construction

For example, for night-time concreting (during the summer season), the order of magnitude of the potential impact of this parameter on the maximum temperature reached would be a temperature decrease of approximately 5 °C.

It is highly recommended, in the case of large-sized elements, not to attempt to do without internal formwork that is prescribed in order to retain just those sections that are essential for the strength of the structure (e.g. bridge piers). It is also advised to avoid creating solid parts when the design calls for hollow parts merely to facilitate construction.

For large-sized elements, it is possible to anticipate separating the concreting into several phases so as to promote heat exchange. This division is only effective if there is a long enough gap (at least a week) between successive castings. It is nonetheless advised to remain within acceptable time limits in order to preserve monolithic structural behaviour, position the concrete cast joints within appropriate zones from a mechanical perspective, and lastly comply with the rules governing the appropriate realisation of concrete cast joints.

4.5.2 Concrete cooling

It is also possible, as a complementary approach, to cool the concrete after setting by installing coils in the concrete, with such coils providing a cooling circuit through which cool water is passed. Such a cooling system must be the outcome of a design process to avoid the development of thermal gradients inside the concrete mass, especially in the vicinity of tubes, given that these gradients can produce radial or tangential cracks. Furthermore, the method is only truly effective when the concrete is only weakly or moderately exothermic. In the case of a highly exothermic composition, which is not optimal from the standpoint of thermal problems specific to the element being concreted, the (temperature-activated) heat release is much faster than the rate at which heat is removed by the cooling circuit. Lastly, cooling circuit installation interferes with reinforcement work and leads to longer construction times. Cooling in the concrete mass must be undertaken only as a last resort (it should also be noted that ultimately the pipes must be plugged with a cement grout).

Detail of a cooling system for mass concrete.
Construction of the Puentol del Atlantico in Panama



Photo credit: Photo library of VINCI and its subsidiaries

4.5.3 Choice of formwork

For medium-sized parts, formwork that favours heat exchanges may limit the maximum temperature reached within the concrete.

As an example, a 40 cm shear wall concreted in wooden formwork with a C40/50 class concrete and a CEM I 52.5 R content of 400 kg/m³ could lead to a 40 °C temperature rise. The use of metal formwork would reduce this increase to 35 °C.

Use of metal formwork to construct the piers of the viaduct on the new coastal road in La Réunion



Photo credit: Viaduc Littoral photo library, Sébastien Marchal

4.6 Specific provisions for precasting

The need to decrease the length of time during which the production means and to increase the number of daily production runs may lead to accelerate, via external heat input, the various chemical reactions involved in the concrete hardening process.

Concrete hardening is accelerated in order to provide the concrete with sufficient mechanical strength to perform, depending on the case, demoulding, handling, the release of prestressing forces or even appearance treatments.

As a general rule, both the heat treatment and means used for its application must be determined with reference to the geometry and dimensions of the element, the composition of the concrete, its plasticity and fabrication conditions, such that the steps of element demoulding, prestressing, lifting or transport may be performed upon completion of treatment. Moreover, treatment protocols must be examined by including the ambient temperature and relative humidity conditions associated with production and storage, so as to avoid any thermal shock as well as the occurrence of cracks or surface defects detrimental to either concrete durability or the appearance of the element.

Pre-casting piers during construction of the new coast road in La Réunion



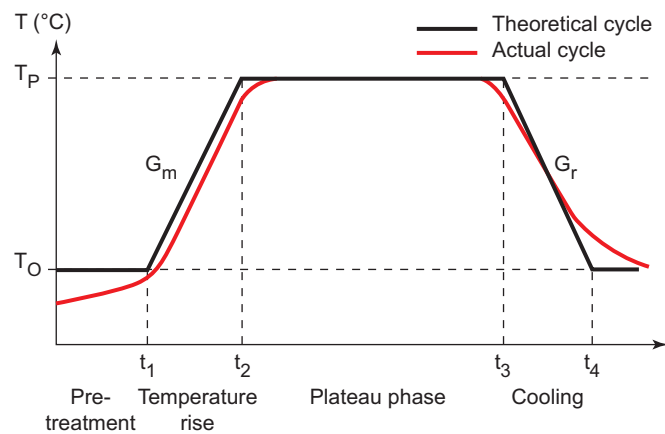
Photo credit: Viaduc Littoral photo library, Sébastien Marchal

In its most general form, a thermal cycle comprises four phases (figure 1), with each defined by a duration-temperature pair or by a rate:

- pre-treatment phase
- temperature rise phase
- temperature plateau phase
- cooling phase

Preliminary tests are carried out to optimise each of these phases.

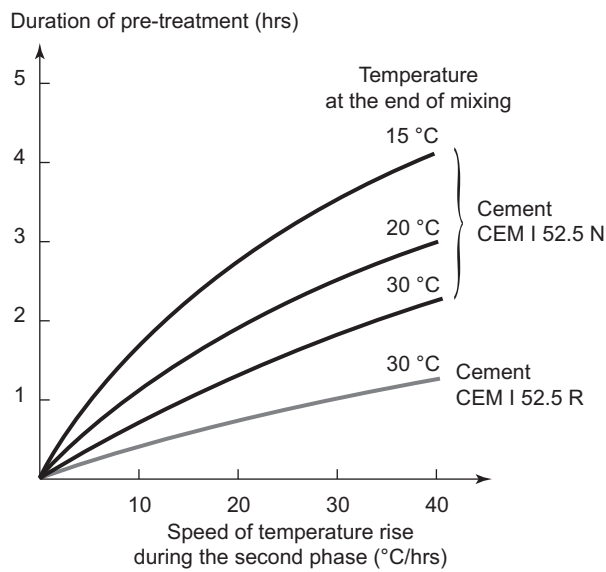
Figure 1
General form of a cycle



Pre-treatment phase

The pre-treatment phase is intended to provide the concrete with enough cohesion for it to absorb the internal stresses generated by the thermal expansion of its constituents, in particular water and air, when the temperature rises. The higher the rate of temperature rise during the subsequent phase and the slower the setting of the concrete, the longer the required pre-treatment (figure 2).

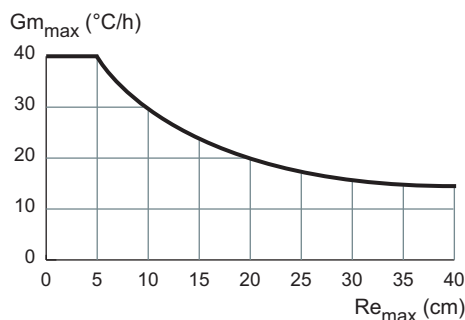
Figure 2
Influence of the type of cement and rate of temperature rise on the duration of the pre-treatment phase



Temperature rise phase

The rate of temperature increase must be such that the expansion stresses within the element caused by expansion can be absorbed at any time by the concrete, which gradually becomes stiffer. For discussion purposes, figure 3 shows the order of magnitude for the maximum rate of temperature increase $G_{m_{max}}$ (as measured in °C/hr) vs. the “maximum radius of steam curing” Re_{max} (in cm).

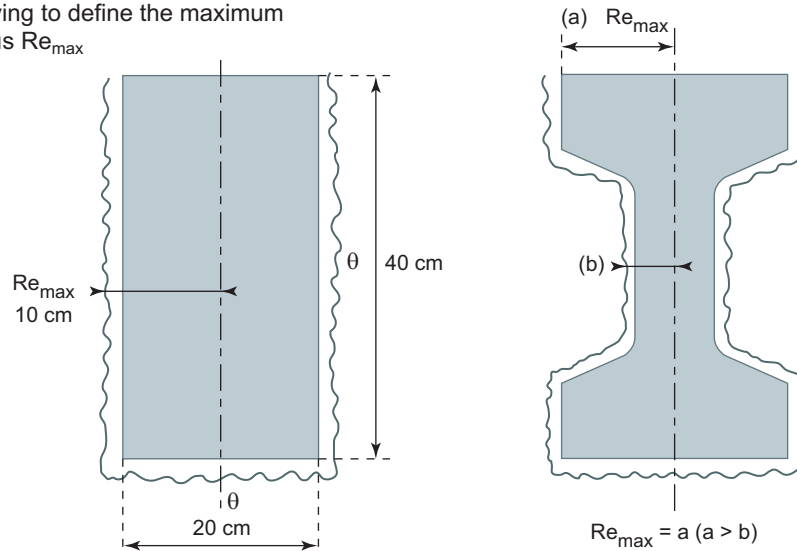
Figure 3
Values of the maximal rate of temperature increase $G_{m_{max}}$ vs. maximum radius of steam curing Re_{max}



Definition of Re_{max}

If we consider all the shortest distances between each point in the concrete and the heated face, the value of Re_{max} corresponds to the greatest of these distances (figure 4).

Figure 4
Examples serving to define the maximum steaming radius Re_{max}



Temperature plateau phase

The duration and temperature of this phase, during which the concrete hardening process, which already started during the previous phase, continues, depend on:

- the level of curing acquired by the concrete upon completion of the temperature rise phase
- the number of daily production runs
- the desired level of strength

The duration of this plateau depends on its temperature; typically it is between 1 and 3 hours at 85 °C, and between 4 and 12 hours at 65 °C. As regards the pertinent standards, NF EN 13369 stipulates maximum concrete temperature as a function of the environment to which the pre-cast element will be exposed.

During this phase, special attention must be given to applying the necessary precautions to prevent the concrete from drying so as to ensure that hydration takes place as completely as possible.

It must also be ensured that the temperatures between the various points in the large elements or in the various elements subjected to the same treatment remain close in value and constant in order to obtain identical strength levels and avoid the detrimental consequences of differential expansion.

Cooling phase

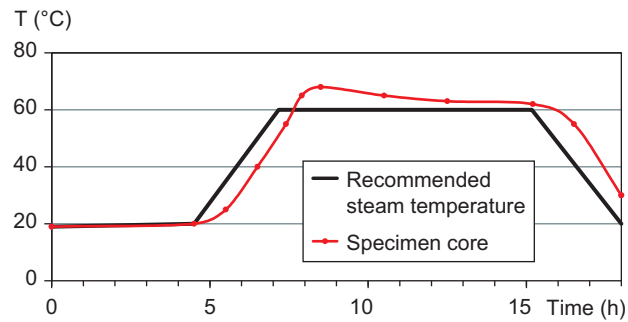
Cooling must also proceed in a homogeneous manner. Disorders are in fact more the result of the temperature differences between the various points in an element than to the cooling rate. The surface cooling rate is higher than the rate at the element core. The standard NF EN 13369 stipulates that the temperature difference between adjacent* parts of the product during heating and cooling should be limited to 20 °C.

* In the framework of these recommendations, the adjacent parts are considered to be the core and the surface of the element.

Example of heat treatment

Figure 5 depicts an example of a thermal cycle applied to a self-compacting concrete mix design. The maximum temperature reached at the concrete core was equal to 68 °C for the first cycle. The mechanical compressive strength values obtained on 11 x 22 cylindrical specimens at 18 hours and 28 days were 39 MPa and 59.5 MPa, respectively.

Figure 5
Example of a thermal cycle



Appendix 1.

Some precisions on the Delayed Ettringite Formation

The phenomenon of Delayed Ettringite Formation (DEF) involves the delayed formation of a mineral called ettringite, whose chemical formula is $3\text{CaO}\cdot\text{Al}_2\text{O}_3\cdot 3\text{CaSO}_4\cdot 32\text{H}_2\text{O}$. This delayed formation of ettringite is capable of causing expansion, as revealed by the occurrence at the concrete surface of multidirectional cracking with a relatively wide grid pattern. Yet ettringite is not systematically detrimental to concrete given that it is a normal product of cement hydration. For this reason, we will provide below some background information on the various types of ettringite found in concrete.

The various forms of ettringite (primary, secondary and delayed)

At present, several terms are used in the literature to distinguish between the various modes and timetables for the formation of ettringite in concrete. Three types of ettringite capable of coexisting in a given concrete sample can be identified. We have used the names proposed for them by the French Association of Civil Engineering (AFGC), as the result of the work of its “GranDuBé” group (French acronym for Quantities associated with Concrete Durability) [3]:

- primary ettringite, which does not cause any expansion
- secondary ettringite, which might cause expansion
- delayed ettringite, subsequent to a temperature rise in the concrete at an early age, which can also cause expansion

Primary ettringite is a product of cement hydration that occurs due to a reaction between the setting/hardening regulator (calcium sulphate) and the calcium aluminates. Ettringite takes the form of needle-shaped (or acicular) crystals, which do not cause swelling since they are formed prior to concrete hardening within unoccupied spaces in the material. They even serve a beneficial purpose by contributing to cement paste cohesion at an early age by decreasing porosity while simultaneously increasing the mechanical strength of the mix. In particular, this phenomenon is predominant during the setting of supersulphated cements and sulfoaluminate cements.

Secondary ettringite is formed once the concrete has already hardened and is the result of water movement inside the concrete (dissolution / precipitation phenomena) and external sulphates from soils, selenitic water, etc., or internal sulphates from aggregate that contains sulphates or the mixing water. In the latter case, compliance with standards provides a means of avoiding such a reaction. For the dissolution / precipitation phenomena, ettringite forms needle-shaped (acicular) crystals within the unoccupied spaces in the concrete and typically does not display expansive behaviour. On the other hand, this secondary-stage formation subsequent to an external contribution of sulphates may generate swelling. Unlike the non-expansive facies of the material, this pathological ettringite crystallises into a massive and compressed material.

Delayed ettringite occurs mainly in concretes that were exposed during early age to temperatures in excess of 65 °C. Above this temperature, the primary ettringite does not form during cement hydration reactions and/or is broken down. The source of sulphate ions is thus internal, given that these ions are generated by the absence or decomposition of primary ettringite. After returning to ambient temperature and in the presence of moisture, ettringite is able to form or reform, at which point it may generate swelling pressures under certain conditions.

A layer of delayed ettringite at the interface between an aggregate grain and the cement paste

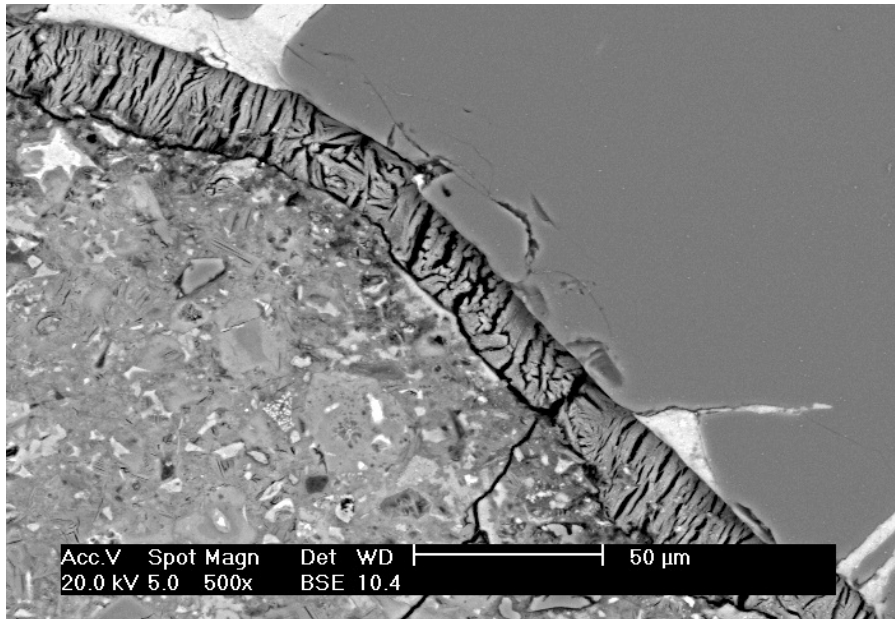


Photo credit: Béatrice Desrues, Ifsttar

The physicochemical mechanisms associated with DEF

DEF therefore occurs in a cementitious material subsequent to setting and without any contribution from external sulphates. The mechanism may be broadly broken down into several processes, chiefly the creation of a source of potentially-remobilised sulphates, the delayed precipitation of ettringite and expansion which causes concrete cracking [4].

Creation of a sulphate source

A considerable rise in temperature modifies cement hydration reactions. What takes place is that the sulphates provided by the setting/hardening regulator cannot be entirely mobilised to form primary ettringite. Moreover, the solubility of primary ettringite increases with temperature, which is reflected by a higher sulphate ion concentration in the concrete pore solution. A significant quantity of these ions is also trapped by physical adsorption at the surface of some cement hydration products. This phenomenon is indeed reversible, thus offering a reserve of sulphate for subsequent ettringite formation.

Ettringite precipitation

After the hardening of concrete and a return to ambient temperature, possibly in conjunction with leaching of alkalis contained in the concrete pore solution, ettringite precipitation can arise at reactive sites that include aluminates. Concentration conditions may then become dominant which lead to a highly unstable local chemical system along with the formation, within an enclosed space, of an ettringite which is often described as poorly-crystallised. This process can locally produce high pressures and cause swelling.

Expansion

The expansive or non-expansive nature of ettringite depends on the initial chemical composition, particularly on the type of cement (contents of aluminates and alkali, quantity of potentially-formed Portlandite) and the quantity of sulphates capable of being mobilised. Unanimous agreement has not yet been reached regarding the precise mechanism by which ettringite formation is able to generate pressures inside concrete. Two principal mechanisms, which are to some extent linked to each other, have been proposed to explain the swelling caused by ettringite formation:

- swelling due to the crystallisation pressures inherent in ettringite crystal growth
- swelling due to the osmotic pressures caused by an increase in the amount of colloidal ettringite

In reality, it is likely that both of these mechanisms play a role simultaneously and cannot be dissociated from one another.

The impact of DEF on mechanical properties

DEF leads to swelling which impairs the mechanical characteristics of concrete according to the extent of the reaction. In the rest of this section, we shall analyse the changes in the mechanical characteristics of concrete whose 28 days strength was of the order of 35 to 40 MPa.

In the case of concretes that are exposed to thermal cycles which are typical of massive structures (a temperature plateau of 81 °C maintained for three days), Martin [5] obtained reductions in compressive strength of over 75 % in the case of a concrete in which DEF has generated a maximum unrestrained expansion of approximately 1.4 % (concrete tested after approximately 1,450 days of ageing). The same material's Young's modulus was reduced by almost 90 % (as determined by linear regression of the stress-strain plot over three loading cycles at between 5 and approximately 30 % of the concrete's compressive strength). These drastic reductions in the mechanical performance should nevertheless be seen in the context of the very high level of expansion exhibited by this material. In the case of more moderate unrestrained expansion (0.12 %), Martin [5] observed a much smaller reduction in Young's modulus of approximately 14 % at 1,350 days.



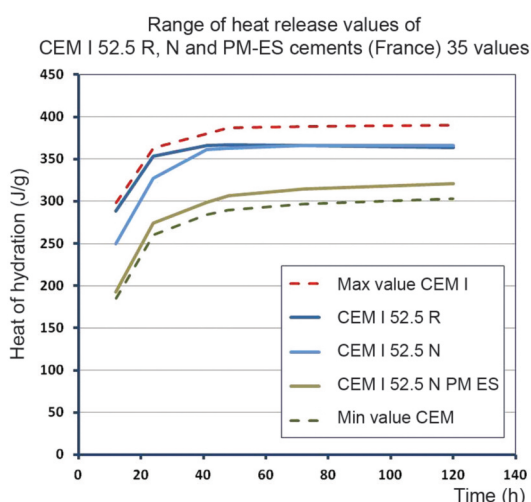
Appendix 2.

Review of the exothermic reaction of concretes

Concrete setting and hardening is accompanied by heat release due to the exothermic nature of hydration reactions. Depending on the concrete mix design and especially the type and content of binder, the amounts of heat can vary widely, as can the rate of heat release and the sensitivity of this rate to temperature.

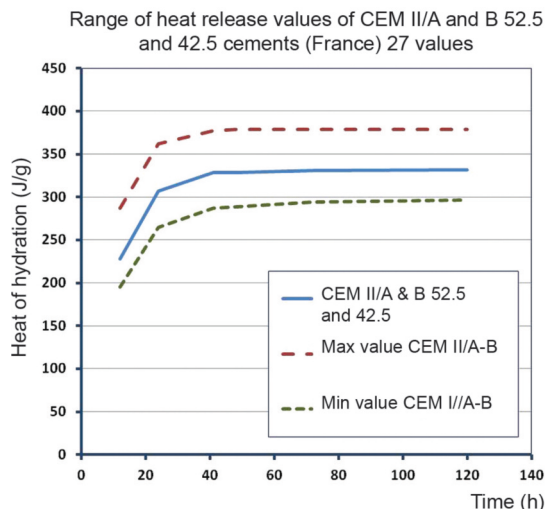
For purposes of illustration, the four figures below will show the minimum, mean and maximum plots of the heat released by different types of French manufactured cements, measured in standardised mortars conserved under quasi-adiabatic conditions within a thermally-insulated “Langavant” bottle.

Figure 6
Examples of heat release curves for different CEM I type 52.5 cements produced in France (mean values obtained from four manufacturers' data: 35 values)



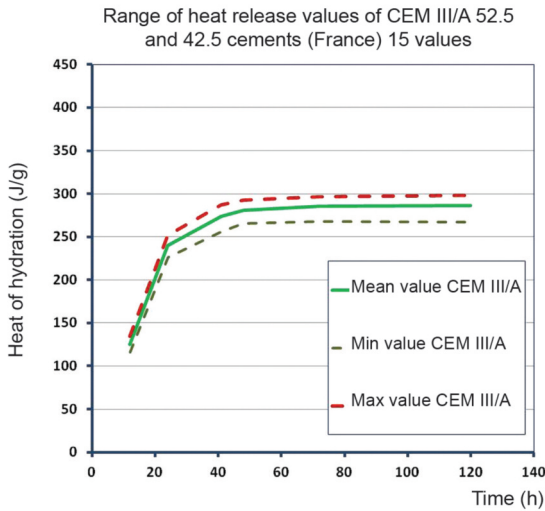
Source: ATILH

Figure 7
Examples of heat release curves for different CEM CEM II/A and B 42.5 and 52.5 cements produced in France (mean values obtained from 4 manufacturers' data; 27 values mainly for CEM II/A-LL, CEM II/A-S and CEM-II/B-M(S-LL))



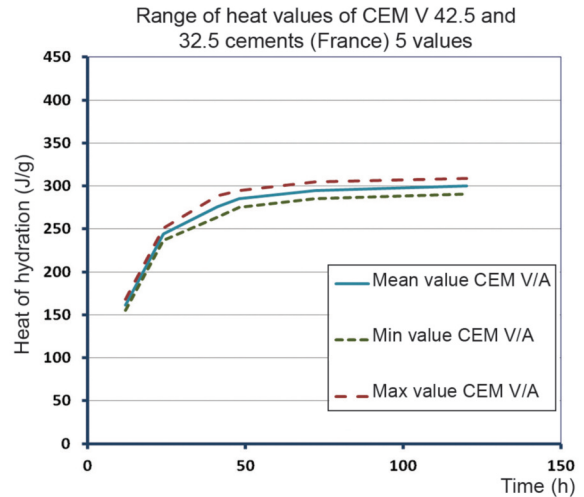
Source: ATILH

Figure 8
Examples of heat release curves for different type CEM III/A cements produced in France (mean values obtained from 3 manufacturers' data; 15 values)



Source: ATILH

Figure 9
Examples of heat release curves for different type CEM V/A cements produced in France (mean values obtained from 2 manufacturers' data; 5 values)

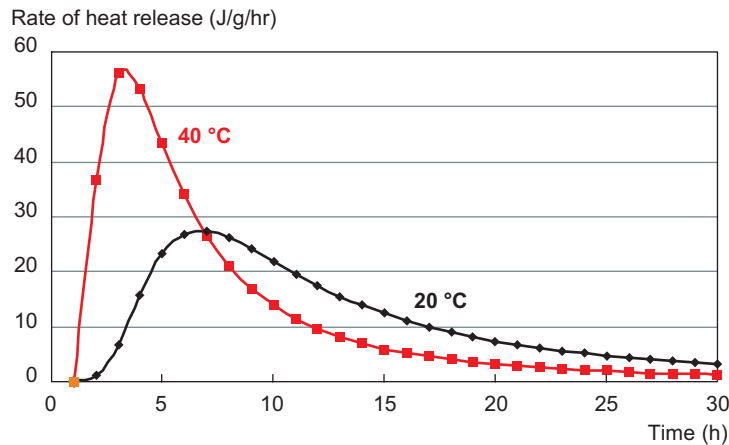


Source: ATILH

A study conducted by Technical Committee CEN/TC 51 in 1995 on heat release under semi-adiabatic conditions for a dozen European cements revealed that the heat released at 41 hours (Q_{41}) varied between 210 and 320 J/g. The ratios between the heat released at 41 hrs (Q_{41}) and that released at 72 hrs (Q_{72}) varied between 0.85 and 0.95 depending on the type of cement.

In general terms, hydration reaction rates are heavily influenced by temperature (e.g. acceleration by a factor ranging from 2 to 4 is observed at 40 °C compared to 20 °C). In addition, the sensitivity of concrete mix design is described by a parameter referred to as the activation energy.

Figure 10
Rate of heat release for a CEM I 52.5 R cement vs. temperature



This sensitivity (and thus this energy) drops when the cement shows greater reactivity during early age, as illustrated in table 5, which offers a number of sample values for the activation coefficient for different types of cement. E/R stands for activation energy divided by the perfect gas constant – note that this value can still vary significantly from one concrete to the next for a given type of cement:

Table 5
Sample values of the activation coefficient for various types of cement

Cement	CEM I 52.5 R	CEM I 42.5 R	CEM I 42.5 N	CEM II 32.5 R	CEM II 32.5 N	CEM III/C 32.5 N
E/R (K)	3.540	3.970	4.150	4.810	5.530	6.700

The temperature rise within a concrete element depends not only on the exothermic reaction of the material, but also on its geometry, the initial temperature of the material and thermal losses. It is impossible to establish a precise limit regarding element thickness (in most cases, thermal losses occur in a preferred direction, and the dimension of the element in this direction yields the “thickness” as referred to here). Based on this “thickness”, the term “massive element” may be employed, and in principle this raises fears over an excessive increase in the temperature of the concrete, since competition between heat release and heat losses involves the material, the geometry of the element and the boundary conditions. A footing measuring 1.5 m thick with a C30/37 concrete containing 370 kg/m³ of CEM III/A 42.5 N cement could experience a 29 °C temperature rise, whereas a 60 cm wall concreted using wooden formwork with a C40/50 concrete containing 400 kg/m³ of CEM I 52.5 R would undergo a temperature rise of 45 °C. The notion of massive element thus proves irrelevant for preventing the risk of DEF.

The notion of “critical” element will therefore be used in reference to a concrete element whose heat release will only be very slightly discharged to the outside and will lead to a sizeable temperature rise in the concrete.

It should be noted that temperature is not uniform within the concrete and that relatively sharp gradients (depending on the insulation provided by the formwork) are present near the outside of the element. For this reason, the maximum temperature that is of interest with regard to delayed ettringite formation is that reached at the element core.

In order to evaluate the temperature rise within a concrete element and determine whether the conditions for a “critical” element have been met, a number of tools are provided in Appendix 3 of this report.

It should also be pointed out that a temperature rise within a concrete part can cause damage at several levels:

- cracking due to restrained thermal shrinkage
- impairment the concrete’s long-term mechanical properties
- risk of delayed ettringite formation

A substantial temperature rise is in fact often correlated not only with high temperature gradients between the concrete core and skin, which are capable of causing cracks during initial surface cooling (these cracks are the least serious since subsequent cooling of the core tends to keep them closed), but also with temperature gradients relative to the previously-cast adjacent elements that have had sufficient time to cool. In this latter case,

regularly distributed through-cracking is observed whose distribution becomes even more regular where the rebar density is higher, and which begins at the concrete cast joints. This cracking may facilitate water ingress and promote delayed ettringite formation. Gradients may also develop in zones featuring an abrupt change in concrete element cross-section, where they may also give rise to harmful through-cracking.

Furthermore, as mentioned above, hydration reactions may be accelerated as a result of an inflow of heat, yet it is well known that high curing temperatures modify the nature of hydrates formed and mean that in the long term the concrete has inferior characteristics - particularly in terms of compressive strength - than the same concrete that had not been steam-cured. As was the case for the impact on short-term strength values, the impact on long-term strength depends to a great extent on the concrete mix design. As an example, a 70 °C heat treatment lasting 9 hours (with a 10 °C/hr temperature rise) can cause a 10 % loss in compressive strength at 700 days, whereas a 90 °C treatment for 6 hours (at the same rate of temperature increase) can result in a 20 % loss [2].

In the case of concretes that are made with CEM I 52.5 R cements and concretes that are subjected to heating typical of that of massive elements (different heat treatment profiles applied with maximum temperatures of between 71 °C and 81 °C applied for durations varying between 1 and 3 days), Kchakech [6] obtained 28 days compressive strengths that were systematically lower by about 20 %, and in some cases 30 %.

It goes without saying that by optimising the set of heat treatment parameters, durable concretes can still be obtained despite the decrease in mechanical strength.

Appendix 3.

Estimation of temperatures reached in planned structures

1. Purpose

This appendix is intended to propose a simplified methodology for assessing, prior to breaking ground on a building project, whether some elements need to be considered as critical with respect to the risks of delayed ettringite formation (in correlation with the risk of excessive temperature at the core of cast elements), in light of the concrete mix design principles adopted as part of the contract technical specifications.

The present appendix thus makes it possible to estimate the maximum temperature rise at the core of an element for which only the thickness (at its smallest dimension) and a small amount of basic data on concrete composition are known. On the basis of the maximum temperature T_{max} which must not be exceeded at the core of the element in question (which depends on the desired level of prevention), the maximum possible initial temperature T_{ini_max} of the fresh concrete during concreting is deduced.

The accuracy of this method remains limited due to the small number of parameters involved (i.e. those known or easily obtained during the preliminary phase). It should be used as a warning tool; if its conclusion is that the element is critical, then a more in-depth study must be carried out or the parameters must be modified. This is also why the temperature values obtained have been rounded to the nearest degree in all the examples given below.

2. Data required to estimate the maximum allowable initial temperature

The simplified method can be run once the following parameters have been determined:

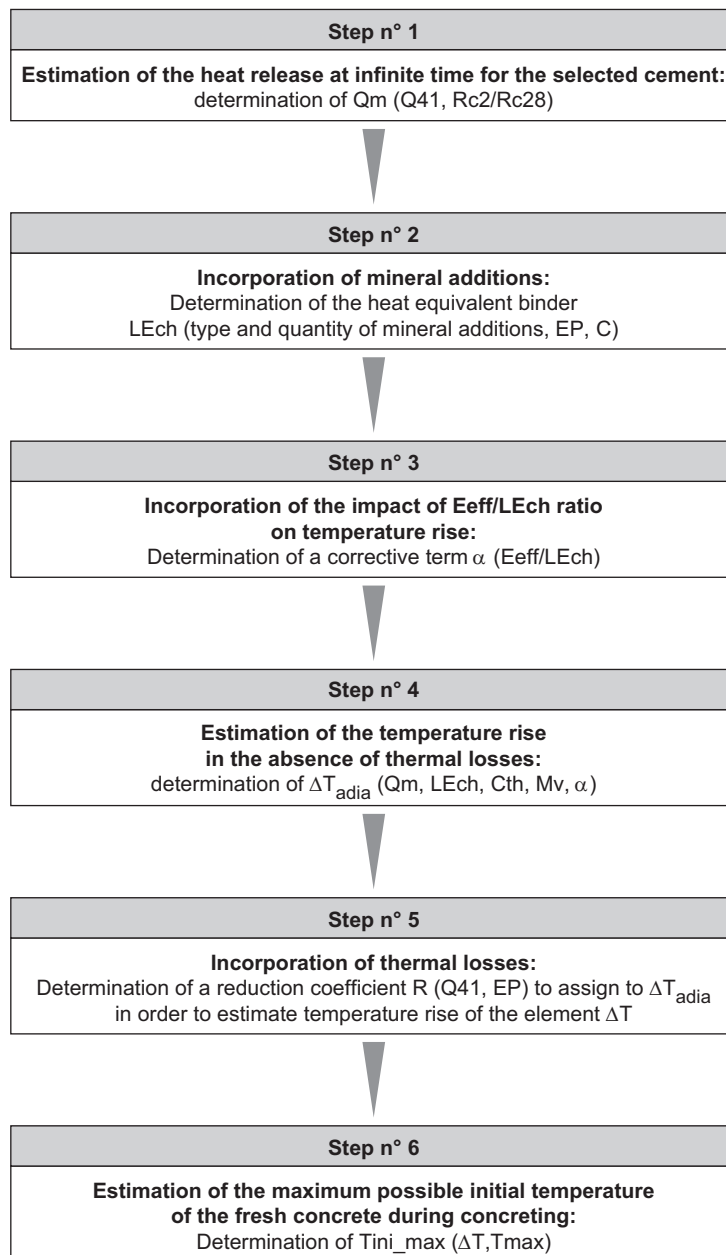
- Maximum allowable temperature according to the level of prevention T_{limite} (in °C) (see Chapter 3)
- Cement content of the concrete mix, C (in kg/m^3)
- Mineral additions content, A (in kg/m^3)
- Mass density of the concrete, M_v (in kg/m^3)
- Effective water content of the concrete, E_{eff} (in kg/m^3)
- 2-day compressive strength of the cement, R_{c2} (in MPa), according to Standard NF EN 196-1
- 28-day compressive strength of the cement, R_{c28} (in MPa), according to Standard NF EN 196-1
- Cement hydration heat according to the Standard NF EN 196-9 at 41 hours, Q_{41} (in kJ/kg), or preferably at 120 h Q_{120} (in kJ/kg)
- Element thickness, EP (in m) - only if greater than 0.25 m (below this threshold value, the element is not critical with respect to the risks of delayed ettringite formation) readers are referred to the concept of critical part (Chapter 1, Section 1)

The cement-related data are generally available from technical data sheets and/or on demand from suppliers. Q120 is particularly appropriate for cements with slow heat release such as CEM III and CEM V.

Element thickness is established from the smallest dimension (in the preferred direction for thermal losses).

3. Calculation steps

The sequence of calculation steps is presented in the flowchart below (figure 11) and then described in greater detail in the following text:



3.1 Estimation of the heat release at infinite time Q_m for the selected cement

We shall consider two cases here: one where the Q_{120} heat is available, and another when only the Q_{41} heat is. If Q_{120} is available, it will be preferred for the estimation of the heat release at infinite time for the concrete.

The estimation of the heat release at infinite time for cement Q_m (in kJ/kg) is obtained by applying the following formulae:

- For CEM I and CEM II cements: $Q_m = 1.05 Q_{120}$ [eq. 1]
- For CEM III and CEM V cements: $Q_m = 1.15 Q_{120}$ [eq. 2]

Or if Q_{120} is not available

$$Q_m = \max(Q_{41}, Q_{41} \times \text{ratio_}Q_m/Q_{41}) \text{ [eq. 3]}$$

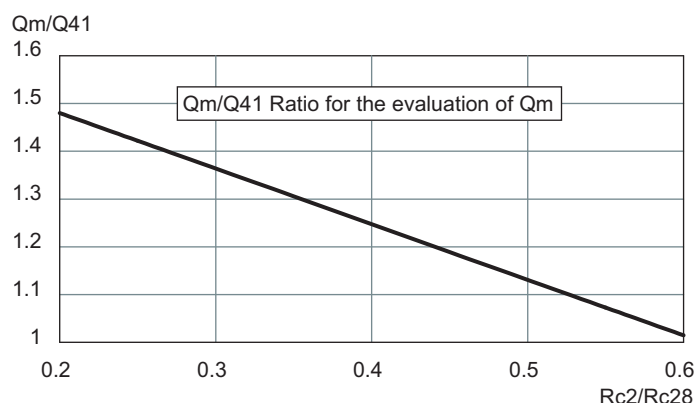
where the value $\text{ratio_}Q_m/Q_{41}$ is given by the following equation on the basis of the R_{c2}/R_{c28} ratio:

$$\text{Ratio_}Q_m/Q_{41} = 1.71 - 1.16 R_{c2}/R_{c28} \text{ [eq. 4]}$$

where R_{c2} is the 2-day compressive strength and R_{c28} is the 28-day compressive strength.

Figure 11 illustrates this relationship.

Figure 11
 Q_m/Q_{41} ratio vs R_{c2}/R_{c28} [eq 4]



CEM IV cements are not considered in this appendix because they are not used in France so we have no data. They are therefore outside the scope of this appendix

3.2 Incorporation of mineral additions

Additions contribute to concrete's heat release. In order to allow for their impact, we have introduced the concept of "heat equivalent binder", denoted by LE_{ch} (kg/m^3). It is given by the formula:

$$LE_{ch} = C + \sum K'_i A_i \text{ [eq. 5]}$$

where A_i is the proportion of addition i

K'_i is the weighting coefficient for addition i which is given by the following equations:

- in the case of silica fume $K'_{fs} = 1$
- in the case of metakaolin $K'_m = 1$
- in the case of calcareous additions and siliceous additions, $K'_a = 0$
- in the case of fly ash

$$\left\{ \begin{array}{l} \text{if } EP \leq 1 \text{ m, } K'_{cv} = 0 \\ \text{if } 1 \text{ m} < EP \leq 5 \text{ m, } K'_{cv} = -0.0357 EP^2 + 0.4143 EP - 0.38 \text{ [eq. 6]} \\ \text{if } EP > 5 \text{ m, } K'_{cv} = 0.8 \end{array} \right.$$

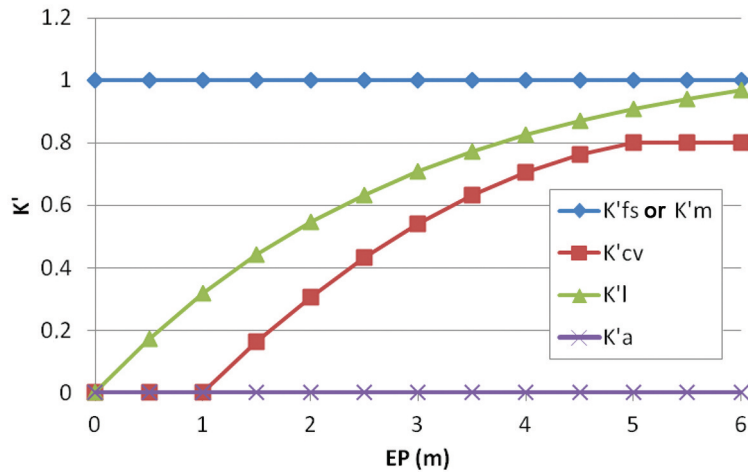
• in the case of slags: $K'I = 1.12 (1 - \exp(-EP/3))$ [eq. 7]

In the absence of additions, $LE_{ch} = C$ is applied.

Figure 12 shows a plot of the weighting coefficients for the additions according to element thickness (EP).

Figure 12

Coefficient K' of the additions for calculating the heat equivalent binder LE_{ch} on the basis of element thickness (EP) (fs = silica fume; m = metakaolin; cv = fly ash; l = blast furnace slag; a = calcareous or siliceous additions)



3.3 Incorporation of the impact of the E_{eff}/LE_{ch} ratio

The temperature rise resulting from the heat released by the binder also depends on the E_{eff}/LE_{ch} ratio (effective water divided by equivalent binder) which determines the maximum long-term rate of hydration. Here “equivalent binder” is defined as in the Standard NF EN 206/CN (readers are referred to this standard for a detailed account of the calculation). The lower this ratio, the less complete is hydration and the less heat is released. This is expressed by a multiplicative corrective term α given by the following equation:

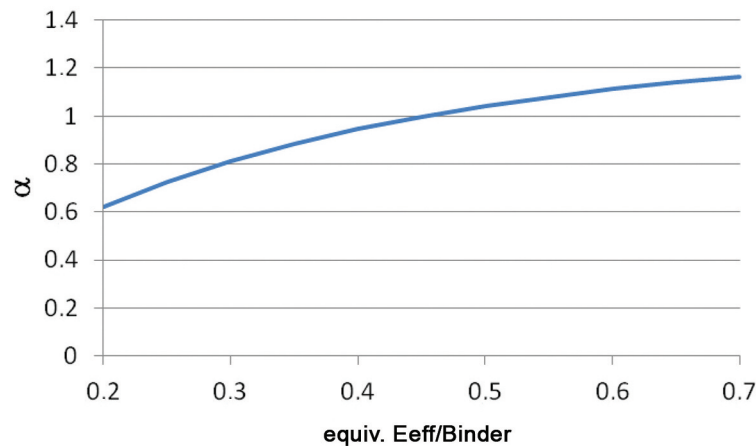
$$\alpha = 1.29 \left(1 - e^{-3.3 \frac{E_{eff}}{Binger \text{ eq.}}} \right) \text{ [eq. 8]}$$

Figure 13 shows how the corrective term varies with the ratio $E_{eff}/Binger$ equivalent.

Figure 13

Correction of the temperature rise related to the ratio E_{eff} / equivalent Binder [Eq. 8]

The formula is calibrated to give no correction ($\alpha = 1$) for an E_{eff} / equivalent Binder ratio equal to 0.45



3.4 Estimation of the temperature rise in the absence of thermal losses

At this stage, it is possible to evaluate the temperature rise ΔT_{adia} ($^{\circ}\text{C}$) under adiabatic conditions (i.e. perfect insulation) based on the formula:

$$\Delta T_{adia} = \alpha \times (Q_m \times L_{Ech}) / (C_{th} \times M_v) \quad [\text{Eq 9}]$$

where C_{th} is the thermal capacity of the concrete, taken as $1 \text{ kJ}/(\text{kg} \cdot ^{\circ}\text{C})$.

3.5 Incorporation of thermal losses

Thermal losses are particularly affected by the cement reactivity and element thickness (as the rate of heat production competes with the rate of heat dissipation). The formula below provides the reduction coefficient R (lying between 0 and 1) which allows these heat losses to be factored in, with the reactivity of the cement being expressed via the parameter Q_{41} :

$$R(EP, Q_{41}) = \min(1; (1 / (1 + ((\text{MAX}(0.3; -0.0057 \cdot Q_{41} + 2.0558)) / EP)^{1.5}))) \quad [\text{eq. 10}]$$

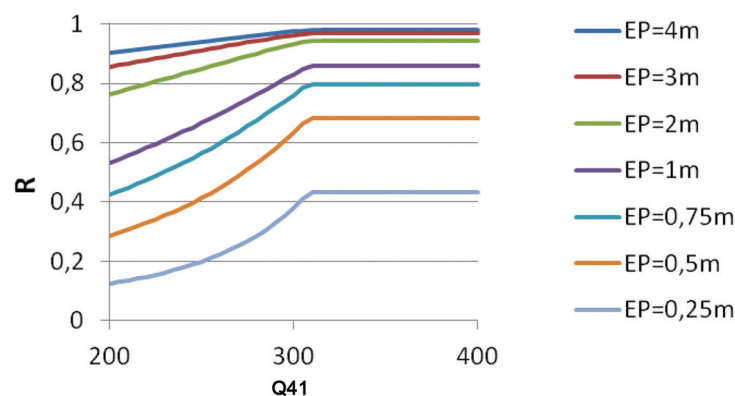
If the thickness EP is equal to or greater than 5m, the value $R = 1$ is applied.

The R value serves to estimate the temperature rise ΔT ($^{\circ}\text{C}$) using the formula:

$$\Delta T = R \times \Delta T_{adia}$$

Figure 14

Reduction coefficient R



3.6 Estimation of the maximum possible initial temperature for the fresh concrete during concreting

The initial maximum temperature of the concrete T_{ini_max} is calculated by the formula:

$$T_{ini_max} = T_{max} - \Delta T \text{ [eq. 11]}$$

where T_{max} is the maximum allowable temperature for the concrete in the structure.

If under worksite conditions the forecast temperature of the fresh concrete at the time of concreting is greater than T_{ini_max} , the element is deemed to be critical and only a more detailed study could justify the acceptability of heating from the standpoint of the risk of DEF.

4. Application examples

In the examples below, the calculated values of the ratios (Q_m/Q_{41}) et (E_{eff}/L_{iant} eq.), of α , of K' and of R have been rounded to two decimal points, the other results have been rounded to the nearest unit.

4.1 Example No. 1: concrete with no addition (element thickness: 1 m)

Data:

• Type of cement	CEM I
• Cement content of concrete	$C = 350 \text{ kg/m}^3$
• Mineral additions content of concrete	$A = 0 \text{ kg/m}^3$
• Mass density of concrete	$M_v = 2400 \text{ kg/m}^3$
• Effective water content of concrete	$E_{eff} = 175 \text{ kg/m}^3$
• 2-day compressive strength of cement	$R_{c2} = 27 \text{ MPa}$
• 28-day compressive strength of cement	$R_{c28} = 68 \text{ MPa}$
• Hydration heat of cement at 41 h	$Q_{41} = 306 \text{ kJ/kg}$
• Element thickness	$EP = 1 \text{ m}$
• Maximum allowable temperature	$T_{limite} = 65 \text{ }^\circ\text{C}$

Step n° 1: estimation of heat release at infinite time for the selected cement

With the above data, the ratio $R_{c2}/R_{c28} = 0.40$

Use of equation 4 makes it possible to determine the ratio $Q_m/Q_{41} = 1.25$

The maximum long term heat release is thus $Q_m = Q_{41} \times 1.25 = 383 \text{ kJ/kg}$

Step n° 2: incorporating mineral additions

The concrete mix design does not include a mineral addition $A = 0 \text{ kg/m}^3$

In this case the equivalent binder heat $LE_{ch} = C$

Step n° 3: taking account of the impact of the ratio E_{eff}/L_{iant} eq.

In this example, E_{eff}/L_{iant} eq. = $E_{eff}/C = 0.50$. Equation 8 then gives $\alpha = 1.04$

Step n° 4: estimation of the rise in temperature in the absence of heat losses

$$\Delta T_{adia} = \alpha \times (Q_m \times LE_{ch}) / (C_{th} \times M_v) = 1.04 \times (383 \times 350) / (1 \times 2400) = 58 \text{ }^\circ\text{C}$$

Step n° 5: taking account of heat losses

Equation 10 with cement hydration heat $Q_{41} = 306 \text{ kJ/kg}$ and the thickness of the element to be cast $EP = 1 \text{ m}$ gives a reduction coefficient $R = 0.85$

The temperature rise in the element is estimated by the formula:

$$\Delta T = R \times \Delta T_{adia} = 0.85 \times 58 = 49 \text{ }^\circ\text{C}$$

Step n° 6: estimation of the highest possible initial temperature of the fresh concrete

As the maximum allowable temperature of the concrete is $65 \text{ }^\circ\text{C}$, the initial temperature of the fresh concrete at the time of concreting must be equal to $16 \text{ }^\circ\text{C}$ at the most

4.2 Example No. 2: concrete with fly ash (element thickness: 3 m)

Data:

• Type of cement	CEM I
• Cement content of concrete	$C = 320 \text{ kg/m}^3$
• Mineral additions content of concrete	$A = 80 \text{ kg/m}^3$
• Mass density of concrete	$M_v = 2400 \text{ kg/m}^3$
• Effective water content of concrete	$E_{\text{eff}} = 175 \text{ kg/m}^3$
• 2-day compressive strength of cement	$R_{c2} = 27 \text{ MPa}$
• 28-day compressive strength of cement	$R_{c28} = 68 \text{ MPa}$
• Hydration heat of cement at 41 h	$Q_{41} = 306 \text{ kJ/kg}$
• Element thickness	$EP = 3 \text{ m}$
• Maximum allowable temperature	$T_{\text{limite}} = 75 \text{ }^\circ\text{C}$

Step n° 1: estimation of heat release at infinite time for the selected cement

With the above data, the ratio $RC2/R_{c28} = 0.40$

Use of equation 4 makes it possible to determine the ratio $Q_m/Q_{41} = 1.25$

The maximum long term heat release is thus $Q_m = Q_{41} \times 1.25 = 383 \text{ kJ/kg}$

Step n° 2: incorporation of mineral additions

The concrete mix design includes fly ash $A = 80 \text{ kg/m}^3$. In this case the equivalent heat binder $LE_{\text{ch}} = C + K' \times A$. The coefficient $K'_{cv} = 0.54$ was obtained using equation 6 for $EP = 3.00 \text{ m}$ and silicoaluminous fly ash

$$LE_{\text{ch}} = C + K' \times A = 320 + 0.54 \times 80 = 363 \text{ kg/m}^3$$

Step n° 3: incorporation of the impact of the ratio $E_{\text{eff}}/L_{\text{iant}}$ eq.

In this example, $E_{\text{eff}}/L_{\text{iant}} \text{ eq.} = E_{\text{eff}}/(C + 0.4 \times A) = 175/352 = 0.50$

Equation 8 then gives $\alpha = 1.04$

Step n° 4: estimation of the temperature rise in the absence of heat losses

$$\Delta T_{\text{adia}} = \alpha \times (Q_m \times LE_{\text{ch}})/(C_{\text{th}} \times M_v) = 1.04 \times (383 \times 363)/(1 \times 2400) = 60 \text{ }^\circ\text{C}$$

Step n° 5: incorporation of thermal losses

Equation 10, with cement hydration heat $Q_{41} = 306 \text{ kJ/kg}$ and the thickness of the element to be cast $EP = 3.00 \text{ m}$, gives the reduction coefficient $R = 0.97$. The temperature rise in the element is estimated by the formula:

$$\Delta T = R \times \Delta T_{\text{adia}} = 0.97 \times 60 = 58 \text{ }^\circ\text{C}$$

Step n° 6: estimation of the highest possible initial temperature for the fresh concrete

As the maximum allowable temperature of the concrete is $75 \text{ }^\circ\text{C}$, the initial temperature of the fresh concrete at the time of concreting must be equal to $17 \text{ }^\circ\text{C}$ at the most

4.3 Example No. 3: concrete with slag addition (element thickness 1.50 m)

Data:

• Type of cement	CEM I
• Cement content of concrete	$C = 180 \text{ kg/m}^3$
• Mineral additions content of concrete	$A = 140 \text{ kg/m}^3$
• Mass density of concrete	$M_v = 2460 \text{ kg/m}^3$
• Effective water content of concrete	$E_{\text{eff}} = 153 \text{ kg/m}^3$
• 2-day compressive strength of cement	$R_{c2} = 34.6 \text{ MPa}$
• 28-day compressive strength of cement	$R_{c28} = 65 \text{ MPa}$
• Hydration heat of cement at 41 h	$Q_{41} = 297 \text{ kJ/kg}$
• Element thickness	$EP = 1.50 \text{ m}$
• Maximum allowable temperature	$T_{\text{limite}} = 65 \text{ }^\circ\text{C}$

Step n° 1: estimation of heat release at infinite time for the selected cement

With the above data, the ratio $RC2/R_{c28} = 0.53$

Use of equation 4 makes it possible to determine the ratio $Q_m/Q_{41} = 1.09$

The maximum long term heat release is thus $Q_m = Q_{41} \times 1.09 = 324 \text{ kJ/kg}$

Step n° 2: taking account of mineral additions

The concrete mix design includes ground slag in a proportion of $A = 140 \text{ kg/m}^3$

The coefficient K' , because of the 1.5 m thickness is equal to 0.44

In this case the equivalent binder heat is equal to $LE_{\text{ch}} = C + 0.44 \times A = 242 \text{ kg/m}^3$

Step n° 3: taking account of the impact of the ratio $E_{\text{eff}}/L_{\text{iant}}$ eq.

In this example, $E_{\text{eff}}/L_{\text{iant}} \text{ eq.} = E_{\text{eff}}/(C + 0.9 \times A) = 153/306 = 0.50$. Equation 8 then gives the corrective term $\alpha = 1.04$

Step n° 4: estimation of the rise in temperature in the absence of heat losses

$\Delta T_{\text{adia}} = \alpha \times (Q_m \times LE_{\text{ch}})/(C_{\text{th}} \times M_v) = 1.04 \times (324 \times 242)/(1 \times 2460) = 33 \text{ }^\circ\text{C}$

Step n° 5: taking account of heat losses

Use of equation 10 with the hydration heat of the concrete $Q_{41} = 297 \text{ kJ/kg}$ and the thickness of the element to be cast $EP = 1.50 \text{ m}$ leads to a reduction coefficient $R = 0.89$

The temperature rise in the element is estimated by the formula

$\Delta T = R \times \Delta T_{\text{adia}} = 0.89 \times 33 = 29 \text{ }^\circ\text{C}$

Step n° 6: estimation of the maximum possible initial temperature of the fresh concrete

As the maximum allowable temperature of the concrete is $65 \text{ }^\circ\text{C}$, the initial temperature of the fresh concrete at the time of concreting must be equal to $36 \text{ }^\circ\text{C}$ at the most

4.4 Example No. 4: concrete made with CEM III, comparison between the results with Q120 and those with Q41

Data:

• Type of cement	CEM III
• Cement content of concrete	$C = 385 \text{ kg/m}^3$
• Mass density of concrete	$M_v = 2310 \text{ kg/m}^3$
• Effective water content of concrete	$E_{\text{eff}} = 165 \text{ kg/m}^3$
• 2-day compressive strength of cement	$R_{c2} = 15 \text{ MPa}$
• 28-day compressive strength of cement	$R_{c28} = 56 \text{ MPa}$
• Hydration heat of cement at 41 h	$Q_{41} = 220 \text{ kJ/kg}$
• Hydration heat of cement at 120 h	$Q_{120} = 250 \text{ kJ/kg}$
• Element thickness	$EP = 1.80 \text{ m}$
• Maximum allowable temperature	$T_{\text{limite}} = 65 \text{ }^\circ\text{C}$

1st case: taking Q120 into account

Step n° 1: estimation of heat release at infinite time for the selected cement

The maximum long-term heat release is given by the equation $Q_m = 1.15$
 $Q_{120} = 1.15 \times 250 = 288 \text{ kJ/kg}$

Step n° 2: taking account of mineral additions

As the concrete mix design includes no addition, $LE_{\text{ch}} = C = 385 \text{ kg/m}^3$

Step n° 3: taking account of the impact of the ratio $E_{\text{eff}}/L_{\text{ant}}$ eq.

In this example, $E_{\text{eff}}/L_{\text{ant}}$ eq. = $E_{\text{eff}}/C = 0.43$ which corresponds, based on equation 8, to the corrective term $\alpha = 0.98$

Step n° 4: estimation of the rise in temperature in the absence of heat losses

$$\Delta T_{\text{adia}} = \alpha \times (Q_m \times LE_{\text{ch}}) / (C_{\text{th}} \times M_v) = 0.98 \times (288 \times 385) / (1 \times 2310) = 47 \text{ }^\circ\text{C}$$

Step n° 5: taking account of heat losses

Use of equation 10 with the cement hydration heat $Q_{41} = 220 \text{ kJ/kg}$ and the thickness of the element to be cast $EP = 1.80 \text{ m}$ gives the reduction coefficient $R = 0.77$

The temperature rise in the element is estimated by the formula

$$\Delta T = R \times \Delta T_{\text{adia}} = 0.77 \times 47 = 36 \text{ }^\circ\text{C}$$

Step n° 6: estimation of the maximum possible initial temperature of the fresh concrete

As the maximum allowable temperature of the concrete is $65 \text{ }^\circ\text{C}$, the initial temperature of the fresh concrete at the time of concreting must be equal to $29 \text{ }^\circ\text{C}$ at the most

2nd case: taking account of Q41 (calculation without taking account of Q120)

Step n° 1: estimation of heat release at infinite time for the selected cement

With the above data, the ratio $RC2/Rc28 = 0.27$

Use of equation 4 makes it possible to determine the ratio $Qm/Q41 = 1.40$

The maximum long term heat release is thus $Qm = Q41 \times 1.40 = 308 \text{ kJ/kg}$

Step n° 2: taking account of mineral additions

As the concrete mix design includes no addition, $LEch = C = 385 \text{ kg/ m}^3$

Step n° 3: taking account of the impact of the ratio $E_{eff} / L_{iant \text{ eq.}}$

In this example, $E_{eff} / L_{iant \text{ eq.}} = E_{eff} / C = 0.43$ which corresponds, based on equation 8, to the corrective term $\alpha = 0.98$

Step n° 4: estimation of the rise in temperature in the absence of heat losses

$\Delta T_{adia} = \alpha \times (Qm \times LEch) / (C_{th} \times Mv) = 0.98 \times (308 \times 385) / (1 \times 2310) = 50 \text{ }^\circ\text{C}$

Step n° 5: taking account of heat losses

Use of equation 10 with the cement hydration heat $Q41 = 220 \text{ kJ/kg}$ and the thickness of the element to be cast $EP = 1.80 \text{ m}$ leads to a reduction coefficient $R = 0.77$

The temperature rise in the element is estimated by the formula

$\Delta T = R \times \Delta T_{adia} = 0.77 \times 50 = 39 \text{ }^\circ\text{C}$

Step n° 6: estimation of the maximum possible value for the initial temperature of the fresh concrete.

As the maximum allowable temperature of the concrete is $65 \text{ }^\circ\text{C}$, the initial temperature of the fresh concrete at the time of concreting must be equal to $26 \text{ }^\circ\text{C}$ at the most

5. Comparison between the method and real cases

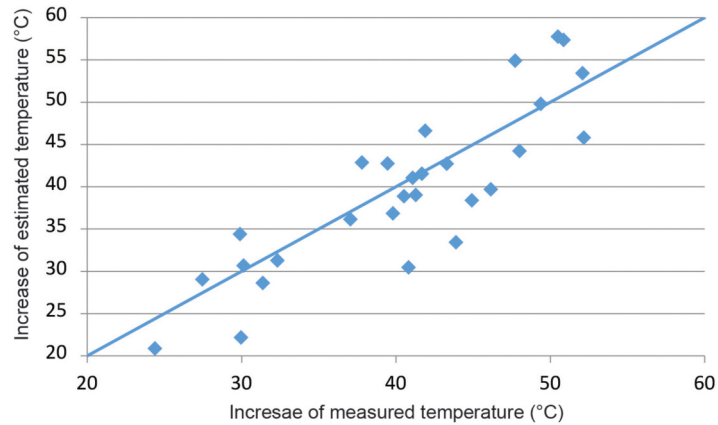
in order to illustrate the predictive capabilities of the method described in this appendix, we have compared it to measurements made on a variety of concretes, first of all under adiabatic conditions and then at worksites. The latter comparison allows us to estimate the “accuracy” of the method.

5.1 Comparison with measurements of adiabatic heat release

As part of Agathe Bourchy’s research for her doctoral thesis [7], adiabatic measurements of heat release were conducted on 27 different concretes. The composition of the concretes was varied, and involved the use of a CEM I cement, a CEM II cement, silica fume and slag. The measurements were compared with the temperature rises obtained using Appendix 3 at the end of Step 4 (figure 15). The comparison clearly showed that the model predicted the temperature rise under adiabatic conditions fairly accurately.

In order to characterise the error of the model in comparison with the measurements, we have used the RMSE (root mean square error) as an indicator. This is the square root of the mean differences between the forecasts and the observations. It provides the standard deviation of the model’s prediction error and thus an indication of its accuracy. In the case of the data in figure 15, the RMSE is equal to $5 \text{ }^\circ\text{C}$.

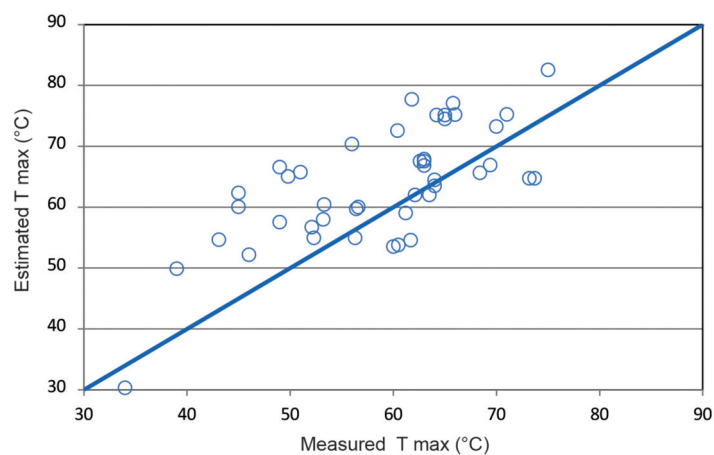
Figure 15
Comparison between temperature rises under adiabatic conditions for 27 concretes, from Agathe Bourchy's thesis [7]



5.2 Comparison with in situ measurements

The full method has been applied to 44 real structures for which we know the temperature rise which was measured within the structure. In these structures, concretes with very differing mix designs using different types of cements and varied additions were used figure 16 allows us to compare the maximum estimated temperatures with the measurements in structures. It is apparent that application of the method proposed in this appendix generally leads to a slight overestimation of the maximum temperature (intentionally, for the purposes of safety). The RMSE is equal to 8.5 °C which shows that, in comparison with the RMSE obtained at the end of step 4, the step 5 (the estimation of heat losses) introduces a considerable amount of additional uncertainty. This may doubtless be explained by the failure to take account of a number of parameters (initial temperature of the concrete, the outside temperature, the type of formwork, etc.).

Figure 16
Comparison between the maximum temperatures for 44 real cases





Appendix 4.

Performance-based testing

General remarks

A performance test conducted on concrete specimens has been developed in partnership between the French Technical Association of the Hydraulic Binder Industry (ATILH), the Concrete Industry Study and Research Centre (CERIB) and the LCPC. This test has been published by the LCPC as: “*LPC Test Method N°. 66: Reactivity of a concrete mix design with respect to delayed ettringite formation - Performance testing*”.

This test procedure was developed in order propose a reliable alternative to the tests described in the literature, whose representativeness is now challenged due to both the excessive temperatures imposed on the material and the selected specimen dimensions.

Since the production of an initial publication as draft LPC test method no. 59 (May, 2003), subsequent tests have been conducted by CERIB and the LPC network of Ponts et Chaussées laboratories on concretes representative of massive structural elements or on products generated by the concrete pre-casting industry with known in situ behaviour, with regard to DEF. These studies have confirmed the accuracy and discriminatory power of the test.

The reproducibility of the test method was then examined during a campaign of cross-referencing tests performed within the scope of the work programme undertaken by the GranDuBé group affiliated with the AFGC association. The level of reproducibility depends on the expansion of the concrete and can be as high as 60 % of the average swelling value when a 0.2 % (m/m) expansion has been measured. Despite this level of dispersion, which may be attributed to swelling, the measurements recorded have led all laboratories involved to the same conclusion as to the “non-reactive” or “potentially reactive” nature of the studied concrete/heating pairs with regard to DEF.

Testing principle

The test consists of characterising the swelling risk for a concrete with respect to DEF. The concrete is defined by both its mix design and heating exposure during early age.

The test comprises four distinct steps, as follows:

- concrete fabrication
- heat treatment to simulate concrete heating
- drying and wetting cycles
- definitive immersion in water, and longitudinal deformation monitoring

The minimum duration of this test calls for 12 months of immersion, and this period may be extended to 15 months should significant expansion be measured.

Interpretation of results

The test report is to contain, at the very least, the following elements:

- The data listed in the test sheets appended to the test method.
- A record of the temperatures inside the climate-controlled containment during thermal treatment and, if possible, those measured inside the concrete.
- A graph showing the plots of the expansion of each specimen as well as the average expansion of all three specimens vs. immersion time.
- Interpretation of the results with regard to the decision-making criteria presented below. To confirm the source of swelling, these measurements may be accompanied by a scanning electron microscope evaluation whenever concrete swelling has been detected.

The “concrete mix design / heating” pair is considered to be suitable for use provided that one of the two following criteria (1 or 2), pertaining to the swelling threshold and gradient of the swelling curve, is being met:

Criterion 1:

The average longitudinal deformation of 3 specimens is less than 0.04 % and no individual value exceeds 0.06 % by the end of the 12-month period,

Criterion 2:

If the mean longitudinal deformation of the 3 specimens exceeds 0.04 % upon expiration of the 12-month period, the test must be extended through the 15th month. In this case, the mean longitudinal deformation of the 3 specimens must be less than 0.06 % at 15 months, and the cumulative variation between the 12th and 15th months must remain below 0.01 %, and moreover no individual value may exceed 0.07 % at 15 months.

Appendix 5.

Approval methodology for CEM I SR3 and SR5 conforming to the marking NF – Liants hydrauliques (NF002)

Goal

Prevention of the risks related to DEF: approval methodology for determining the suitability of CEM I SR3 and SR5 that conform to NF – Liants hydrauliques (NF002) marking for heating with a duration that exceeds 10 hours.

Principle

Fabrication of a “test concrete” and evaluation based on LPC performance test No. 66 applying a given thermal cycle for a desired level of protection.

Test concrete

The content of CEM I SR3 or SR5 cement will be fixed at 400 kg/m³. The Weffice/C ratio will also be constant, and equal to 0.45. Slump will be maintained at a value of 180 +/- 30 mm by using a plasticiser or superplasticiser if necessary. Dry siliceous aggregate from Palvadeau sandpit (NF aggregate, non-reactive as regards AAR) will be used in a proportion of 1,800 kg/m³ applying the granular distribution set out in table 6. In addition, alkali doping should be applied in the test as detailed in the standard NF P18-454, possibly with an increased proportion in order to create an additional margin of variability.

Table 6
Proportion by mass (kg/m³) of each granular fraction

Sand 0/0.315 mm	Sand 0.315/1 mm	Sand 1/4 mm	Chippings 4/8 mm	Chippings 8/12.5 mm	Chippings 12.5/20 mm
130	130	450	210	570	310

Thermal cycles

For the level of protection Bs, (covering Cs and Ds): fresh concrete at ambient temperature, 12 hours of temperature increase to 85 °C, maintained at the 85 °C plateau for three days, temperature reduction to 20 °C in 32 hours (a cooling rate of 2 °C/h).

For the level of protection Cs (covering Ds): fresh concrete at ambient temperature, 12 hours of temperature increase to 80 °C, maintained at the 80 °C plateau for three days, temperature reduction to 20 °C in 30 hours (a cooling rate of 2 °C/h).

For the level of protection Ds: fresh concrete at ambient temperature, 12 hours of temperature increase to 75 °C, maintained at the 75 °C plateau for three days, temperature reduction to 20 °C in 27 hours (a cooling rate of 2 °C/h).

Evaluation

The CEM I SR3 or SR5 cement will have approval for the desired level of protection if the level of expansion of the “test concrete” under the recommended thermal cycle is below the specified limit values at the times stated in Appendix 4 of this document.

Precautions during placement

The proportion of cement in the concretes that are placed in the element of the structure must not be greater than that of the “test concrete”. In addition, the alkali content of the concrete that is placed must not be greater than that of the “test concrete”.

Verification by simulation

In addition, the simulation tool must be employed to verify that the maximum temperature of the concrete that is placed in the element of the structure in question corresponds to the same level of prevention as that applied for the “test concrete”.

Duration of the validity of the approval for the cement

This method allows approval to be given for the use of a cement for a period of 5 years, on condition the same clinker is employed. The concretes that are to be used as a result of this approval must not have an alkali content that exceeds the maximum alkali content used in the “test concrete”.

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