

Recent Advances in Concrete Technology and Sustainability Issues

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American Concrete Institute

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Preface

The Canada Center for Mineral and Energy Technology (CANMET) of Natural Resources of Canada, Ottawa, has played a significant role for more than 40 years in the broad area of concrete technology in Canada. In recent years CANMET has become increasingly involved in research and development dealing with the supplementary cementing materials, high-performance normal weight and lightweight concretes, and alkali-aggregate reactions. As part of CANMET's technology transfer program, an international symposium on Advances in Concrete Technology was sponsored jointly with the American Concrete Institute (ACI) and other organizations in Athens, Greece, May 1992. In June 1995 CANMET, in Association with ACI and other organizations in Canada and the U.S., sponsored the Second CANMET/ACI International Symposium on Advances in Concrete Technology in Las Vegas, Nevada. For the Athens symposium, the CANMET publication, "Advances in Concrete Technology," constituted the proceedings of the symposium. Proceedings from the Las Vegas symposium were published by ACI as SP-154.

In August 1997, CANMET, in association with ACI and other organizations in Canada and New Zealand, sponsored the Third CANMET/ACI International Symposium on Advances in Concrete Technology in Auckland, New Zealand. The main purpose of the symposium was to bring together representatives from industry, universities, and government agencies to present the latest information in concrete technology, and to explore research and development. Thirty-three refereed papers from 15 countries were presented and distributed at the symposium. Proceedings were published as ACI SP-171.

In June 1998, CANMET, in association with ACI, Japan Concrete Institute (JCI), and several other organizations in Canada and Japan, sponsored the Fourth CANMET/ACI International Conference on Recent Advances in Concrete Technology in Tokushima, Japan. More than 80 papers from 20 countries were received and reviewed in accordance with the policies of ACI. Sixty-one refereed papers were accepted for presentation at the conference and for publication as ACI SP-179. In addition to the refereed papers, more than 30 more papers were presented and distributed at the symposium.

In July-August 2001, CANMET, in association with ACI and several organizations in Singapore, sponsored the Fifth CANMET/ACI International Conference on Recent Advances in Concrete Technology in Singapore. More than 100 papers from more than 25 countries were received and reviewed in accordance with the policies of ACI. Forty-six refereed and more than 25 additional papers were accepted for presentation at the conference and published as ACI SP-200. In June 2003 CANMET, in association with ACI and several organizations in Romania, sponsored the Sixth CANMET/ACI International Conference on Recent Advances in Concrete Technology in Bucharest, Romania. More than 40 papers presented at the conference were distributed "as received," and no formal ACI special publication was published.

In May 2004, CANMET, in association with ACI and several organizations in the U.S., sponsored the Seventh CANMET/ACI International Conference on Recent Advances in Concrete Technology in Las Vegas, Nevada. Seventeen refereed papers from more than 10 countries were presented and distributed at the conference. The proceedings, consisting of refereed papers, were published as ACI SP-222. In addition to the refereed papers, 20 additional papers were presented and distributed at the conference.

In May 2006, CANMET, in association with ACI and several organizations in Canada and the U.S., sponsored the Eighth CANMET/ACI International Conference on Recent

Advances in Concrete Technology in Montreal, Canada. Proceedings of the conference, consisting of 17 refereed papers, were published as ACI SP-235. In addition to the refereed papers, more than 30 additional papers were presented and distributed at the conference.

In May 2007, CANMET, in association with ACI and several organizations in Canada, Europe, and the U.S., sponsored the Ninth CANMET/ACI International Conference on Recent Advances in Concrete Technology in Warsaw, Poland. Proceedings of the conference consisted of 10 refereed papers that were published as ACI SP-243. More than 20 additional papers were presented and distributed at the conference.

In October 2009, ACI, in association with several organizations in Canada, Europe and the U.S., sponsored the Tenth ACI International Conference on Advances in Concrete Technology in Seville, Spain. Proceedings of the conference consisting of 20 refereed papers that were published as ACI SP-261. In addition to the refereed papers, more than 20 additional papers were presented at the conference and published in a Supplementary Papers Volume.

In May 2010, the Committee for the Organization of International Conferences (COIC) (formerly CANMET/ACI Conferences), in association with the Chinese Ceramics Society (CCS) and several other organizations in China, sponsored the Eleventh International Conference on Advances in Concrete Technology and Sustainability Issues in Jinan, China. More than 40 papers were presented at the conference. The proceedings of the conference were published by the (CCS), Beijing, China.

In October 2012, the COIC, in association with ACI, sponsored the Twelfth International Conference on Advances in Concrete Technology and Sustainability Issues in Prague, Czech Republic. The proceedings of the conference consisted of more than 30 refereed papers that were published as SP-288. In addition to the refereed papers, more than 40 other papers were presented at the conference and were published in the Supplementary Papers Volume.

In July 2015, the (COIC), in association with the ACI sponsored the Thirteenth International Conference on Advances in Concrete Technology and Sustainability Issues in Ottawa, Canada. The proceedings of the conference consisting of 28 refereed papers were published by the ACI as SP-303. In addition to the refereed papers, more than 40 other papers were presented at the conference and were published in the Supplementary Papers Volume.

Many thanks are extended to the members of the Technical Paper Review Panel who met in Bahamas in October 3 to 11, 2014 to review the papers. Without their dedicated efforts, it would not have been possible to publish the proceedings for distribution at the conference. The cooperation of the authors in accepting the reviewers' suggestions and in revising the manuscripts accordingly is greatly appreciated.

The help and assistance of Dr. Pawan R. Gupta and Prabha Gupta are gratefully acknowledged in the administrative work associated with the conference, and processing of the manuscripts for both the ACI proceedings (SP-303) and the Supplementary Papers Volume.

*Terence C. Holland
Pawan R. Gupta
V. Mohan Malhotra
Editors*

Thirteenth International Conference on Advances in Concrete Technology and Sustainability Issues

July 14 to 17, 2015

Ottawa, Canada

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Responsibility, Durability, and Construction Requirements in ACI 318-14

by Terence C. Holland

ACI 318, “Structural Concrete Building Code,” has been completely reorganized for the 2014 edition. While the technical provisions of most of the code have been unchanged, changes have been made in the provisions dealing with responsibility, durability, and construction. The responsibility for use of the code has been assigned to the licensed design professional (LDP.) Minor changes have been made to the durability provisions to clarify the intent of the code and to help the LDP select exposure classes. The most significant changes in ACI 318-2014 are in a new chapter that addresses construction requirements. Previously, construction provisions were located throughout the code. The new code is based upon the premise that the contractor should not have to search the code for construction requirements. Instead, all such requirements must be in the construction documents that are a part of the contract between the owner and the contractor. The new chapter makes the LDP responsible for writing the applicable design information and compliance requirements into the construction documents.

Keywords: building codes; building codes history; responsibility; durability; construction documents; construction requirements.

INTRODUCTION

The primary purpose of the concrete building code is to provide for the life safety of occupants of concrete structures. Life safety is achieved by ensuring uniformity of design and meeting minimum construction requirements. From the very first code, both structural engineering and construction requirements have been included in the code. Durability requirements were not included in the early codes, but they have gradually been included in more and more detail over the years.

Starting with the very first code, there has been ambiguity over who the code is written for: is it the design engineer or the contractor? Until very recently, there was no attempt to address the requirements at a particular user. While the majority of the code is implicitly directed at the designer, there have always been numerous construction requirements that the contractor has been expected to find and implement.

Prior to the 2014 code cycle, ACI Committee 318 decided it was appropriate to undertake a major review and reorganization of the code. The planning, process, and outcome

of this effort have been discussed elsewhere.¹⁻³ In essence, the code has been organized on the basis of member chapters with tool box chapters providing provisions that are common to all members. The original intent was simply to reorganize existing provisions without introducing technical changes. This intent was generally followed, but some technical changes and new content are included. During the reorganization, the topics of responsibility, durability, and construction requirements were addressed resulting in changes.

This paper looks briefly at the history of responsibility, durability, and construction provisions in the ACI building code, and then discusses the changes made in developing the 2014⁴ edition.

HISTORY

The development of a building code for concrete was not a straight-forward process. In the early years there were actually competing codes developed by two different groups⁵ as well as codes developed by a number of municipalities. One group was a coalition of concrete users (the American Society for Testing Materials, the American Society of Civil Engineers, The American Railway Engineering and Maintenance of Way Association, and the Association of American Portland Cement Manufacturers) and the other was the National Association of Cement Users (NACU), forerunner of ACI. These competing codes were not merged into a single code until the 1940s.

The first document resembling a concrete building code that was published by the NACU appeared in 1904.⁶ This document was not prepared by NACU but rather by National Board of Fire Underwriters. It was presented to the NACU committee on specifications, but it was not adopted. In this document the provisions covering construction requirements outnumbered those dealing with structural design. This weighting toward construction reflected the lack of experience using reinforced concrete and the state of structural design at the time. Other than use of cover for fire protection, there was no mention of durability.

The first actual code document published by NACU appeared in 1910 with the title: "Standard Building Regulations for the Use of Reinforced Concrete."⁷ In this document, the construction provisions, including materials requirements, were about equal in length to design provisions. There was no mention of durability. One particularly interesting provision stated: "6. The specifications shall state the qualities of the materials to be used for making the concrete, and the proportions in which they are to be mixed." This requirement was clearly directed at the designer. As will be discussed below, it is a forerunner of the approach adopted for the 2014 edition.

After the 1910 edition, ACI building codes have appeared on a somewhat regular basis. The frequency of issue has become much more standardized at every six years for the current period. The next sections will highlight provisions dealing with responsibility, durability, and construction from various codes.

Responsibility

The following statement appears in the introduction to the document published by NACU in 1094:⁶ "There is probably no way in which the use of cement as a building material can be better advanced than by adequate laws. The results obtained in cement construction, where the work was intelligently designed and installed, have been excellent." This statement addresses the design and construction responsibilities that appear in the document.

The document itself, however, made no attempt to assign any responsibility explicitly to either the designer or the contractor.

In more recent documents, responsibility for certain provisions has been clearly directed at the designer. Construction requirements have been included in the codes, but there has been no explicit assignment of responsibility to the contractor. This practice was followed until the 2011 edition.⁸ However, in the 1983 edition⁹ (and in subsequent editions) the introduction to the commentary has included the following statements:

General references requiring compliance with ACI 318 in the job specifications should be avoided since the contractor is rarely in a position to accept responsibility for design details or construction requirements that depend upon a detailed knowledge of the design. Generally, the drawings, specifications and contract documents should contain all of the necessary requirements to ensure compliance with the code.

This warning has not been well applied in practice.

Construction provisions

Provisions covering construction practices and materials have always been in the code. This section presents a few examples. Much of the wording in these older codes is very recognizable to users of today's edition.

The 1904 pre code included requirements for cover for fire protection, mixing concrete, forms, cold weather protection, materials including cement ("Only high-grade Portland cement shall be used ..."), aggregates, and reinforcing steel. It also included proportions for 2000 psi (14 MPa) concrete.

The 1910 NACU document also included provisions on materials—cement, aggregates, and reinforcement. For construction, it had provisions on mixing and placing concrete, placing reinforcement, treatment of joints, and removal of forms. Two provisions are clearly recognizable:

26. The materials must be mixed wet enough to produce a concrete of such a consistency as will flow into the forms and about the metal reinforcement, and which, on the other hand, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

37. (b) Beams and Girders. Construction joints in beams and girders shall be vertical and within the middle third of the span.

Here are the corresponding provisions from the 2014 code:

26.5.2.1(f) Concrete shall be placed in accordance with (1) through (5):

(1) At a rate to provide an adequate supply of concrete at the location of placement

(2) At a rate so concrete at all times has sufficient workability such that it can be consolidated by the intended methods

(3) Without segregation or loss of materials

(4) Without interruptions sufficient to permit loss of workability between successive placements that would result in cold joints

(5) Deposited as near to its final location as practicable to avoid segregation due to rehandling or flowing

26.5.6.2(b) Except for prestressed concrete, construction joints in floor and roof systems shall be located within the middle third of spans of slabs, beams, and girders unless otherwise approved by the licensed design professional.

The construction provisions in the 1941 code¹⁰ are similar to what is in the 2011 edition of the code. Included were requirements for materials, storage of materials, strength to be determined at 28 days or earlier, proportions were to be developed using a table or by tests resulting in a water content versus 28-day strength curve. Chapter 4, “Mixing and Placing Concrete,” referenced the 1938 edition of ASTM C 94.* Chapter 5, “Forms and Details of Construction,” is very similar to what has appeared as Chapter 6 in more recent codes.

From 1941 through the present, the construction provisions have increased in number but the construction provisions retained the same general organization. The 1989 edition¹¹ was the first to place concrete mixture proportioning and construction practices into a single chapter. Chapter 6 covered formwork, embedded pipes, and construction joints. Other construction provisions were scattered throughout the code. Those chapters and the other construction provisions remained through the 2011 edition.

Durability provisions

Durability provisions appeared first in the 1941 edition. Since that time, they have increased in number and complexity covering additional durability concerns. The following description shows the development of the durability provisions:

- The 1941 edition required that for concrete exposed to weather, the water content shall not exceed 6 gal/sack (SI) ($w/c = 0.53$). Additionally this edition stipulated cover requirements for concrete exposed to weather.
- The 1956¹² edition contained no requirement for entrained air, but there was a provision that concrete exposed to action of freezing was limited to 6 gal/sack.
- The 1963¹³ edition included a provision stipulating the use of entrained air. There were also provisions dealing with concrete exposed to the action of sulfates.
- The 1971¹⁴ edition included provisions for watertight structures (controlled by a maximum w/c).
- The 1977¹⁵ edition was the first to include the table showing required entrained air content based upon aggregate size.
- The 1983 edition included limits on chloride ions for corrosion protection.
- The 1989 edition was the first to have a chapter devoted to durability addressing freezing and thawing, sulfate exposure, corrosion of reinforcement, and concrete required to have low permeability.
- The 2008¹⁶ edition introduced exposure categories and exposure classes to the code. This change was in keeping with the practice followed by other codes worldwide.

*ASTM C94 is the Standard Specification for Ready-Mixed Concrete.

DEVELOPMENT OF 2014 CODE

Basic approach

The reorganization of the code to prepare the 2014 edition started with a sorting of provisions into the basic agreed-upon structure with provisions generally sorted into the member and toolbox chapters. A significant number of provisions were orphans that did not seem to have a location in the reorganized code. These were primarily the construction related provisions. There were several iterations of the chapter containing these construction provisions before a consensus was reached by the committee.

Originally, there were three chapters covering materials, durability, construction, and inspection issues. These were:

- Concrete design and durability requirements;
- Requirements for concrete and grout; and,
- Construction documents and inspection.

As the organization of the construction documents chapter came into focus, the provisions of the latter two chapters were merged into Chapter 26, “Construction Documents and Inspection.” The first chapter became Chapter 19, “Concrete: Design and Durability Requirements.”

Responsibility

As the construction chapter was being prepared, it became evident that the code would be improved by clearly assigning the responsibility for using the code to the LDP As will be seen in the discussion of construction provisions, the licensed designer was tasked to include all necessary information to construct the project in the “construction documents.” ACI 2014 defines construction documents as “Written and graphic documents and specifications prepared or assembled for describing the location, design, materials, and physical characteristics of the elements of a project necessary for obtaining a building permit and construction of the project.” Preparation of construction documents is the responsibility of the licensed design professional. Construction documents are part of the project contract documents.

This change made the LDP responsible for the code and relieved the contractor from searching for construction responsibilities. The change was partially based upon the statement that had appeared in the instruction to the commentary since 1983 and the following provision from the International Building Code (IBC):¹⁷

106.1.1 Construction documents shall be of sufficient clarity to indicate the location, nature and extent of the work proposed and show in detail that it will conform to the provisions of this code and relevant laws, ordinances, rules and regulations, as determined by the building official.

Durability provisions

The durability provisions in the 2014 code are in Chapter 19. This chapter is organized as follows:

Chapter 19: Concrete: Design and Durability Requirements

19.1 – Scope

19.2 – Concrete design properties

Table 1. Revised descriptions of the Exposure Classes for concrete exposed to freezing and thawing. The first description is that from ACI 318-11. The revised description is shown underlined and replaces the original.

Class	Condition
F0	Concrete not exposed to freezing-and-thawing cycles.
F1	Concrete exposed to freezing and thawing cycles and occasional exposure to moisture <u>Concrete exposed to freezing-and-thawing cycles with limited exposure to water.</u>
F2	Concrete exposed to freezing and thawing cycles and in continuous contact with moisture <u>Concrete exposed to freezing-and-thawing cycles with frequent exposure to water .</u>
F3	Concrete exposed to freezing and thawing cycles and in continuous contact with moisture and exposed to deicing chemicals <u>Concrete exposed to freezing-and-thawing cycles with frequent exposure to water and exposure to deicing chemicals .</u>

19.3 – Concrete durability requirements

19.4 – Grout durability requirements

Section 19.3 is essentially the durability requirements that previously appeared in chapter 4 of the code. While there have been no major changes in the requirements, there have been several clarifications. These are explained below.

1. Exposure Category P, “Requiring low permeability,” has been changed to Exposure Category W, “In contact with water.” This change was made because permeability is not an exposure; it is a concrete property. The requirements have not been changed.

2. Two changes were made within Exposure Category F, “Freezing and thawing.” First, the descriptions of the exposure classes have been rewritten to make it easier for the licensed design professional to assign the correct class, Table 1. This change was made in response to comments from code users that the existing descriptions were not adequate. Additionally, the commentary for this portion of the code has been expanded to help differentiate among the four exposure classes. The term “saturated” was introduced and used to describe the potential for damage from exposure to freezing and thawing.

3. Second, the maximum w/cm and minimum compressive strength requirements for concrete exposed to freezing and thawing have been modified, Table 2. These changes were also made in response to user comments and to harmonize the requirements for Exposure Category F and Exposure Category C, “Corrosion protection of reinforcement.”

4. Two provisions that appear in the 2011 edition have been moved to Chapter 26. These are the provision limiting the amount of supplementary cementitious material for concrete in Exposure Class F3 and the provision allowing the use of alternative cementitious materials based upon testing for sulfate exposure. Callouts for these provisions are included in Chapter 19, but the actual requirements are in Chapter 26.

5. The list of allowable cements for Exposure Category S, “Sulfate,” has been expanded to accommodate new types of cements in ASTM C595¹⁸ and ASTM C1157.¹⁹ The cements covered by these standards are limited to those bearing the designation MS, moderate sulfate resistance, or HS, high sulfate resistance.

6. Wording has been added to the commentary to advise the licensed design professional that air content is typically measured as the concrete is discharged from the mixer or transferred

Table 2. Revised requirements for the Exposure Classes for concrete exposed to freezing and thawing. New requirements are underlined.

Exposure Class	Maximum w/cm	Minimum f'c, psi (MPa)	Additional requirements
F0	N/A	2500 (18)	No changes
F1	~~0.45~~ <u>0.55</u>	~~4500 (31)~~ <u>3500 (25)</u>	No changes
F2	0.45	4500 (31)	No changes
F3	~~0.45~~ <u>0.40</u>	~~4500 (31)~~ <u>5000 (35)</u>	No changes

portation unit. If it is required to measure air content at additional locations, provisions must be included in the construction documents.

7. The final change in Chapter 19 is the addition of Section 19.4, which limits chlorides in grout for bonded tendons. There had not been a limit for this grout in past codes. This provision states:

19.4.1 Water-soluble chloride ion content of grout for bonded tendons shall not exceed 0.06 percent when tested in accordance with ASTM C1218, measured by mass of chloride ion to mass of cement.

Construction provisions

By far, the most changes in the 2014 edition are found in Chapter 26, “Construction Documents and Inspection.” Compared to the 2011 edition, this chapter includes the mixture proportioning requirements that were in the first half of chapter 5, the mixing and placing requirements that were in the second half of chapter 5, the formwork requirements that were in chapter 6, and all of the other construction requirements that were found throughout the code. Also, all inspection requirements have been moved into this chapter. This chapter is now the longest in the document containing 33 pages of code and commentary.

The philosophy of this chapter is explained in the following two sections taken from the commentary:

This chapter establishes the minimum requirements for information that must be included in the construction documents as applicable to the project. The requirements include information developed in the structural design that must be conveyed to the contractor; provisions directing the contractor on required quality, and inspection requirements to verify compliance with the construction documents.

This chapter is directed to the licensed design professional responsible for incorporating project requirements into the construction documents. The construction documents should contain all of the necessary design and construction requirements for the contractor to achieve compliance with the Code. It is not intended that the Contractor will need to read and interpret the Code.

Table 3. Organization of Chapter 26, ACI 318-14.

Section	Coverage
26.1	Scope
26.2	Design Criteria
26.3	Member information
26.4	Concrete materials and mixture requirements
26.5	Concrete production and construction
26.6	Reinforcement materials and construction requirement
26.7	Anchoring to concrete
26.8	Embedments
26.9	Additional requirements for precast concrete
26.10	Additional requirements for prestressed concrete
26.11	Formwork
26.12	Concrete evaluation and acceptance
26.13	Inspection

To address this philosophy, the committee defined two types of information that are to be included in Chapter 26:

- Design information – project specific information to be incorporated into construction documents by the licensed design professional, as applicable.*
- Compliance requirements – construction-related code requirements directed to the contractor to be incorporated into construction documents by the licensed design professional, as applicable.*

The chapter is consistent throughout by dividing all provisions into one of these two categories. In many cases, a design information provision is accompanied by a compliance requirement. For example, here are complimentary provisions from the section on placing concrete:

- Design information: Locations where slab and column concrete placements are required to be integrated during placement in accordance with 15.3.
- Compliance requirement: At locations where slab and column concrete placements are required to be integrated during placement, column concrete shall extend full slab depth at least 2 ft into floor slab from face of column and be integrated with floor concrete.

Chapter 26 is organized as shown in Table 3. Although there are major changes in the philosophy and format of the provisions, there have been only a few significant technical changes. These changes are explained in the following discussion.

1. Section 26.4.1 Materials.
 - Includes requirements for all concrete materials. Some of the requirements are presented in tables rather than text, which is new for the code.
 - Update the allowable cement to include ternary and limestone blends from ASTM C595.
 - Added corrosion inhibiting admixtures meeting ASTM C 1582.²⁰
 - Added a discussion of ASTM C494²¹ Type S admixtures to the commentary.
2. Section 26.4.2 Concrete mixture requirements.

- Committee 318 discussed but did not accept making contractor responsible for choosing critical requirements for concrete mixtures by simply specifying exposure classes. Including the governing exposure classes in the construction documents is permitted but not required.
- As a result of this decision, there is a long list of design information that must be provided to ensure that the correct concrete mixtures are developed.

3. Section 26.4.3 Proportioning of concrete mixtures.

The first provisions of this section are very similar to those that have been in the code for a very long time:

26.4.3.1 Compliance requirements:

(a) Concrete mixture proportions shall be established so that the concrete satisfies (1) through (3):

(1) Can be placed readily without segregation into forms and around reinforcement under anticipated placement conditions

(2) Meets requirements for assigned exposure class in accordance with either 26.4.2.1(a) or 26.4.2.1(b)

(3) Conforms to strength test requirements for standard-cured specimens

The most significant changes appear in the provisions dealing with the method of proportioning. All of the detailed statistically-based requirements have been removed from the code. This change was made because the committee believed that such detailed information does not belong in the code. The previous materials have been replaced with this provision:

(b) Concrete mixture proportions shall be established in accordance with Article 4.2.3 of ACI 301 or by an alternative method acceptable to the licensed design professional. Alternative methods shall have a probability of satisfying the strength requirements for acceptance tests of standard-cured specimens that meets or exceeds the probability associated with the method in Article 4.2.3 of ACI 301.

The article referenced in ACI 301²² contains the same detailed statistical approach to proportioning that has been in ACI 318. However, the provision allows the contractor to use an alternative approach to proportioning if the probability of cylinders failing the acceptance testing remains the same. This probability of failure has been in the code and commentary over the years as limiting failure to one test in 100.

4. 26.7 – Anchoring to concrete.

- The provisions that were previously in Appendix D of the 2011 edition have been moved into the body of the code as Chapter 17, but the format was not updated to that of the 2014 edition.
- Some but not all of the design information and compliance requirements have been moved to chapter 26. It is anticipated that Chapter 17 will be updated during the 2019 code cycle.

5. 26.10 – Additional requirements for prestressed concrete.

- In the 2011 edition, the provisions for grout for bonded tendons (Section 18.18) were very weak. These provisions were deleted during the preparation of the 2014 code.

- A reference to the Post Tensioning Institute document²³ on grouting bonded tendons has been added to the commentary. This reference may move into the code in future editions.

Inspection

Although not a primary topic of this paper, it is worth mentioning the changes in the 2014 edition regarding inspection. Similar to the construction provisions, all inspection requirements are now located in the same section, 26.13.

As the code and the IBC are currently written, the inspection requirements in ACI 318 only come into effect in the absence of a locally adopted building code. Other materials standards have harmonized their inspection requirements with those in the IBC. The IBC now simply reference those provisions like the remainder of the code.

An attempt was made during the preparation of the 2014 code to reach the same situation for the concrete inspection requirements. The inspection provisions were reformatted to follow the IBC requirements for continuous and intermittent inspection. However, Committee 318 did not accept the complete set of revisions, so the goal of having the inspections requirements incorporated by reference was not achieved.

FUTURE WORK

There are several areas where needed changes to the code have already developed. These items, as well as others, will be looked at during the next code cycle.

- Add provisions regarding alkali-aggregate reactions.
- Add provisions governing the use of shotcrete.
- Move much of the “specification-like” material (compliance requirements) out of the code. Once all of these provisions was collected into Chapter 26, many committee members realized that material may not belong code.
- Make the mixture proportioning provision a performance provision by simply referring to the probability of failing the acceptance criteria. The reference to ACI 301 in the mixture proportioning section was a compromise to achieve consensus in Committee 318. Some members now believe that it may be more appropriate to remove this reference from the code. Interestingly, the criterion of one test in 100 failing either of the acceptance criteria was in the 1971 edition. This approach was abandoned and the detailed statistical procedure was added.

CONCLUSIONS

The 2014 edition of ACI 318 has achieved several major goals regarding responsibility, durability, and construction:

- The question of who the code is written for has been cleared up. The LDP is now clearly responsible for including all appropriate information in the construction documents.
- Several issues regarding durability have been clarified without making major changes.
- All construction and inspection provisions have been collected into a single chapter.

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Grouts for Bridge Post-tensioning Tendons at Below-Freezing Temperature

by Yang Fumin, Ma Rongtian, Zhang Yuanqing, and Zhang Yong

Materials for grouting post-tensioning tendons of a bridge at below freezing temperature (-10°C or 14°F) were studied. The water/cementitious material ratio was a maximum of 0.32 and the grout properties were tested at -10°C (14°F) according to applicable Chinese standards for post-tensioning grouts. After special treatment, the efflux times of the modified grouts were between 14 and 22 seconds after mixing and less than 30 seconds after 60 minutes without agitation. The final strength of the grout reached 50-60 MPa (7975-8700 psi) after 35 days of curing (7 days at below freezing temperature and 28 days at a standard temperature), which was 95%-100% of the reference strength for curing at a standard temperature.

Keywords: grout; bridge; post-tensioning tendon; winter construction; efflux time; strength.

INTRODUCTION

Grouts provide the primary protection for post-tensioned beams in railway bridge construction. Cementitious grouts provide an alkaline environment that passivates the surface of steel reinforcements and inhibits the corrosion process. Therefore, the durability of post-tensioned bridges is directly affected by the quality of the grouting.

In general, the grout temperature at time of grouting is critical and the grouting process must not be conducted if the ambient temperature is below 5°C (40°F) or above 35°C (95°F). According to the current Chinese national standard GB/T50448-2008¹, “Cementitious Grouting Materials Specification”, when grouting post-tensioned beams, the grout temperature should be between 5°C and 30°C (41°F and 86°F). For 3 days after grouting completed, the curing environment should be maintained higher than 5°C (41°F); otherwise, special heating procedures should be used to maintain the minimum curing temperature required.

In the current state of practice, it is easy and safe to use some procedures to guarantee the grout curing temperature in the factory²⁻⁴: 1) Building insulated enclosures, and maintaining the temperature inside by injecting high temperature steam, until 3 days after grouting or until the grout strength exceeds 25 MPa (3625 psi); 2) Increasing the grout

Table 1 Physical parameters of SAC

Strength grade	Specific (m ² /kg)	Setting time(min)		Compressive(MPa)			Flexural(MPa)		
		Initial	Final	1d	3d	28d	1d	3d	28d
42.5	365	25	135	30.0	42.5	45.0	6.2	6.8	7.5

Table 2 Chemical composition (by mass) of sulphoaluminate cement, %

SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	SO ₃	TiO ₂	K ₂ O	Na ₂ O	IL
16.40	25.83	1.09	37.18	1.82	9.22	0.58	0.31	0.16	7.40

temperature during mixing by using high temperature water; 3) Adding air entraining agent into the grout during mixing.

Although the measures above can ensure the minimum curing temperature and early strength gain of grout in precast elements, for long-span, cast-in-place members, it is expensive to maintain the minimum curing temperature in winter. Therefore, it is very important and necessary to develop a new grouting material which needs no heating to ensure proper curing.

In this paper, a new cementitious-based grout, which could be used in winter construction in north China without extra curing measures, is proposed, and its workability and development of mechanical properties are discussed.

EXPERIMENTAL

Raw Materials

Rapid-hardening calcium sulphoaluminate cement from Shanxi Yangquan Tianlong Engineer Materials Co. Ltd was used. Its physical characteristics and chemical composition are listed in Table 1 and 2. Its main mineral components are C₄A₃S (3CaO·3Al₂O₃·CaSO₄), C₃S (3CaO·SiO₂) and C₂S (2CaO·SiO₂), as shown by the X-ray diffraction pattern in Fig. 1.

Special anti-freezing admixtures for cold-weather construction, water-reducing retarding, and accelerating admixtures were also used.

The mix proportion used in this research is shown in table 3.

Procedures

Except water, all the mixing materials and molds were placed in a -10 °C (14 °F) chamber for 24 hours, and then mixed with water. After mixing 1 minute at low speed and 2 minutes at high speed, the specimens were molded and the grout was tested immediately for the flowability and the setting time.

Initial temperature after molding

The initial temperature of the grout was adjusted by varying the water temperature, and five different water temperatures, 20 °C (68 °F), 30 °C (86 °F), 40 °C (104 °F) and 60 °C (140 °F) were selected to mix with the other materials, which were held at -10 °C (14 °F) in advance. The actual temperatures at grout specimens were measured right after molding.

Temperature measurement

Grout temperature was measured by the thermo sensor placed inside the specimen.

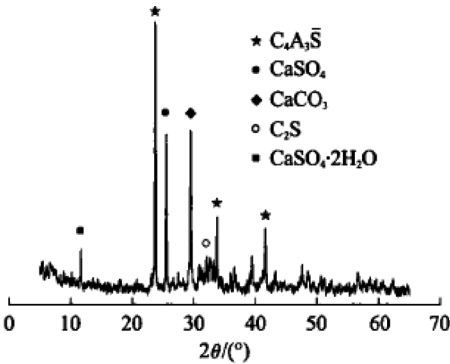


Fig.1 XRD pattern of sulphoaluminate cement

Table 3 Mix proportion

	Water	Cement	Anti-freezing admixtures	Water-reducing and retarding agent	Accelerating admixtures
A1	100	100	0%	2%	2%
A2	100	100	0.2%	2%	2%
A3	100	100	0.4%	2%	2%
A4	100	100	0.6%	2%	2%
A5	100	100	0.8%	2%	2%

The temperature development of reference samples at an environmental of 15 °C(59 °F) and -10 °C (14 °F), as well as the specimens with special treatment and curing in an environmental of -10 °C (14 °F), were measured until 350 minutes.

Flowability test

Flowability was measured by the efflux time from a standard flow cone based on the Chinese Railway standard TB/T3192-2008⁵, “Grouting for Railway Bridge Post-tensioning Tendons Specification”. The flowability test measures the time required to empty a given volume(1725 ±5mL)[105.3 ± 0.3 in.³] of grout through the orifice of a standard flow cone. Fig.2 is the schematic of the flow cone.

Considering of maximum length post-tensioned members of 200m(656.17fts), the flow-ability of grout should less than 30s at 45 minutes after mixing.

Setting time test

Setting time testing was processed according to the test method in Chinese National Standard GB/T50448-2008, “Cementious Grouting Materials Specification”.

Strength test

Three specimens with dimensions of 40mm×40mm×160mm(1.57 in.×1.57 in.×6.30 in.) were molded immediately after mixing and cured at -10°C(14°F). The compressive strength was determined at 8h, 24h (R-1d), 7d (R-7d) and 28d (R-28d). For the optimized test

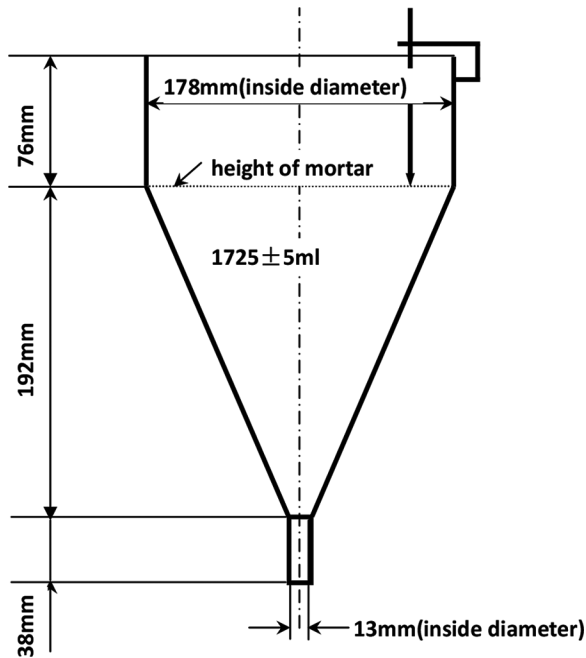


Fig.2 Schematic of flow cone for flowability test

scenario in table 3, strength was also measured after specimens were cured at 20°C(68°F) for 1d (R-7d+3d), 3d (R-7d+1d), 28d (R-7d+28d) and 56d (R-7d+56d) after they had been cured at -10°C(14°F) for 7 days.

RESULTS AND DISCUSSION

Initial temperature after mixing

Fig.3 is the relationship between mix water temperature(water was mixed with the other materials at a temperature of in -10°C(14°F)) and final temperature of the grout A2. From it we can see that the final temperature of grout after mixing will increase as the temperature of mixing water increases. For winter temperatures as low as -10°C(14°F), the minimum mixing water temperature should higher than 25°C(77°F)for a final temperature after mixing of at least 5°C (40°F). Comparing other methods, such as keeping all the materials in a relative high temperature room or heating during mixing, increasing the water temperature before mixing is the most efficient method for providing a minimum grout temperature after mixing.

Temperature measurement in specimens

Fig.4 is the specimen temperature development of a reference sample without special treatment (A1). From it we can see that for the 20°C(68°F) curing environment, the exothermic peak appears at about 70 minutes after mixing. If we put the grout into -10°C(14°F) curing chamber, the temperature of will drop to zero after 10 minutes and start

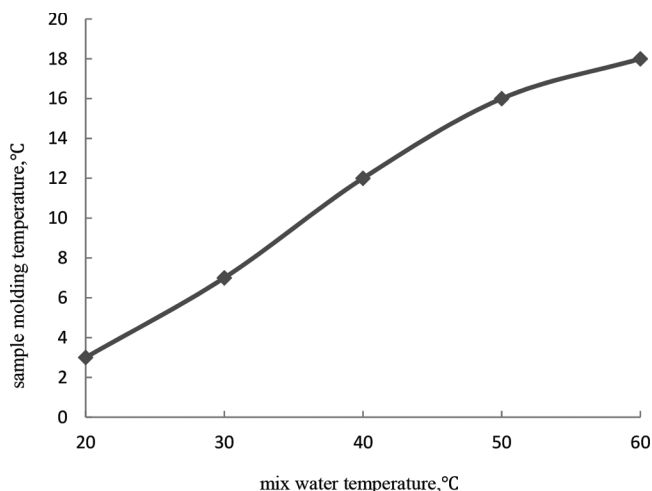


Fig.3 Relationship of mix water temperature and sample molding temperature

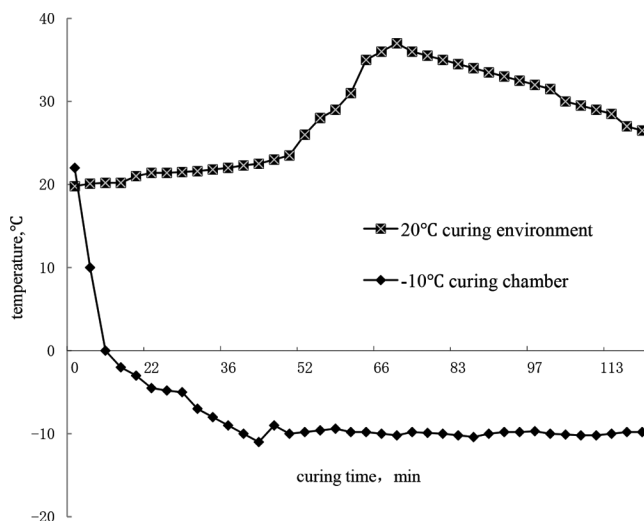


Fig.4 Specimen temperature development in reference sample

to freeze and will reach equilibrium with the environment after 40 minutes. If we took the sample out after 2 hours of curing at the low temperature, it would turn to grout gradually, which means no strength development in this circumstance, as shown in Fig.5.

Fig.6 is the specimen temperature development of a grout specimen with special treatment(A2).From it we can see that after the specimen was placed in the -10°C(14°F) curing chamber, temperature dropped to zero after 10 minutes curing. However, there is



Fig.5 Reference sample's shape change after bring back from negative to positive temperature

an obvious exothermic peak that begins to form at 50 minutes and ends at 140 minutes, which is similar to the reference sample in Fig.4. The time to reach temperature equilibrium is 350 minutes. From this, we can conclude definitely that hydration happened in the specimen and some strength must have developed during this time. Visual evidence of strength development can be seen in Fig.7, in which there is no shape change after the specimen with special treatment was brought back from negative to positive temperature. The sample should be observed in polished section under the microscope to clearly observe if ice formation occurred.

Flowability and setting time

Grout flowability and setting time test results for grouts A1 - A5 with initial molding temperature of 5°C(41°F) and cured at -10°C(14°F) are shown in the Table 4. The flowability of grout with 5 different contents of retarding agent was measured at 0s, 30 min, 45 min, and 1h after mixing. From it we can see that the flowability of the grout was affected by dosage of retarding agent. Without any retarding agent (0%), the grout flowability will be reduced quickly, which is not suitable for field construction. While the higher dosage of retarding agent (0.4% and 0.6%) will ensure flowability is maintained for field use, it will result in strength reduced early and later curing ages. When admixture dosage is as high as 0.8%, the flowability is maintained for so long period that the grout will start to freeze before the setting occurs.

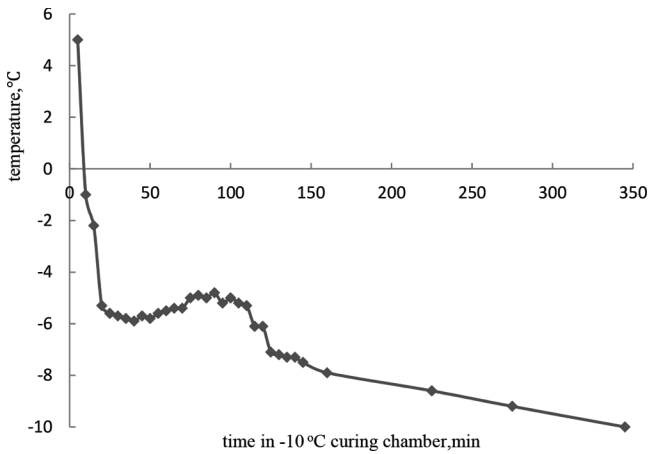


Fig.6 Temperature development in sample with special treatment

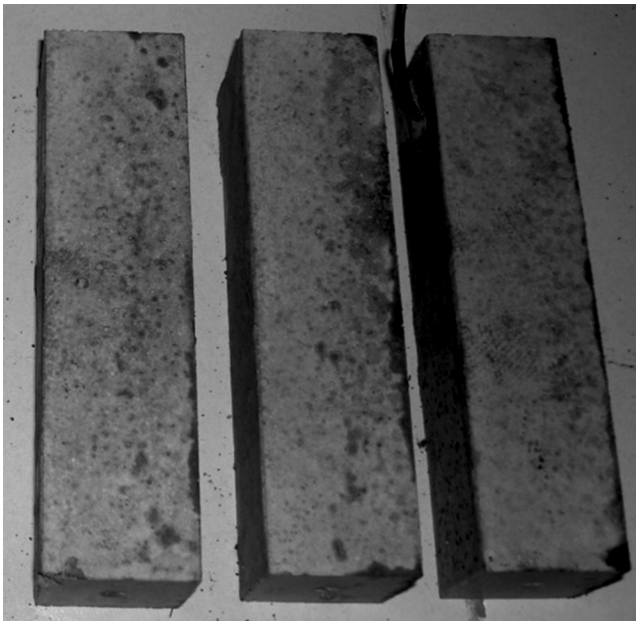


Fig.7 Shape change of specimen with special treatment after going from negative to positive temperature

The best combination of flowability retention and setting time is the grout with 0.2% retarding agent(A2). For this mixture (bold emphasized in Table 4), the flowability will be maintained for 1 hours for curing at -10°C(14°F), and it will set after 105 minutes. Further-

Table 4 Grout flowability, setting time, and strength test results with initial molding temperature at 5°C (41 °F) and cured at -10°C (14°F)

Items		A1	A2	A3	A4	A5
Flowability at 0min/s		16.2	15.3	15	14.5	14.4
Flowability at 30min/s		30	18.6	15.5	16	15
Flowability at 40min/s		0	22	18	19.2	16.2
Flowability at 1hr/s		—	24	20	20.2	17.2
Setting time, min		50	105	150	300	Frozen
Strength (MPa)	R-1d	37.8	33.4	31.5	22.8	
	R-7+28d	64.5	62.5	58.4	48.3	
	R7d	54.3	52.2	50	45.5	
	R28d	65.2	64	60.3	50.6	

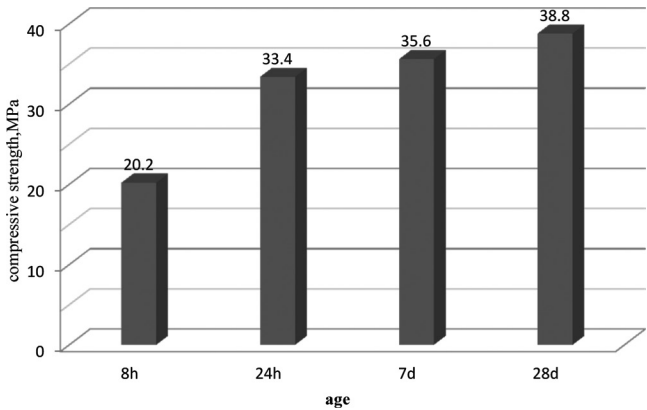


Fig.8 Strength development of A2 at -10°C (14 °F)

more, its strength at every curing ages(R-1d, R-7+28d, R+7d, R+28d) are relatively high and enough for the construction requirement.

Strength

Specimens of grout with 0.2% retarding agent (A2) were molded and cured at -10°C(14°F), and strength was measured at 8h, 24h, 7d and 28d. Other specimens were cured at -10°C(14°F) for 7 days and then moved to a 20°C(68°F) chamber for further curing for 1d, 3d, 28d and 56d.

Fig.8 shows strength development at -10°C(14°F). From it we can see that strength increased rapidly within the first 24 hours and increased gradually with extended curing time at -10°C(14°F). Even so, the strength at 24 hours was 33.4 MPa (4843 psi) which is higher than the demolding demand from construction requirements.

For specimens cured initially at -10°C(14°F) and then at 20°C(68°F), the rate of strength gain obviously increased when switched to the 20°C(68°F) chamber, as is in Fig.9. The strength developed very fast in the first 3 days at 20°C (68°F) and reached 56.0 MPa (8120 psi) after 3 days at the higher temperature. After that, the strength developed slowly and

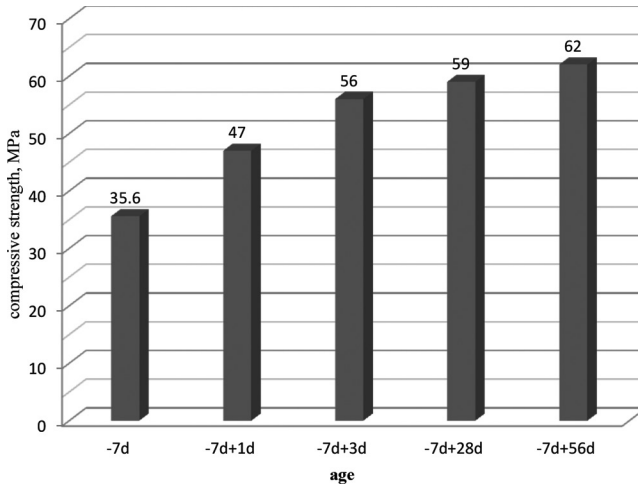


Fig.9 Strength development of combined curing of A2 at 20°C (68°F) after 7 days processed at -10°C (14°F)

stabilized. At end of the 28 days and 56 days curing at 20°C (68°F), the strength is 59.0 MPa (8555 psi) and 62.0 MPa (8990 psi), which is greater than 50.0 MPa (7250 psi), the minimum grout strength requirement according to construction standard.

SEM Test

SEM test has been processed on the specimens of grout with 0.2% retarding agent (A2) at age R-7+28d, the results have been shown in fig.10 and fig.11. From it we can see the needlelike ettringite crystal and flocculent C-S-H gel were the main hydration products of A2 grout after curing at -10°C for 7 days and then moved to a 20°C for 28 days.

The needlelike ettringite crystal were main obtained by the hydration of C_4A_3S , and the reaction is shown in Eq.(1).⁶



The flocculent C-S-H gel were mainly formed from the hydration process of C_3S and C_2S . The hydration equations are shown in Eq.(2) -(3).⁷



Field Case

Fig.12 and fig.13 show an example of field application of this special modified grout for post-tensioned beams in Datong-Xi'an high-speed railway. The maximum length of grouting in this case is 200 meters (656 ft), and the whole grout mixture was continually finished at one time. By using this specially modified grout mixture, there is no need for

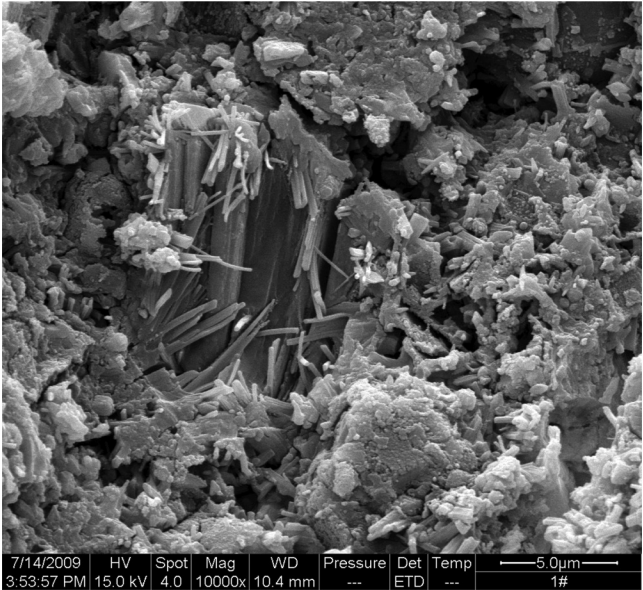


Fig.10 Ettringite crystal structure in the A2 grout at R(-7+28)d

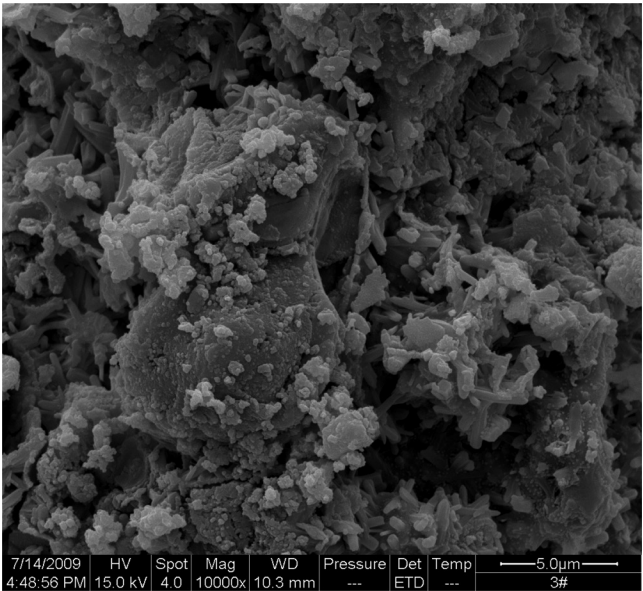


Fig.11 Flocculent C-S-H gel in the A2 grout at R(-7+28)d



Fig.12 Construction of post-tensioned beams in Datong-Xi'an high speed railway in winter



Fig.13 Construction of post-tensioned beams in Datong-Xi'an high speed railway in winter

the heating equipment in the field, which makes construction much easier and cheaper. Compared with the traditional methods for grouting in cold weather, at least 80% overall costs could be saved.

CONCLUSIONS

A grout mixture that can be used at -10°C (14°F) was proposed in this paper. After 1 day of curing at subzero temperature, the strength could reach 33 MPa (4785 psi). After

switching to standard curing temperature for 28 days, the final strength could reach 55 -60 MPa (7975-8700psi), which represents 95%-100% of the strength for curing at standard temperature.

The new grout mixture has been used in field in winter conditions. Compared with the traditional heating methods, it could reduce energy consumption and save at least 80% of capital costs in total.

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Case Studies of CO₂ Utilization in Concrete

by Sean Monkman and Mark MacDonald

SYNOPSIS

While the cement industry is currently working on a number of initiatives to reduce CO₂ associated with cement production (i.e. thermal and electrical efficiency, alternative fuels and clinker substitution) a significant CO₂ mitigation is still desired and will require novel solutions. The reaction of carbon dioxide with hydrating cement is known to produce stable carbonate reaction products. These reactions can serve to effectively bind CO₂ in the cement matrix during the fresh state while offering material benefits. Carbonation treatments have been integrated into industrial concrete production. Producers can sequester carbon dioxide in their products and more favorably position concrete in a competitive green building product marketplace. The case studies examine the application of carbon dioxide to concrete masonry production (strength, absorption and density) and ready-mix concrete production (workability, strength). The carbon dioxide integration has minimal impact on conventional process operations and works with existing production equipment. Estimates of the net environmental impact are made.

Keywords: carbonation; carbon dioxide; concrete masonry unit; ready-mixed concrete; sustainability; upcycling.

INTRODUCTION

Carbon dioxide emissions are recognized as a significant issue for the cement and concrete industry. It is estimated that 5% of the world's annual CO₂ emissions are attributable to cement production (Damtoft et al. 2008). Portland cement clinker typically has embodied CO₂ on the order of 866 kg CO_{2e}/tonne of clinker (2103 lb CO_{2e}/ton of clinker) (Cement Sustainability Initiative 2009). Cement production releases CO₂ due to the calcination of limestone that is heated to drive off CO₂ and yield reactive CaO phases. Carbon dioxide emissions are further associated with the energy required to operate the cement kiln. About 40% of the process emissions are associated with the energy consumption and 60% are associated with the calcination.

The resulting cement clinker will then typically be blended with additional materials (i.e. blast furnace slag, fly ash, silica fume) to create a blended cement. If the substitution rate of the clinker reaches 22% (the world average in 2006 (Cement Sustainability Initiative

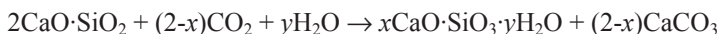
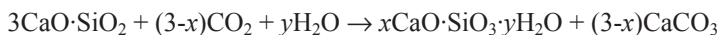
2009)) then the specific emissions intensity can be driven down from about 866 kg/tonne of clinker to around 670 kg/tonne (1627 lb CO_{2e}/ton of clinker) of blended cement.

A number of approaches have been identified to reduce the emissions intensity of the cement produced and used (IEA 2009). The thermal and electrical efficiency of cement production can be improved by deploying the best available technology in new cement plants and retrofits. Alternative and less carbon-intensive fuels can be used as the energy source. The rate of substitution in blended cements can be maximized. Finally, carbon capture and storage (CCS) can capture cement industry CO₂ emissions before they are released and store them permanently.

It is clear, however, that practical limits on the impacts of these measures mean that it will be difficult (Barcelo et al. 2014) to attain the industry goal to reduce emissions 50% below 2006 levels by 2050 that is outlined in the IEA & WBCSD roadmap (IEA 2009). Innovative approaches are sought and are likely to be a part of a portfolio strategy. The most significant improvements in efficiency and cement substitution are likely to be already known and available. Future emissions improvements will likely be incremental. Thus, a range of further approaches will also have to be pursued.

One potential method is to upcycle carbon dioxide into concrete products. If an industrial process could successfully use carbon dioxide as a feedstock in the production of concrete building products then there would be widely distributed carbon utilization that would effectively ‘close the loop’ for some of the carbon dioxide emitted during the cement production.

The mechanism of the carbonation of freshly hydrating cement was systematically studied in the 1970s at the University of Illinois. The main calcium silicate phases in cement were shown to react in the presence of water to form calcium carbonate and calcium silicate hydrate gel (Berger et al. 1972)



The carbonation reaction is exothermic. The carbonation heats of reaction for the main calcium silicate phases are 347 kJ/mol for C₃S and 184 kJ/mol for β-C₂S (Goodbrake et al. 1979). The calcium silicate hydrate (C-S-H) gel that forms is understood (Berger et al. 1972) to be intermixed with CaCO₃. C-S-H gel formation occurs even in an ideal case of β-C₂S and C₃S exposed to a 100% CO₂ at 1 atm given the observation that the amount of carbonate that forms does not exactly correspond to the amount of calcium silicate involved in the reaction (Goodbrake et al. 1979).

Since carbonation reactions proceed starting with the dissolution of carbon dioxide in water to form carbonic acid, the initial vigorous carbonation stages of carbonation can be described accordingly (Young et al. 1974):



The reaction of carbon dioxide with a mature concrete microstructure is acknowledged as a durability issue given effects such as shrinkage, reduced pore solution pH, and carbon

ation induced corrosion. In contrast, a carbonation reaction integrated into concrete production reacts CO₂ with freshly hydrating cement, rather than the hydration phases present in mature concrete, and does not have the same effects.

RESEARCH SIGNIFICANCE

Carbon dioxide upcycling during concrete production effectively offers permanent CO₂ storage in the concrete product allowing producers to create a more sustainable material with improved performance. If carbon dioxide can be beneficially absorbed into fresh concrete there will be an attractive new tool for reducing the carbon footprint of the built environment and to help the concrete industry compete with other construction materials in the green building space. The authors believe that the described research regarding a novel carbon dioxide utilization integrated into the production of concrete masonry units and ready-mixed concrete will offer environmental benefits.

EXPERIMENTAL INVESTIGATION

A series of industrial scale experiments were performed in 2013 whereby gaseous carbon dioxide was injected into concrete products during their batching and mixing. The carbonation was integrated with both concrete masonry unit manufacture and ready-mixed concrete production. The goal was to assess the potential of using the concrete to sequester carbon dioxide during production and to determine the attendant material science impacts. Carbon dioxide uptake was assessed to determine how much gas became locked in the concrete as stable reaction products.

A tank of liquid CO₂ was connected to a gas control system and manifold. The liquid was expanded to a gas and metered for injection into the mixer (block) or mixing drum of the ready-mixed concrete truck. The gas was delivered, at a specified flow rate over a fixed injection interval, whereupon it reacted with the hydrating cement before the mixing was completed.

Use of industrial gas

The carbon dioxide used in the experiment was purchased as liquid CO₂ from an industrial gas supplier. Industrially supplied carbon dioxide, almost without exception, is captured as a byproduct from an industrial process (e.g. ammonia, fertilizer or ethanol production). Consequently, carbon dioxide purchased from an industrial supplier represents emitted carbon dioxide that has been diverted and captured to meet market demand. Any carbon dioxide captured in the concrete carbonation process using industrially supplied carbon dioxide can represent a net utilization benefit. Any carbon dioxide from the carbonation process that is lost to the atmosphere (as opposed to being absorbed by the concrete) is taken to be returned to the destination it had prior to the industrial capture.

The use of purchased carbon dioxide does come with an environmental cost given that the gas must be captured, liquefied, and transported. Therefore net utilization will have to be assessed on a case by case basis according to the specific capture scheme and transportation requirements.

Table 1–Mix design used at the concrete masonry block trial

	Amount per m ³ (per yd ³)	Fraction
Birdseye washed gravel, kg (lb)	311 (524)	8.7%
Torpedo sand, kg (lb)	1960 (3304)	54.8%
Silica sand, kg (lb)	567 (956)	15.9%
Cement, kg (lb)	454 (765)	12.7%
Rainbloc 80, ml (oz)	1478 (38)	—
Mix water	—	7.9%

Masonry unit trial

The block experiment was conducted at an Illinois-based masonry concrete producer. The carbon dioxide injection equipment was retrofit onto the conventional block production line. The carbon dioxide was injected into the concrete mixer. The gas was injected once all the dry materials had been loaded and was completed in 3 minutes. The duration was ultimately limited by the default mixing time given the intention to avoid extending the process cycle time. A medium weight mix design (Table 1) was used to make standard 8” (200 mm) concrete blocks. Carbon dioxide was injected at a rate of 0.5 to 1.5% by weight of cement.

The fresh concrete was assessed visually and via feedback from the machine (i.e. compaction time) and production personnel. Samples were taken periodically to assess the water content of the concrete and monitor whether the carbon dioxide had any impact on the “wetness” of the fresh product. The mix water was adjusted as deemed necessary by the operator. The finished blocks were transported to a curing room heated with forced humid air. The temperature is maintained at 95 to 100 °F (35 to 38 °C) and the relative humidity between 85 and 90%. Blocks are typically held for 3 or 4 days in the curing room.

Concrete performance was assessed through compressive strength at 7, 28 and 56 day testing at the producer’s testing laboratory with 5 blocks for each carbonated condition at each test age. Water absorption and density testing was also conducted on three blocks per condition. Testing was conducted according to ASTM C90 - Standard Specification for Loadbearing Concrete Masonry Units. The control baseline was comprised of two batches contributing 8 strength specimens (5 from the first batch, 3 from the second) and 6 absorption/density specimens.

Ready-mixed concrete trial

The ready-mixed concrete experiment was conducted at a Dartmouth, Nova Scotia ready-mixed concrete producer. It is known, from preliminary lab testing, that the reaction rate between carbon dioxide and cement increases with decreasing w/c. For experimental purposes, the mixing sequence was slightly altered to promote the carbonation reaction in the mixing drum. The mix water was reserved until after the carbon dioxide injection was complete. The amount of mix water added to the truck mixer for the control batch was quantified by timing a manual addition with a hose. The same amount of water was added to the subsequent carbonated batches. All of the aggregates and binder were loaded and mixed briefly before the carbon dioxide was injected. CO₂ was delivered over 180 seconds before the water and admixtures were added. Carbon dioxide was injected in two trials at

Table 2—Mix design used in ready-mix concrete trial

	Amount per m ³ (per yd ³)	Fraction
Sand, kg (lb)	770 (1298)	33.5%
20 mm Stone, kg (lb)	1030 (1736)	44.8%
GU Cement, kg (lb)	336 (566)	14.6%
Water, kg (lb)	163 (275)	7.1%
Daracem 50, ml (oz)	1350 (35)	
Darex II, ml (oz)	~200 (5)	

1.0% and 1.5% by mass of cement by means of a gas delivery line retrofit onto the truck mixer.

The trial considered batches with a constant amount of water rather than aiming for a specific slump. The mix design targeted 27.5 MPa (4000 psi) and is reported in Table 2. The batches were 2 cubic meters (2.6 yd³) and mixed in an 8 cubic meter (10.5 yd³) truck. The weather during the trial was about 23 °C (73 °F) and clear. The concrete was used in a precast operation on site and was delivered to the testing location 20–25 min after the mixing started. The fresh concrete was characterized in terms of temperature, slump (ASTM C143), and air content (ASTM C231). The compressive strength of the hardened concrete was assessed at 1, 7, 28 and 56 days by testing 3 cylinders per condition according to ASTM C39.

Carbon quantification

The samples for analyzing the carbon dioxide content of the concrete were created by taking a fresh sample from the production line, drying the concrete on a hot plate to remove the water, and subsequently sieving the material through a 160 µm sieve. Samples of the raw materials were examined to determine how much of each component passes a 160 µm sieve and the carbon content of the passing material. This information, along with the concrete mix design, allows for the calculation of a theoretical control carbon content against which analyzed samples can be compared.

The samples were then analyzed via the combustion infrared detection method using an Eltra CS 800 elemental analyzer. A 0.1 - 0.2 gram sample of sieved material is loaded into the induction furnace of the analyzer where it is heated to ~2500 °C in a pure oxygen environment. During the combustion process all carbon-containing compounds react with the available oxygen to produce carbon dioxide gas (CO₂). This gas is then passed through an infrared detection cell. Based on the amount of measured CO₂ the analyzer then calculates the original carbon content of the sample. The carbon content can be recalculated to be an equivalent CO₂ content.

EXPERIMENTAL RESULTS AND DISCUSSION

Carbonated concrete masonry unit trial

The start of the trial injected carbon dioxide into the concrete mix without making any attendant mix or process adjustments. The operator concluded that the carbonated mix required additional water and the procedure was modified to include an assessment of the batch in the mixer prior to discharge. Additional mix water was added based upon visual assessment of need. Three carbonated batches were produced with carbon dioxide dosages

Table 3–Test batch summary at the concrete masonry block trial

	Control	CO ₂	CO ₂	CO ₂
Code	Avg	602	603	605
CO ₂ dose by weight cement	-	0.5%	1.0%	1.5%
CO ₂ uptake	-	0.28%	0.54%	1.40%
Uptake efficiency	-	57%	54%	93%
w/c by control sensor	0.399	0.430	0.440	0.455
Temperature, °C (°F)	26.5 (79.7)	28.1 (82.6)	29.6 (85.3)	31.8 (89.2)

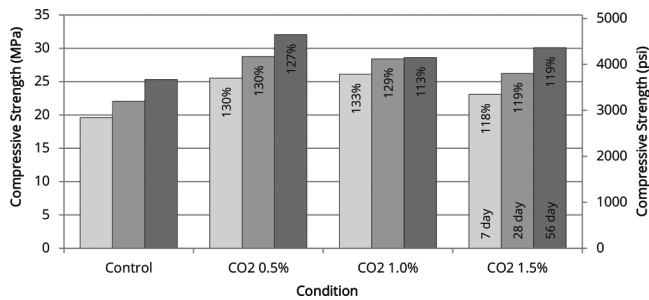


Fig. 1–Masonry block compressive strength results

of 0.5%, 1.0% and 1.5% by weight of cement. The outline of batches, temperate and carbon uptake results are summarized in Table 3. The overall mix water increases, as calculated based upon a w/c sensor on the batching system, were 8, 10 and 14% for the three carbonated batches respectively. The temperature increase observed in the carbonated batches is indicative of the exothermic nature of the carbonation reaction and can be seen to increase with the supplied CO₂ dose. The carbon uptake of the three batches was shown to be more efficient at the highest dose wherein 93% of the supplied CO₂ was found to be absorbed by the concrete.

The strength development of the test batches is shown in Fig. 1. The low dose of CO₂ provided 30% strength increases at the two early ages and 27% at 56 days. The middle dose showed a 33% benefit at 7 days, 29% at 28 days and 13% at 56 days. Finally, the high dose resulted in a strength increase in the range 18-19% at all three ages.

The density (Fig. 2) was equivalent to the control for the two higher doses and 2% higher for the lowest dose. The absorption was improved (Fig. 3) in all three carbonated batches with reductions of 36%, 34% and 18% over the three increasing dosages.

The performance of the two higher doses of CO₂ (10-14% increase in mix water, equivalent density to the control, 29% strength increase at 28 days, 34% better absorption) avoids attributing any difference to an improved block density. The addition of the CO₂ may have had a chemical impact that contributed to a strength increase, in a general sense it may have allowed a higher water content in the block and increased the overall hydration. The conclusion is that the carbon dioxide injection can be simply integrated as a retrofit into a conventional block production line and unlock material benefits alongside achieving permanent storage of CO₂ into the blocks.

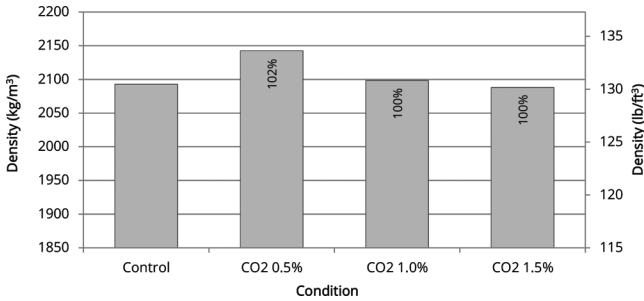


Fig. 2—Masonry block density results

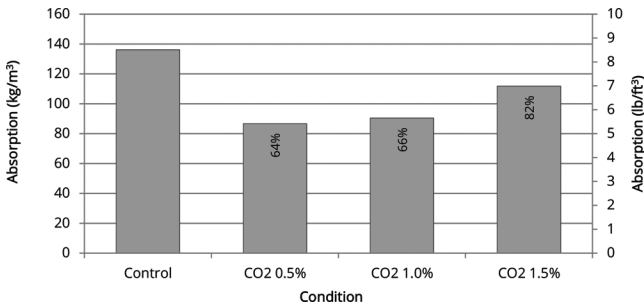


Fig. 3—Masonry block absorption results

Carbonated ready-mixed concrete trial

Three batches were produced during the ready-mix test program - a control, CO₂ dosed at 1.0% of the weight of the cement, and CO₂ dosed at 1.5%. The concrete subjected to the lower dose contained 0.44% CO₂ by weight of cement (44% efficiency) while the higher dose concrete contained 1.69% (or a 113% efficiency). It was concluded, lacking any reasonable interpretation of the inflated efficiency, that the quantification procedure would benefit from more experience with the industrial mix and materials or generally requires further refinement. The carbon quantification of concrete produced in lab studies has been consistent so the expectation is that the approach and technique can be viable. A range of subsequent industrial trials have shown that the efficiency of uptake often increases with supplied dose and thus the markedly different efficiencies here are somewhat expected.

The fresh concrete properties are summarized in Table 4. It was shown that, noting that the mix water was held constant between batches, the initial slump decreased with increasing dose of CO₂. The concrete temperature showed an increase due to carbonation.

Although the three batches had the same amount of mix water it was seen observed that less water was detectable in the carbonated concretes during drying on a hot plate as part of the carbon quantification sample preparation. The control concrete was 9.6% water by mass, while the 1.0% CO₂ batch was 9.2% and the 1.5% was 8.5% (reported as averages of two samples). The reduced observable water content of the carbonated batches is likely

Table 4–Mix properties at ready-mix concrete trial

	Control	1.0% CO ₂	1.5% CO ₂
Air Content	5.9%	5.9%	4.5%
Slump, mm (in)	178 (7)	102 (4)	76 (3)
Temperature, °C (°F)	25.8 (78.4)	28.1 (82.6)	28.6 (83.5)

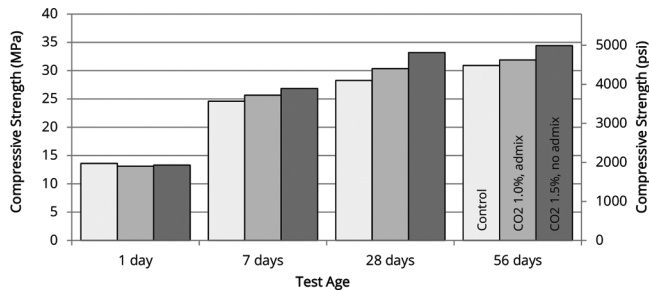


Fig. 4–Strength development of the trial ready-mix concrete mixes

related to an increase in the chemically bound water and/or increased adsorbed water on new solid reaction products.

The air content of the first carbonated batch matched that of the control concrete while the second was notably lower. Further testing would be required to make a definitive conclusion though the possibility is noted that the carbonation reaction may decrease the air entrainment.

The temperature of the two carbonated concretes were very close. While it is currently only a semi-quantitative indication of carbonation uptake, the temperature evidence supports the interpretation that the carbonate quantification was questionable and needs added rigor. If the temperature increase was close then the total uptake was anticipated also to be close.

The reduced slump of the carbonated concrete observed in the 0.44% CO₂ batch prompted the use of an experimental admixture. It should be noted that conventional plasticizing admixtures, as tested in the lab, had not always had the desired effects when used with the carbonated concrete. The admixture was added to the truck after the first slump measurement of 4 inches. It was found that when the slump was tested 20 minutes later it was 5.5 inches. The result is a proof of concept that if the slump of a carbonated concrete is too low then it can be improved with the addition of an admixture after the CO₂ treatment. The second batch had an even lower slump than the first but the precast operation deemed it acceptable and used it as normal.

The compressive strength development of the concrete produced during the trial is presented in Fig. 4. The strength of the 1.0% CO₂ dosed batch was 96% of the control at 1 day, and between 103% to 107% at the three later ages. The strength specimens were produced prior to the addition of the admixture to the batch. The magnitude of the difference in the strength results suggest that the control and the CO₂ batch dosed at 1.0% were effectively equivalent.

Table 5—Net carbon utilization estimates for industrial trials

	Per 1125 blocks	Per 8 m ³ (10 yd ³) ready-mix
Cement content, kg (lb)	2569 (5663)	2560 (5396)
CO ₂ uptake efficiency	86%	80%
CO ₂ absorbed, kg (lb)	33.1 (73.1)	25.6 (54.0)
Emissions – gas processing kg (lb)	3.9 (8.7)	0.01 (0.03)
Emissions – gas transport, kg (lb)	0.6 (1.3)	6.0 (12.7)
Total CO ₂ produced, kg (lb)	4.5 (10.0)	6.0 (12.7)
Net CO ₂ absorbed, kg (lb)	28.6 (63.1)	19.6 (41.3)
Net efficacy, %	86.3	76.5

The 1.5% CO₂ dosed batch was 98% of the control strength at 1 day, 98% at 7 days, 117% at 28 days and 111% at 56 days. It appeared that the carbonated batch had a greater later strength than the control. The air content, as previously noted, was lower for this batch than for the control and a possible contributing factor to the strength performance.

Net environmental benefit

An estimate of the net environmental benefit is summarized in Table 5. The analysis considers the amount of carbon dioxide absorbed by the concrete compared to the emissions associated with the processing and transport of the total dosed amount. Further factors would have a very small impact on an exhaustive carbon footprint calculation (such as emissions associated with manufacture and transport of the gas delivery equipment) but the two factors that are the most, by far, are included. No energy usage is associated with the vaporization of the liquid CO₂ to become gas if an ambient gas vaporizer is employed.

The masonry analysis is considered in terms of 1125 blocks or the nominal amount of blocks required to construct a 1000 ft² (93 m²) wall. The blocks are assumed to be 17.7 kg (39 lb) and contain 12.9% cement by total fresh mass. The total cement usage is 2569 kg (5663 lbs). The analysis considers a 1.5% by weight of cement dose of CO₂ that is absorbed at 86% efficiency (a representative value determined for that mix design and dosage after further testing, or a 1.29% uptake by weight of cement).

The industrial gas processing to create the liquid CO₂ requires on the order of 150 kWh/tonne CO₂ (464,402 BTU/ton CO₂) with an electrical emissions rate that is dependent upon the location in which the work is performed. Cargo transport of the total dosed CO₂ in liquid form is estimated according to an emissions rate of 222 g CO₂/ton·mile of cargo (United States Environmental Protection Agency 2011) (alternately 245 g CO₂/tonne·mile or 152 g CO₂/tonne·km). The masonry trial took place 101 km (63 miles) away from the nearest industrial source of CO₂ in Whiting, IN. The 2010 CO₂ emission rate for electrical power produced in that region was 682 g CO₂/kWh (1503 lb CO₂/MWh) (United States Environmental Protection Agency 2014).

For the masonry case it was found that the net CO₂ utilization in a generic wall that would be produced using blocks from the masonry trial would amount to 28.6 kg (63.1 lbs). The net utilization amounts to 25.4 g (0.9 oz) per block. The CO₂ emissions associated with the production of the cement in the wall (assuming a clinker emissions intensity of 866 kg/t and a clinker content in the cement of 94%) are about 2091 kg (4610 lbs). The absorbed CO₂ (28.6 kg) represents a direct offset of 1.37% of the cement production emissions (2091 kg).

The ready-mixed analysis is considered in terms of a single ready-mixed truck (alternately 8 m³ and 10 yd³ of concrete) and a concrete mix that is 320 kg cement/m³ (539 lbs cement/yd³). The ready-mix analysis assumes that 1% by weight of cement CO₂ uptake is achieved from an 80% efficient dose (a generic number identified as reasonable for carbonation treatments in enclosed mixers provided the dose and dosage rate is leading to excessive gas loss). The gas transport is considered in parallel terms to what was shown for the masonry case. The closest CO₂ source to the ready-mix trial is in Varennes, QC. The 2011 CO₂ emissions rate for electrical power in Quebec was 2.3 g CO₂/kWh (5.1 lb CO₂/MWh) (Environment Canada 2014). The gas supplier is 1235 km (767 miles) from the trial location.

The ready-mix net calculation indicated that a net of 19.6 kg of CO₂ would be absorbed in a 8 m³ truck load of concrete and 41.3 lbs in a 10 yd³ load. The net utilization is 2.4 kg CO₂/m³ of concrete (4.1 lbs CO₂/yd³). The CO₂ emissions associated with the clinker production (using the same assumptions described in the masonry case) would be about 2084 kg in 8 m³ and 4392 lbs in 10 yd³. The absorbed CO₂ (19.6 kg) represents a direct offset of 0.94% of the cement production emissions (2084 kg).

It is shown in comparing the two cases that transportation of the CO₂ has the greater impact than the gas processing. The long term outlook for CO₂ utilization suggests that technological development will reduce the costs and energy of capture and allow for smaller scale, more widely distributed, CO₂ capture will occur. For example, a sensitivity analysis of the ready-mix gas supplier can alternately consider if the gas could have been transported from a source in Nova Scotia (using a nominal distance comparable to the masonry case of 101 km or 63 miles) and used the grid emissions intensity for Nova Scotia (790 g CO₂/kWh or 1742 lb CO₂/MWh in 2011 (Environment Canada 2014)). The net utilization efficacy would be improved from 78.5 to 83.3% if the gas transport distance could be cut 92% despite the emissions intensity of the gas processing increasing 304-fold.

FURTHER RESEARCH

The industrial experiments demonstrated carbon dioxide could be injected into masonry and ready-mixed concrete during batching and mixing to sequester CO₂ into the paste fraction of the concrete. Concrete properties have been observed to be generally comparable to the control in service with some possible benefits.

The industrial trials have inspired further laboratory development. Work is ongoing and has several aims.

1. Increase the absorbed CO₂ content to improve the net environmental benefit offered by the utilization.
2. Conduct a broader study to assess more properties of the concrete including compressive strength, flexural strength, drying shrinkage, abrasion, RCPT, service carbonation, salt scaling, pore solution chemistry, bulk diffusion and ASTM G109 corrosion testing.
3. Characterize the mechanism and morphology of carbonate reaction product formation. A deeper understanding of the physiochemical aspects of the carbonation reaction can determine the best practices and applications of the technology. It is clear that a carbonation uptake between 0.5 and 1.0% by weight of cement is sufficient to markedly effect block compaction or concrete rheology, as appropriate.

CONCLUSIONS

Two industrial trials investigated the performance of concrete produced utilizing carbon dioxide gas. The masonry blocks were shown to absorb up to 1.40% CO₂ by mass of cement. Strength was improved about 18% for the highest dose and up to 30% for lower doses. Absorption was reduced significantly.

The addition of gaseous carbon dioxide to a ready-mixed concrete production cycle achieved beneficial utilization of carbon dioxide in the concrete. The fresh concrete properties were impacted such that the slump was lower for the carbonated concrete. If the mixing water is unchanged versus the control baseline then the lower slump can be addressed through an admixture addition. Alternately, increased mixing water may be required to reach the target slump but at the expense of slightly decreasing the compressive strength. Strength in service was comparable at all ages with a possible 17% benefit seen at 28 days on the higher dosed batch.

An analysis of the carbon footprint suggests that the carbonation treatment for masonry could offer a net utilization of 25.4 g (0.9 oz) CO₂ per block and a direct offset of 1.37% of the emissions from the cement production. A parallel analysis of the ready-mix case suggests a direct net utilization of 2.4 kg CO₂ per m³ (4.1 lbs per yd³) and a direct offset of 0.94% of the emissions from the cement production. The net utilization is sensitive to how far the gas must be transported.

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Effect of Steam Curing on Mechanical Properties of Self-Compacting Concrete Containing Pozzolan

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The effect of initial steam curing and different types of mineral additives (Metakolin, Pumis, and Trass) on mechanical and durability characteristics of self-compacting concrete (SCC) was investigated and evaluated by various test methods. Concrete specimens were exposed to six different steam curing regimes after casting, namely two different temperatures (60, and 70 °C), and three total durations (16, 18, 20 hours). Results of surface electrical resistivity, and Rapid chloride penetration test (RCPT) for durability, and compressive strength for mechanical properties showed that exposing specimens to higher temperature could significantly improve compressive strength by promoting hydration.

Keywords: compressive strength; initial steam curing; mineral additive; rapid chloride penetration test; self-compacting concrete; surface electrical resistivity.

INTRODUCTION

Self-Consolidating Concrete (SCC) was first developed at the University of Tokyo, Japan¹ in order to reduce some construction problems; especially in urban regions where vibration for consolidation is not allowed.² Besides, due to little distances between bars in some structures, especially pre-tensioned bridges ordinary concrete may not spread all over the reinforcement bars. On the other hand because of traffic congestion which traps truck mixers, there is always need for maintaining slump for long distances. Hence, SCC can be categorized as high performance concrete due to its several abilities such as high flowability, passing ability through the work without any segregation and without any need of vibration because of adding water reducers (WR), and viscosity modifying admixtures (VMA),^{3,4} while maintaining the ability to flow more than ordinary concretes.

Some projects have to be done in a short period of time, especially in urban regions.⁵ The rate of concrete element production hence, plays an important role in the development of the whole project since molds have to be demolded as soon as possible. Therefore,

Table 1—Mix design of SCC

Label	Binder (kg/m ³)		Aggregate (kg/m ³)			FA/TA (%)	Water (kg/m ³)	Superplasticizer dosage (%)
	Cement	Filler	Fine Agg. (FA)	Coarse Agg. (CA)	Total			
M	400	100	1006	697	1703	0.59	190	0.89
P	400	100	1006	697	1703	0.59	190	0.82
T	400	100	1006	697	1703	0.59	190	1.25
L	400	100	1006	697	1703	0.59	190	0.91

Note: 1 kg/m³ = 0.063 lb/ft³

initial heat or steam curing (IHC/ISC) at atmospheric pressure highly needed to achieve a considerable compressive strength so that molds can be used for another casting. Some researchers have mentioned that steam curing at low pressure could increase the strength development of mortars⁵; it has also been stated that the temperature of steam curing suitable for the strength of concrete was 60°C, and more pre-curing can increase the strength as well⁶; however, few studies have been performed on the effect of different steam curing temperatures on mechanical and durability properties of SCC.

RESEARCH SIGNIFICANCE

In this paper, the effect of several steam curing regimes and different types of mineral additives on compressive strength and durability of SCC was investigated by applying 60 and 70 °C steam curing temperature with the total steam curing durations of 16, 18, 20 hours. Rapid chloride penetration test (RCPT), and surface electrical resistivity tests were performed for assessing durability of concrete specimens. It was also concluded that if available, Pumis can be used as a natural mineral additive and is as effectively as Metakaolin.

EXPERIMENTAL INVESTIGATION

The mixture design used in this study is the design applied in the production of precast segments for a real project in which the consultant for the project has defined a minimum initial compressive strength of 24 MPa that segments should obtain before demolding. Water/Binder ratio was held constant for all mixture designs at 0.38, and in order to be able to compare the influences of mineral additives, the contents of mineral additives were used at 20% of cementitious weight for all mixture designs. The mixture designs of SCC specimens are presented in **Table 1**. In this table, M, P, T, and L stand for Metakaolin, Pumis, Trass, and Limestone respectively. Also, “M-60-16” signifies that the specimen containing Metakaolin was exposed to 60°C steam curing for 16 hours. For all mixtures, the amount of total aggregates was maintained constant at 106.31 lb/ft³ (1703 kg/m³), limestone was used as filler, and different dosages of superplasticizer was used to meet SCC characteristics regarding EFNARC.⁷ Hence, superplasticizer dosage was not held constant; therefore, it varied from 0.5% to 1.5%.

Materials

A typical type II Portland cement conforming to ASTM C150⁸ was used. Crushed limestone was used as mineral admixture and fine materials. Metakolin, a thermally treated mineral additive, Pumis, and Trass were used as supplementary cementitious materials in

Table 2—Chemical and physical characteristics of the cement and filler

Chemical Composition (%)		Cement (C)	Limestone (L)	Metakaolin (M)	Pumis (P)	Trass (T)
CaO		62.08	50.17	3.46	4.6	3.96
SiO ₂		21.10	3.12	74.9	64.9	67.2
Al ₂ O ₃		4.18	1.19	17.2	16.9	12.5
Fe ₂ O ₃		3.34	0.53	0.82	3.85	2.19
MgO		3.79	3.46	0.23	1.34	1.58
SO ₃		2.84	0.20	0.45	0.36	0.068
K ₂ O		0.69	0.25	0.39	1.15	1.8
Na ₂ O		0.14	-	-	4.7	1.8
Pozzolanic Activity at 7 days		-	-	0.84	0.76	0.57
Pozzolanic Activity at 28 days		-	-	0.98	0.85	0.68
Loss on ignition(%)		3.12	40.31	2.53	2.2	8.5
Physical properties	Specific gravity	3.17	2.48	2.53	2.38	2
	Blaine fineness (cm ² /g)	3519	6500	10500	4800	3200

Note: 1 cm²/g = 70.3 in²/lb

order to compare their effects with limestone. The chemical and physical characteristics of the cement, filler, and the mineral additives are presented in **Table 2**. Natural river sand with maximum aggregate size of 0.19 in. (4.75 mm) as fine aggregate, and with maximum aggregate size of 1 in. (25 mm) as coarse aggregate were used. Potable water was used for casting and curing the specimens. A liquid polycarboxylic ether-base high range water reducer (HRWR) with solid contents of 47% and specific gravity of 1.1 was used in all concrete mixtures as superplasticizer.

Samples

More than 300 3.94in. x 3.94in. (10cmx10cm) cubic specimens were cast in order to measure compressive strength according to ASTM C39,⁹ which then were tested after curing, at the ages of 1, 7, 28, and 90 days. The test was carried out on 3 specimens for each age. 3.94in. x 7.78in. (10cmx20cm) cylinder specimens were cast in order to measure the surface electrical resistivity, according to FM 5-578 test method¹⁰ by an electrical resistivity meter right after steam curing, and at the ages of 28 and 90 days. The test was performed on 3 specimens. This non-destructive test indirectly indicates the permeability of concrete. The measurements were performed by four point Wenner array probe technique at four quaternary locations. This method of testing is especially suitable for in-site elements since the results can be obtained fast, easy, and without any damaging to the elements. The higher value of surface electrical resistivity (SR) shows that the concrete is more durable. Rapid chloride penetration test (RCPT), as for the ASTM C-1202 procedure,¹¹ was performed on 3.94in. x 7.78in. (10x20cm) specimens. The cylinders were sliced into 10x5 cm cylinders and exposed to a 60V potential for 6 hours. The total charge passed through the specimens was then recorded in order to evaluate the permeability and consequently corrosion potential of each mixture design. The test was carried out on 3 specimens

Table 3—Steam curing regimes

Mixture	(A)Duration of pre-curing (h)	(B) Pre-curing Temp(°C)	(C)Rate of heating (k/h)	(D) Duration of steam curing at max. Temp.	(E) Rate of cooling (k/h)	(F) Total duration of curing (h)
M-60-16	2	20	10	10	10	16
-18				12		18
-20				14		20
P-60-16	2			10		16
-18				12		18
-20				14		20
T-60-16	2			10		16
-18				12		18
-20				14		20
L-60-16	2			10		16
-18				12		18
-20				14		20
M-70-16	2			10		16
-18				12		18
-20				14		20
P-70-16	2			10		16
-18				12		18
-20				14		20
T-70-16	2			10		16
-18				12		18
-20				14		20
L-70-16	2			10		16
-18				12		18
-20				14		20

Items of investigation

Coarse and fine aggregates with the cement and supplementary cementitious materials were batched and mixed for 30 seconds, and then the entire water was added. After another 30s of mixing, superplasticizer was added until the suitable SCC characteristics were achieved. Several tests on fresh concrete such as, slump flow diameter, slump flow time (T50), and slump flow loss were performed according to BS EN 12350-Part2.⁸ Also, wet density was measured as well as L-Box test. Specimens then were cast in lubricated mold without any vibration in order to measure compressive strength, surface electrical resistivity, and rapid chloride penetration test. Molds were then exposed to 12 different steam curing regimes. As shown in **Table 3** and in **Fig.1** schematically, M-60-16 denotes the steam curing regimen for specimen containing Metakaolin exposed to the steam temperature of 60°C for 16 hours.

ANALYTICAL INVESTIGATION AND EXPERIMENTAL RESULTS
Fresh concrete properties

The properties of fresh concrete are presented in **Table 4**. The slump flow diameter for all mixtures were about 25-28 in. (65-70 cm); slump flow time was observed to be less than 15,

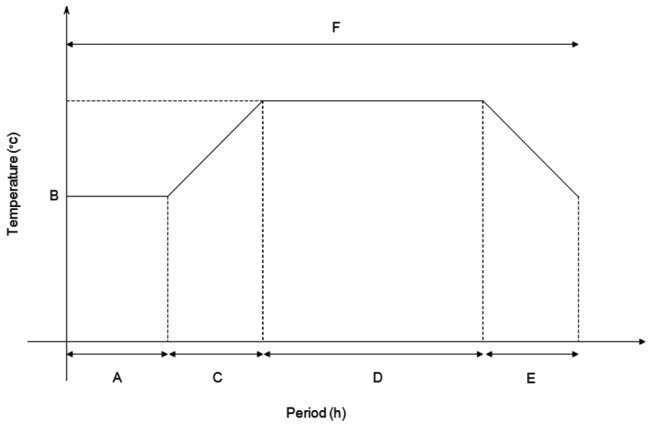


Fig.1. Schematic figure of steam curing regimes applied to specimens

Table 4—Properties of fresh SCC

Slump flow diameter in. (cm)		Slump flow loss (%)	L-Box test		Wet density, lb/ft ³ (kg/m ³)
Slump flow (0h)	Slump flow (after 1h)		Ratio (%)	Time (s)	
25.5-27.5 (65-70)	22.8-23.6 (58-60)	10-15%	0.85	42	147.95 (2370)

and the diameter of slump flow after 1h, slump flow loss, were measured to be about 24-26 in. (61-66 cm). The wet density of the mixtures was about 147.95 lb/ft³ (2370 kg/m³).

Compressive strength

The results of compressive strength are presented in Fig.2 and Fig.3 respectively. Test results indicate that the initial compressive strength is directly related to the maximum temperature of steam curing. The increase in compressive strength is due to extended hydration which is promoted by increasing the temperature. Moreover, during initial steam curing, because of the presence of excessive moisture and temperature, Calcium-Silica-Hydrate, C-S-H gel, which is the most important binding phase in hardened concrete,¹² is formed faster than normal curing condition in water or air. In fact, it can be related to the smooth rate of pre-curing, and lower steam curing temperature because with lower temperature and rate of heating and cooling, C-S-H gel and CH crystalline can gradually fill the inner pores, whereas by higher temperature and higher rate of heating and cooling, less pores may be filled completely. Moreover, by increasing the duration of steam curing, the proportion of C-S-H gel will increase as well, which leads to the formation of finer pores.

As mentioned previously, the consultant has defined a minimum initial compressive strength of 24 MPa that segments should obtain before demolding, it can be concluded that the total 16 hours of steam curing is not adequate for specimens to meet the criteria. Therefore, 16 h of steam curing duration at any temperature is not acceptable, for the materials, curing conditions, and testing procedures as stated.