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Testing program on the use of coarse aggregate in concrete with respect to a withdrawal of notes 2 and 3 of Table 12 of the CSA standard A23.1

Final report

Date: April 21th 2017
Ref. No.: 072-P-0009583-0-05-500-IM-R-0001-01

April 21th, 2017

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Object: *Testing program on the use of coarse aggregate in concrete with respect to a withdrawal of notes 2 and 3 of Table 12 of the CSA standard A23.1*
Final report
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We are pleased to submit a copy of our final report regarding the above-referenced project.

We hope the informations include in this report will be useful. If you have any questions, please contact us.

Yours truly,

Englobe

François Santerre, Eng.
Vice-President - Eastern Quebec

Englobe

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Revision No.	**Date**	**Modification and/or Publication Details**
0A	2016-12-09	Preliminary report – Laboratory results
0B	2016-12-23	Preliminary report – Laboratory results – Revision 01
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1 INTRODUCTION

1.1 CONTEXT

The “Regroupement professional des producteurs de granulats (RPPG)”, concerned to provide a high quality product, decided under the aegis of the “Association des constructeurs de routes et grands travaux du Québec (ACRGTQ)”, to verify the accuracy of notes 2 and 3 of Table 12 of the CSA standard A23.1, which specified an extension of test limits respectively from 6% to 9% and 17% to 19% for unconfined freeze-thaw and micro-Deval test method, especially for limestones and dolomites of St. Lawrence Lowlands.

To successfully develop and implement a testing program to verify the accuracy of notes 2 and 3, the RPPG needed to get an overall picture of the actual situation. In this case, the RPPG proceeded to the following activities:

- an inventory of quarries in St. Lawrence Lowlands which produce coarse aggregates for use in concrete;
- a verification, based on producers data (unconfined freeze-thaw test results, micro-Deval, petrographic number, etc.), of the number of quarries in the St. Lawrence Lowlands which need notes 2 and 3 to qualify their coarse aggregates for use in concrete;
- a visual inspection of the condition of various structures (curbs and sidewalks) exposed to de-icing salts aged 5 years and older that have been made with concrete containing coarse aggregates from different sources that whether or not require notes 2 and 3;
- a quick literature review on the subject.

1.1.1 Inventory of quarries

The territory covered by the St. Lawrence Lowlands is illustrated on Figure 1.

From the 2009 listing of sources of aggregates provide by the RPPG, 34 companies were identified to produce aggregates from sources located in St. Lawrence Lowlands. 19 of these 34 companies operate sources without producing aggregates for use in concrete and 1 company consider being located outside the St. Lawrence Lowlands. The 15 remaining operate a total of 28 sources of aggregates located in St. Lawrence Lowlands and are known to produce coarse aggregates for use in concrete.

In order to get additional informations, other companies which potentially operate sources of aggregates located in St. Lawrence Lowlands were identified using section 213 “granular materials” of the 2015-2016 ACRGTQ membership directory. A total of 74 companies excluding the 34 identified earlier using the RPPG 2009 listing, were identified. From these 74 companies, 20 indicated that they operate sites located outside the St. Lawrence Lowlands. One company from the 54 remaining has confirmed that they operate one source producing coarse aggregates for use in concrete. The 53 remaining companies did not response to the survey.

From these informations, we’ve got a total of 29 operating sites that are known to produce coarse aggregates from the St. Lawrence Lowlands for use in concrete.

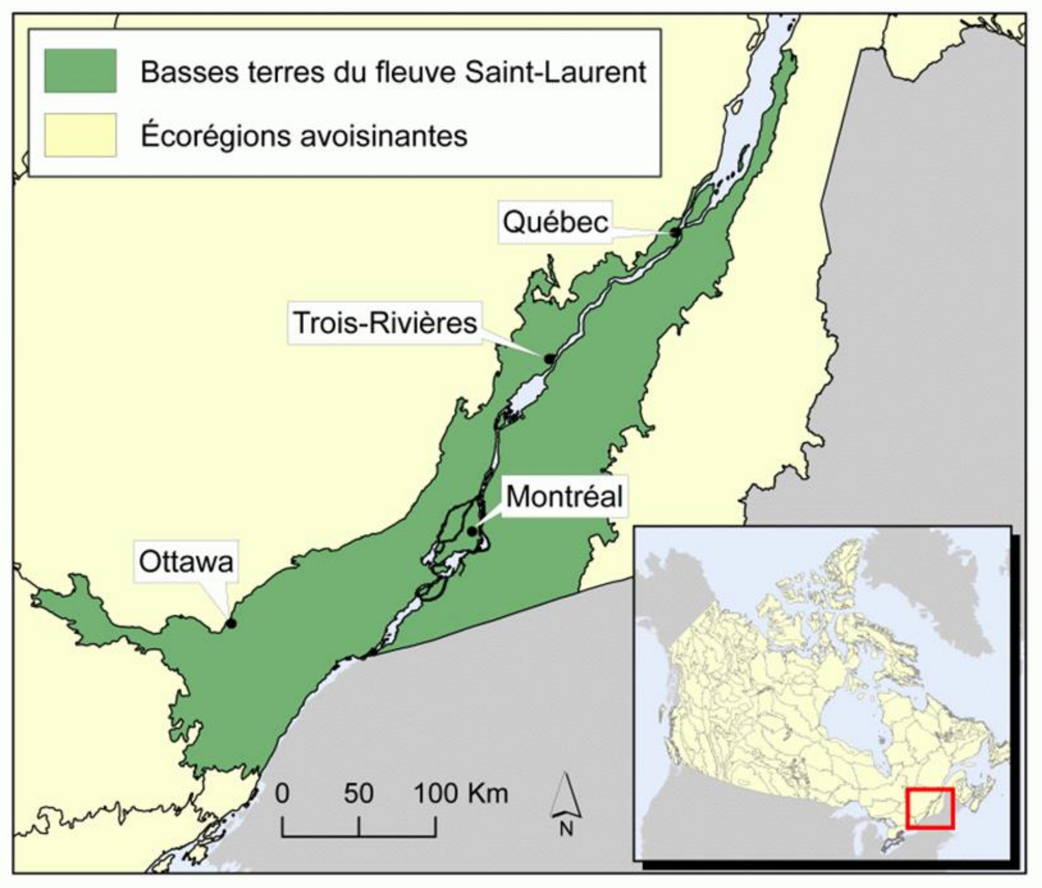

Figure 1 : Territory covered by the St. Lawrence Lowlands

1.1.2 Quarries requiring notes 2 and 3

The properties of coarse aggregates such as the type of aggregates and petrographic number, the potential AAR and/or $MgSO_4$ reactivity and the resistance to unconfined freeze-thaw and micro-Deval test method, are presented in Table 1. These properties were provided in 2015 by the 29 coarse aggregates producers identified previously.

Table 1 : Properties provided by the 29 coarse aggregates producers

Source	Type of aggregates	AAR / $MgSO_4$ (%)	micro-Deval (%) CSA A23.2-29A				Unconfined freeze-thaw Per sieve fraction (%)		Petrographic number
			5-10 mm	10-14 mm	14-20 mm	Avr.	CSA A23.2-24A	Avr.	CSA A23.2-15A
1	N/A	Non-reactive/ N/A	2.5-10 7.8	5-14 9.2	10-20 10.0	9	1.0 – 1.4 – 1.8	1.4	100
2	N/A	Non-reactive/ 1 to 1.7	5-10 8.3	5-14 8.9	10-20 7.6	8.3	0.8 – 1.1 – 2.3	1.4	
3	Dolomite	Non-reactive/ 2,7	LC-2170 (%)				1.8	1.8	102
			6.2	8.1	7.0				
4	Syenite	Non-reactive/ N/A			5-20 4.2	4.2	0.4	0.4	100
5	Dolomite	Non-reactive/ 1.7 to 6.7	7.8	7.7	7.9	7.8	7.0 – 6.4 – 3.9	5.8	103
6	Limestone	highly reactive/ 5.23			5-20 13.8	13.8	2.6	2.6	100
7	Limestone	Non-reactive/ N/A	9.6	9.1	9.3	9.3	2.0	2.0	111
8	Limestone	Non-reactive/ 1.0	5-14 11.0	5-20 12.0		11.5	1.5	1.5	N/D
9	Limestone	Non-reactive/ 1.5 to 9.0	5-14 9.0	5-20 15.0	10-20 13.0	12.3	3.7	3.7	N/D

Source	Type of aggregates	AAR / MgSO$_4$ (%)	micro-Deval (%) CSA A23.2-29A				Unconfined freeze-thaw Per sieve fraction (%)		Petrographic number
			5-10 mm	10-14 mm	14-20 mm	Avr.	CSA A23.2-24A	Avr.	CSA A23.2-15A
10	Dolomite	Non-reactive/ N/A	5-14 11.0	5-20 15.0		13.0	2.5	2.5	N/D
11	Limestone	Non-reactive/ 10.4 to 2.4	9.3	9.9	10-20 9.6	9.6	2.8 – 1.9 – 1.1	1.9	103
12	Dolomite	Non-reactive/ N/D	2.5-10 7.5	10-20 6.8		7.2	1.7	1.7	100
13	Dolomite	Non-reactive/ N/D	2.5-10 8.3			8.3	1.5	1.5	100
14	Limestone	Non-reactive/ 4.4	9.0	14.0	15.0	12.7	3.4	3.4	111
15	Dolomite	Non-reactive/ 0.3	6.1	6.0	5.7	5.9	1.8	1.8	108
16	Dolomite	Non-reactive/ N/D	3.0	6.0		4.5			110
17	Limestone	Non-reactive/ 2.5 to 4.9	2.5-10 15.6	5-14 15.3	10-20 15.1	15.3	6.9 – 5.7 – 5.1	5.9	103
18	Dolomite	Non-reactive/ 0.2 to 0.6		5-14 10.1	10-20 10.2	10.2	- 1.3 – 0.8	1.1	
19	Syenite	Non-reactive/ 0.0 to 0.2	2,5-10 3.2	5-14 1.3	10-20 1.9	2.1	0.7 – 0.8 – 0.2	0.6	100
20	Syenite	Non-reactive/ 0.8		5-14 10.4		10.4	5.7	5.7	111

Source	Type of aggregates	AAR / $MgSO_4$ (%)	micro-Deval (%) CSA A23.2-29A				Unconfined freeze-thaw Per sieve fraction (%)		Petrographic number
			5-10 mm	10-14 mm	14-20 mm	Avr.	CSA A23.2-24A	Avr.	CSA A23.2-15A
21	Limestone	N/D / N/D		5-14 18.6	5-20 19.5	19.1	0.8	0.8	100
22	Limestone	N/D / N/D		5-14 18.1	10-20 18.1	18.1	1.2	1.2	102
23	N/D	N/D / N/D	19.0			19.0	8.0	8.0	N/D
24	N/D	N/D / N/D	17.0			17.0	9.0	9.0	N/D
25	Limestone	N/D / N/D	19.0	13.4	16.4	16.3	9.1 – 6.7 – 4.9	6.9	123
26	Basalt	Non-reactive/ 3.8 to 4.9	7.5			7.5	5.4 to 9.9	7.7	100
27	Basalt	Non-reactive/ 10.7	7.8	9.6	9.0	8.8	5.1 – 6.5 – 5.1	5.6	100
28	Limestone	highly reactive/ 3.8			16.6	16.6	5.4	5.4	100
29	Limestone	Extremely reactive/ 2.4	13.5		5-20 14.5	14.0	3.2	3.2	118

Note 1: Green values represent aggregates which meet the requirements of Table 12 without notes 2 and 3. Yellow values represent aggregates that require the extension of limits permitted by notes 2 and 3 to meet the requirements of Table 12. Red values represent a test result that doesn't meet the requirements of Table 12 despite notes 2 and 3.

The unconfined freeze-thaw and micro-Deval test results show in Table 1 were plotted on a graph as shown in Figure 2 in order to get an overall picture of the situation of these aggregates producers regarding the acceptance criteria of Table 12 with or without notes 2 and 3. The situation can be interpreted by the following case scenarios:

- Green area: Aggregates meet the requirements of Table 12 without notes 2 and 3.
- Yellow area: Aggregates meet the requirements of Table 12 because of the extended limits permitted by notes 2 and/or 3.
- Red area: Aggregates do not meet the requirements of Table 12 despite notes 2 and 3.

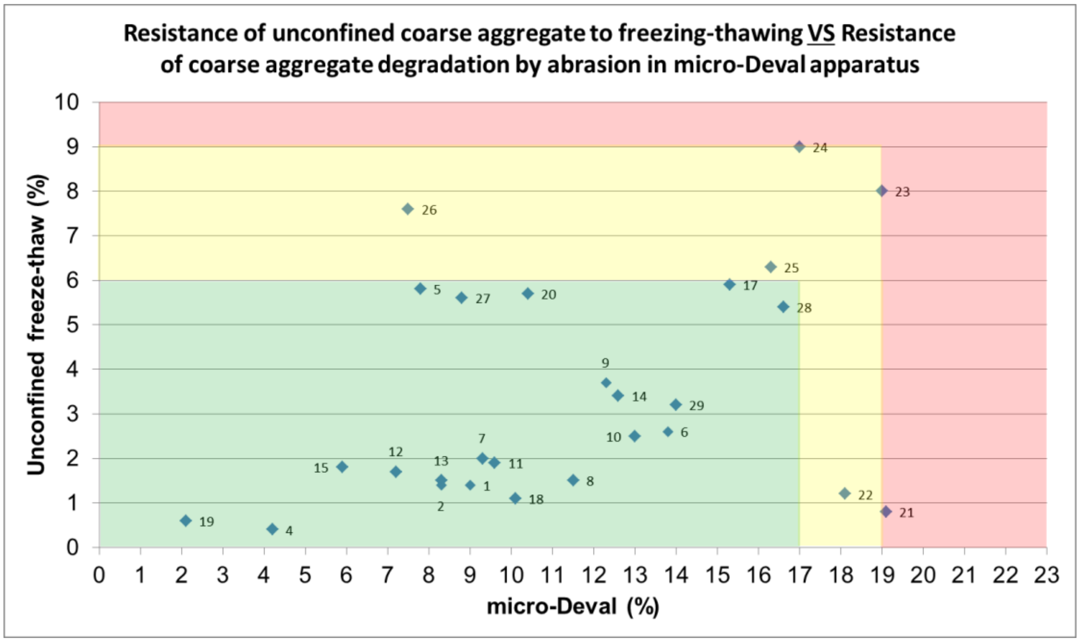

Figure 2: Overall picture of the situation for 29 aggregates producers with respect to notes 2 and 3 of Table 12

It should be noted that test results for sources #3 and #16 are not presented in Figure 2 because the producers were unable to provide unconfined freeze-thaw test results and also test results on micro-Deval in accordance with the CSA standard A23.2-29A. Moreover, source #4 is not actually producing coarse aggregates for use in concrete, but is planning to do it in the more or less long term. Also, sources #6, #28 and #29 are producing aggregates that show a high potential of alkali-aggregates reaction, which potential reactivity varies from high to extremely high. Consequently, these 6 sources mentioned previously (#3, #4, #6, #16, #28 and #29) will not be part of the present testing program.

Based on these informations, we observed that 6 of 29 sources identified (#21, #22, #23, #24, #25 and #26) require notes 2 and/or 3 of Table 12 of the CSA standard A23.1 in order to qualify their coarse aggregates for use in concrete. This corresponds to 21 % of sources identified.

Also, there are 5 of the 23 remaining sources showing test results close to the limits for unconfined freeze-thaw and micro-Deval test method without notes 2 and 3. This corresponds to 17 % of sources identified.

Given this, it means that approximately 38 % of sources known to produce coarse aggregates in the St. Lawrence Lowlands (11 of 29 sources) require notes 2 and/or 3 of Table 12 of the CSA standard A23.1 to qualify their aggregates for use in concrete.

1.1.3 Visual assessment of different concrete structures

Now that notes 2 and 3 have been identified to be an important factor for the aggregates manufacturing industry, a visual assessment of different concrete structures made from St. Lawrence Lowlands coarse aggregates (limestones and dolomites) was necessary to observe if the durability of concrete structures made from aggregates using notes 2 and/or 3 may be affected.

Before proceeding to the selection of the structures that would be evaluated, it should be noted that a number of factors may influence the concrete durability of the structures. Here is a non-exhaustive list of these factors:

- Year of construction;
- Period of the year (season);
- Mix design (compressive strength, type of cement, spacing factor, supplementary cementing materials, etc.);
- Properties of aggregates;
- Water potentially added on-site;
- Concrete placement and period of curing;
- Nature and application rate of de-icing salts;
- Number of repeated freeze-thaw cycles;
- Etc.

In order to facilitate the visual assessment, concrete structures consisting mainly of **curbs** and **sidewalks** were selected. These kinds of structures present flat surfaces significantly exposed to de-icing salts.

However, exterior walls (foundations) have also been assessed to verify the durability performance of exposure class F-2 concrete (concrete in an unsaturated condition exposed to freezing and thawing, but not to chloride). Actually, there is no mention of this exposure class in notes 2 and 3. We consider that this class of exposure should be included in notes 2 and 3 because an exposure class F-2 concrete is usually less exposed than an exposure class F-1 concrete.

The concrete structures were determined on the informations (location, type and age of structure) that concrete suppliers were able to provide, but also from sources of aggregates requiring notes 2 and/or 3 to be accepted for use in concrete. Moreover, the concrete structures had to be at least 5 years of age and significantly exposed to de-icing salts. This process was done to ensure that the concrete structures were exposed for a certain amount of time in order to verify if there is a relationship between indication of concrete damages and the easing of notes 2 and 3. It should be noted that concrete structures made from aggregates that do not need notes 2 and 3, have a good performance in durability.

Table 2 shows the properties of aggregates from sources identified (see Table 1) and the concrete structures associated with these sources, for the visual assessment. Photos taken during the assessment of the concrete structures are detailed in Appendix 1.

The visual assessment performed by Yves Dénommé, Eng. M.Sc.A from the “Association béton Québec (ABQ)” and Charles Abesque, Eng. Ph.D. from the “Association des constructeurs de routes et grands travaux du Québec (ACRGTQ)” demonstrated that the concrete structures made from aggregates which do not meet the requirements of Table 12 of the CSA standard A23.1 without notes 2 and/or 3 (sources #21, #24 and #25), show a good performance in durability. These structures barely show scaling of surface mortar and popouts. The observation was the same for concrete structures made from aggregates that are close to the requirements without using notes 2 and/or 3 (sources #5, #17 and #28).

Thus, based on these informations, a possible withdrawal of notes 2 and 3 will have a major impact on the aggregates manufacturing industry, especially for more than 38 % of sources which produce coarse aggregates for use in concrete, despite a good performance in durability of concrete structures. Reinforced in its approach, the industry decided to perform a laboratory testing program to verify the accuracy of notes 2 and 3 of Table 12 of the CSA standard A23.1. This testing program was developed by the ACRGTQ in collaboration with many specialists in concrete materials. This testing program was then performed by Englobe Corp.

Table 2 : Properties provide by the producers of the 6 sources selected and concrete structures specifications

Source	Nature of aggregates	AAR / $MgSO_4$ (%)	micro-Deval (%)	Unconfined freeze-thaw (%)	Petrographic number	Concrete structures	
			CSA A23.2-29A	CSA A23.2-24A	CSA A23.2-15A	Type	f'c (MPa)
5	Dolomite	Non-reactive/ 1.7 to 6.7	7.8	5.8	103	Sidewalk #1	35
						Sidewalk #2	35
17	Limestone	Non-reactive/ 2.5 to 4.9	15.3	5.9	103	Sidewalk #1	~35
						Sidewalk #2	~35
						Bicycle path #3	~35
21	Limestone	N/D / N/D	19.1	0.8	100	Sidewalk #1	35
						Sidewalk #2	35
						Curbs #3	35
24	N/D	N/D / N/D	17.0	9.0	N/D	Sidewalk #1	35
						Sidewalk #2	35
						Sidewalk #3	35
25	Limestone	N/D / N/D	16.3	6.9	123	Sidewalk #1	35
						Sidewalk #2	35
						Sidewalk #3	35
						Sidewalk #4	32
28	Limestone	Highly reactive/ 3.8	16.6	5.4	100	Foundation walls #1	25 to 30
						Foundation walls #2	

Note 1: Green values represent aggregates which meet the requirements of Table 12 without notes 2 and 3. Yellow values represent aggregates that require the extension of limits permitted by notes 2 and 3 to meet the requirements of Table 12. Red values represent a test result that doesn't meet the requirements of Table 12 despite notes 2 and 3.

1.1.4 Literature review

The evolution over time of the limits authorised by CSA standard A23.1 on unconfined freeze-thaw and on micro-Deval is presented in Table 3.

Table 3 : Evolution over time of CSA standard A23.1 limits on micro-Deval and unconfined freeze-thaw

CSA version	micro-Deval (%)		Unconfined freeze-thaw (%)	
	†Subjected to freeze-thaw	Other classes of exposure	†Subjected to freeze-thaw	Other classes of exposure
1994	No requirements		‡6	‡10
2000	‡14	‡17	‡6	‡10
2004	17	17	6	10
2009	17	21	6	10
2014	17	21	6	10

†: 2000, F-1, C-1 et C-2

‡: Other requirements

2004: Introduction of Note 2 on unconfined freeze-thaw (9 instead of 6 if subjected to freeze-thaw and 13 instead of 10 if other exposure)

2009: Introduction of Note 3 on micro-Deval (19 instead of 17 if subjected to freeze-thaw)

The objective of the 2006 study funded by « Aggregates Foundation for Technology, Research and Education, ICAR 507-1F» was to verify if micro-Deval testing, alone or coupled with another test, would determine if a correlation exist between the aggregate performances for on-site concrete versus laboratory testing.

This study allowed putting into perspective the relations between micro-Deval and unconfined freeze-thaw results of coarse aggregate versus in-service concrete structures (Figure 3).

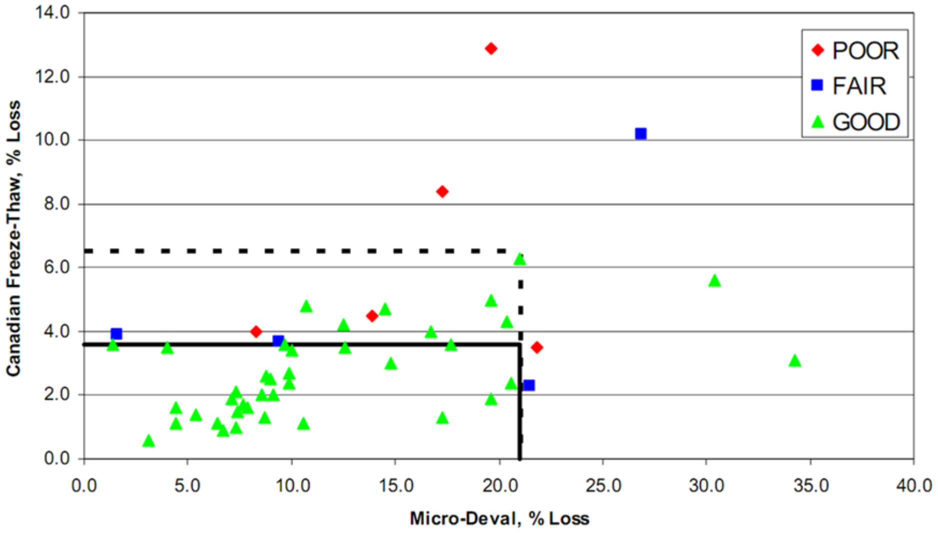

Figure 3 : Performance according to ICAR 507-1F 2006 of in-service concrete structures in terms of micro-Deval and unconfined freeze-thaw results of coarse aggregate used in concrete manufacturing

The 1991 study of Rogers « Laboratory Tests for Predicting Coarse Aggregate Performance in Ontario » indicates the relation between performances of in-service aggregates used in concrete manufacturing versus laboratory testing (Figure 4).

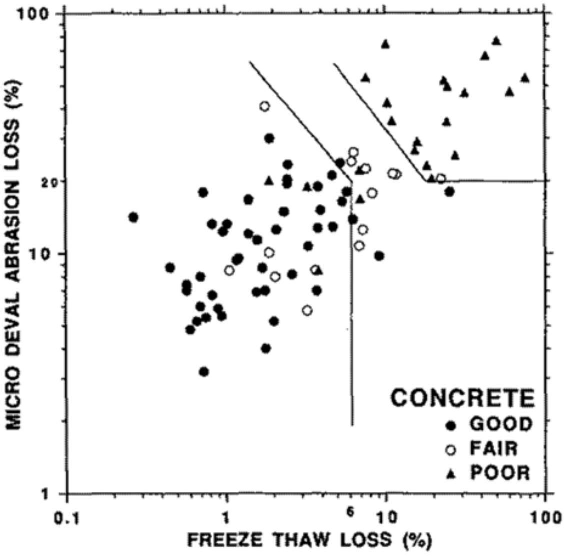

Figure 4 : Performance according to Rogers 1991 of in-service concrete structures in terms of micro-Deval and unconfined freeze-thaw results of coarse aggregate used in concrete manufacturing

These graphs indicate that in-service performance of concrete structures conceived with coarse aggregate with micro-Deval results up to 21 and unconfined freeze-thaw results up to 6.2 have a good performance. However, these graphs indicate that structure with a good performance can be manufactured with higher results of micro-Deval and unconfined freeze-thaw results than the results presented above.

1.2 MANDATE

Englobe Corp. was contracted by the “**Association des constructeurs de routes et grands travaux du Québec”** to perform a testing program on the use of coarse aggregate in concrete with respect to a withdrawal of notes 2 and 3 of Table 12 of the CSA standard A23.1.

In the forthcoming revision of CSA standard A23.1, CSA technical committee members raised the possibility of withdrawing notes 1 to 5 of Table 12. Consequently, the “Regroupement professionnel des producteurs de granulats (RPPG)”, represented by the ACRGTQ, requested that a testing program was performed to verify the relevance of this withdrawal. The consultation of many specialists in concrete materials (Canadian standard association (CSA) technical committee members, laboratories, “Ministère des transports, de la mobilité durable et de l’électrification des transports (MTMDET)”, university lecturers, municipal technical representatives, etc.), results in the development of a testing program performed by Englobe Corp.

This testing program consisted in the evaluation of physical properties of coarse aggregates selected for the purpose of this program. Performance in durability of concrete made from these coarse aggregates was also assessed through durability testing method in laboratory. From these test results, the quality of coarse aggregates that are normally used in exposure class F-1, C-XL, C-1 and C-2 concretes because of notes 2 and 3 of Table 12, will be clarified.

1.3 METHODOLOGY

As mentioned previously, this testing program was developed upon the consultation and recommendation of different specialists in concrete materials (Canadian standard association (CSA) technical committee members, laboratories, “Ministère des transports, de la mobilité durable et de l’électrification des transports (MTMDET)”, university lecturers, municipal technical representatives, etc.).

The testing program consisted in the determination of physical properties of coarse aggregates selected for the purpose of this program, but also in the evaluation of the durability performance of concrete made from these coarse aggregates. The following tests were carried out on the aggregates:

- Test method for the resistance of coarse aggregate to degradation by abrasion in the Micro-Deval apparatus - CSA A23.2-29A;
- Test method for the resistance of unconfined coarse aggregate to freezing and thawing - CSA A23.2-24A;
- Relative density and absorption of coarse aggregate - CSA A23.2-12A;
- Petrographic examination of aggregates - CSA A23.2-15A (Method A).

The following tests were carried out on concrete made from the different sources of aggregates:

- Compressive strength of cylindrical concrete specimens - CSA A23.2-9C;
- Standard test method for microscopical determination of parameters of the air-void system in hardened concrete - ASTM C457;
- Standard test method for resistance to scaling - BNQ 2621-905/2012;
- Standard test method for resistance of concrete to rapid freezing and thawing - ASTM C666 (Procedure A – freezing and thawing in water).

Figure 5 on next page shows a flow chart which specifies the successive steps used for this testing program, such as the procedure for aggregate selection, the determination of physical properties of aggregates and the durability performance of concrete.

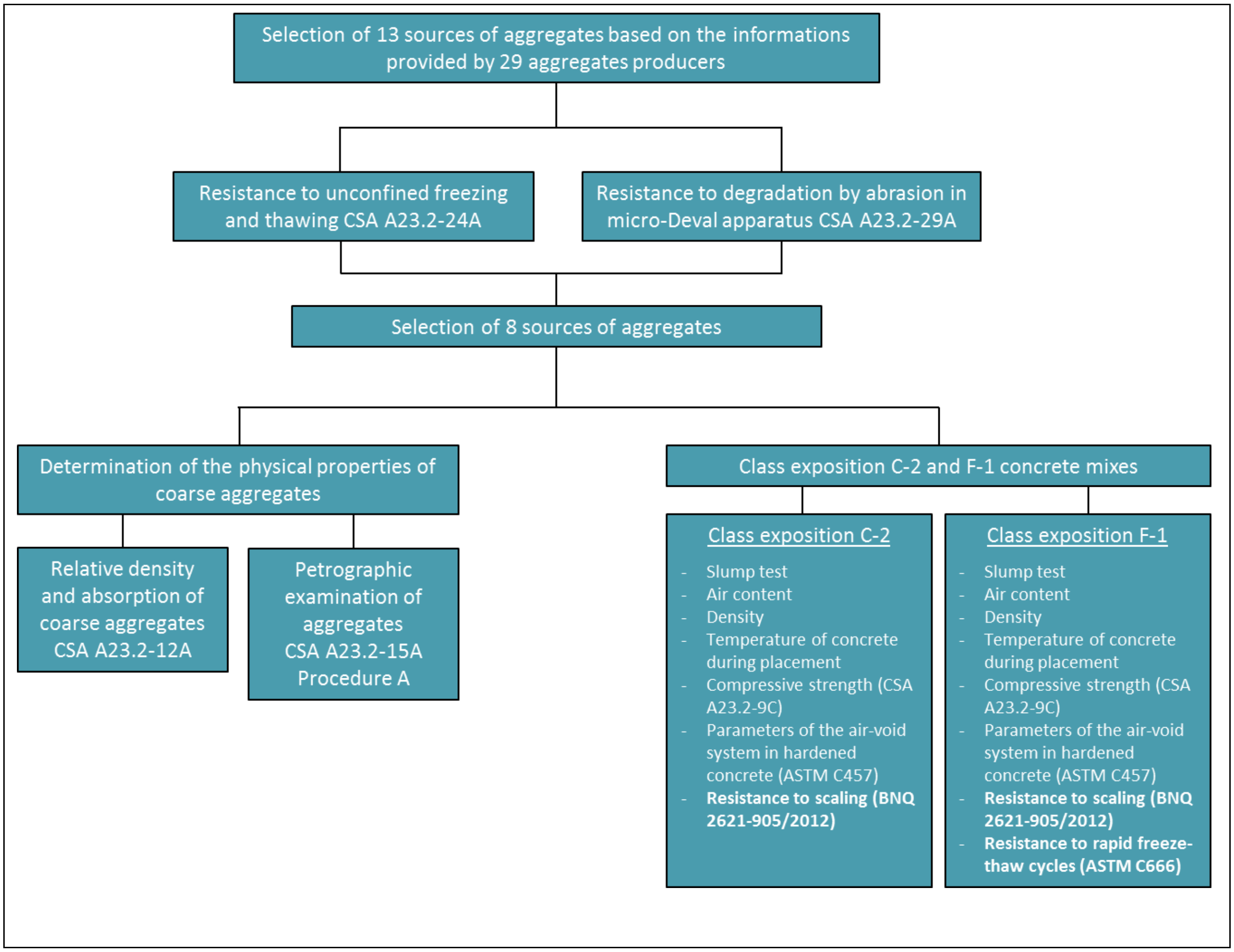

Figure 5 : Laboratory testing program

2 LABORATORY TESTS

2.1 PROCEDURE FOR AGGREGATE SELECTION

To highlight the accuracy of notes 2 and 3 of Table 12, coarse aggregates have been selected from unconfined freeze-thaw and micro-Deval test results provided by 29 aggregates producers. These producers are mostly located in St. Lawrence Lowlands and are known to produce limestones and dolomites for use in concrete. Test results provided by these 29 aggregates producers are shown in Table 1 and Figure 2 in section 1.1.2.

From the data provided by the producer and to respect both completion deadlines and budget constraints, it has been decided that this research program would be done from 8 of the 23 remaining sources. In fact, the following sources: #3, #4, #6, #16, #28 and #29, previously identified, could not have been selected for reasons mentioned earlier.

Unconfined freeze-thaw and micro-Deval tests will be redone by Englobe Corp. on every remaining source. This step is necessary to verify if the characteristics have changed from the last available results. Moreover, micro-Deval testing often leads to variable results. The test has been performed twice and realised in both Englobe's laboratories of Québec and Lévis. The goal was to verify the results variability between laboratories and to be sure not to retain a source that would have showed significant differences.

This is why we have selected more sources than necessary, such as 13, to be sure to put aside every source that would show too important variability with micro-Deval results or for any other reason. Therefore, it was important that each area shown in Figure 2 is represented in the selection of the 13 sources.

The results of the 13 aggregate sources selected are shown in Table 4. These results guided the choice of the 8 aggregates sources that have been retained for the pursuit of the testing program.

Table 4: Abrasion resistance in micro-Deval apparatus and unconfined freezing-thawing resistance test results

Source	Sieve fraction	Unconfined freezing and thawing (%)	micro-Deval (Québec) (%)	micro-Deval (Lévis) (%)	micro-Deval Average (%)
5	5-14 mm	5.4	8.5	9.3	8.9
	10-20 mm	4.2	8.7	9.0	8.9
9	5-20 mm	5.2	18.9	19.0	19.0
11	5-14 mm	5.6	9.1	8.6	8.9
	10-20 mm	4.0	7.8	8.2	8.0
14	5-14 mm	7.0	17.6	15.6	16.6
	10-20 mm	2.8	12.3	13.7	13.0
17	5-14 mm	6.8	15.0	17.0	16.0
	10-20 mm	4.7	15.4	16.1	15.8
19	5-14 mm	0.8	2.6	2.6	2.6
21	5-14 mm	0.4	17.9	20.0	19.0
	10-20 mm	0.6	18.3	30.6	24.5
22	5-14 mm	0.7	17.4	19.7	18.6
	10-20 mm	0.4	18.1	18.9	18.5
23	5-14 mm	5.2	20.8	22.4	21.6
	10-20 mm	2.0	19.5	22.4	21.0
24	5-20 mm	5.5	14.3	14.4	14.4
25	5-20 mm	3.4	19.0	19.9	19.5
26	5-14 mm	4.2	7.7	8.0	7.9
	10-20 mm	5.4	8.8	8.6	8.7
27	5-20 mm	4.4	9.0	8.5	8.8

Note 1: Green values represent aggregates which meet the requirements of Table 12 without notes 2 and 3. Yellow values represent aggregates that require the extension of limits permitted by notes 2 and 3 to meet the requirements of Table 12. Red values represent a test result that doesn't meet the requirements of Table 12 despite notes 2 and 3.

Note 2: Brechin Drain Brothers' aggregates from Stoney Lake quarry was used as control. The weighted loss must be within 11.4 % to 14.8 % for micro-Deval test method and 8.5 % to 15.3 % for unconfined freeze-thaw resistance. Micro-Deval test results show a weighted loss of 13.5 % (sources 9-14-17-21-22-23), 13.8 % (sources 5-11-24-25), 15.0 % (sources 26-27) and 14.1 % (source 19). Unconfined freeze-thaw test results show a weighted loss of 11.5 % (sources 5-11-25), 11.3 % (sources 9-22-23-24), 14.1 % (sources 26-27) and 13.6 % (sources 14-17-21).

In order to verify the accuracy of notes 2 and 3 of Table 12 in CSA standard A23.1, it was important to select and compare sources producing aggregates with test results falling into the red, green or yellow area as illustrated in Figure 2. Test results from Table 2 were then plotted on a similar graph as shown in Figure 2. Figure 6 presents test results for sieve fractions 5-14 mm and 10-20 mm, while Figure 7 shows test results for sieve fraction 5-20 mm.

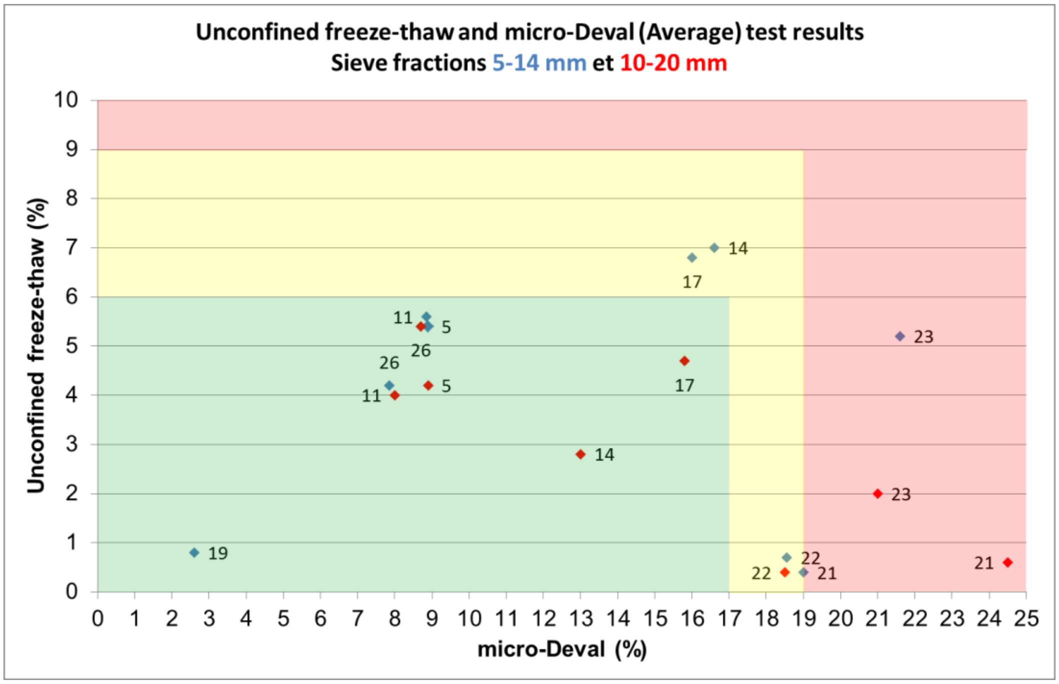

Figure 6: Resistance of unconfined coarse aggregate to freezing and thawing and resistance of coarse aggregate degradation by abrasion in the micro-Deval apparatus test results for sieve fractions 5-14 mm and 10-20 mm. Micro-Deval test results shown represent the average between results from Québec facilities and Lévis facilities.

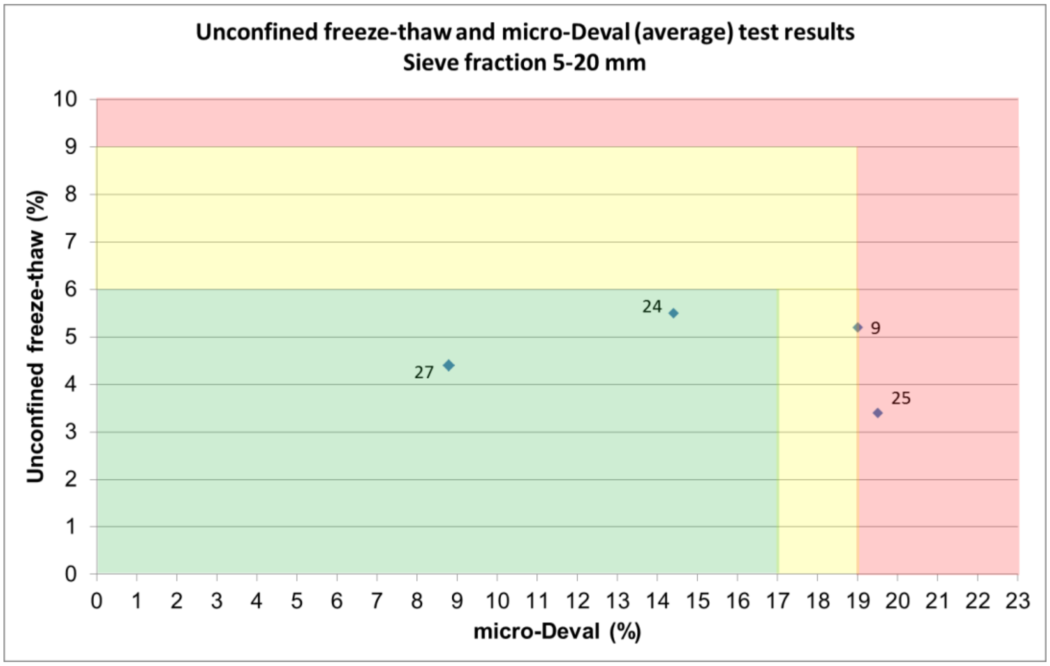

Figure 7: Resistance of unconfined coarse aggregate to freezing and thawing and resistance of coarse aggregate degradation by abrasion in the micro-Deval apparatus test results for sieve fraction 5-20 mm. Micro-Deval test results shown represent the average between results from Québec facilities and Lévis facilities.

These figures show, unlike the 2015 test results, no results within 6 % and 9 %, corresponding to the extended limit permitted by note 2 for resistance of unconfined coarse aggregate to freezing-thawing test method. This applies in particular for aggregates of sieve fractions 10-20 mm and 5-20 mm. Regarding the micro-Deval test results, there are only two sources of aggregates within 17 % and 19 % of weighted loss. These aggregates come from source #21 (sieve fraction 5-14 mm) and source #22 (sieve fractions 5-14 mm and 10-20 mm). Moreover, source #19 shows test results close to the upper limit (19 %) for sieve fraction 5-20 mm.

Given these results and according to Figure 6, aggregates of sieve fraction 5-14 mm are most likely to be affected by freezing and thawing conditions. This is the case for aggregates produced from sources #14 and #17. These sources usually require an extension of limit from 6 % to 9 % to qualify their aggregates for use in exposure class F-1, C-XL, C-1 and C-2 concretes.

These observations suggest that notes 2 and 3 are more critical for the aggregates qualification of sieve fraction 5-14 mm than the other sieve fractions. This can be explained by the preparation process of aggregates, e.g. during the crushing operations, smaller particles represent the brittle part of aggregates. Consequently, this brittle part will break more easily because of its lower resistance. However, it is important to mention that this observation does not apply to every types of aggregate, because of other key factors like hardness, mineralogical composition and presence of impurities or clay minerals which can influence the quality of aggregates.

In order to compare the performance of aggregates in concrete and to limit the number of variables that may have an impact on the study results, it was decided to select sources producing aggregates of sieve fraction 5-14 mm only. Sources #9, #24, #25 and #27 were thus rejected because these sources produce aggregates of sieve fraction 5-20 mm. Source #21 was also dismissed due to significant variation observed between micro-Deval test results on sieve fraction 10-20 mm from Québec and Lévis facilities. With these 5 sources dismissed, only 8 sources were remaining to be selected for the pursuit of the testing program, corresponding to sources #5, #11, #14, #17, #19, #22, #23 and #26.

As illustrated in Figure 2, each case scenario is represented by at least one of the 8 sources selected. The situation of each source is detailed as follow:

- Aggregates produce by sources #5, #11, #19 and #26 meet the requirements of Table 12 without notes 2 and 3. Aggregates from source #19 were used as control sample for this testing program. These aggregates show a good performance for unconfined resistance to freezing and thawing and micro-Deval test method.

- Aggregates produce by sources #14 and #17 are usually accepted for use in exposure class F-1, C-XL, C-1 and C-2 concretes due to the extended limit from 6 % to 9 % permitted by note 2 for resistance of unconfined freezing and thawing test method. These aggregates also meet the micro-Deval requirement without note 3.

- Aggregates produce by source #22 represent the case scenario where the aggregates meet the requirement for unconfined resistance to freezing and thawing test method without note 2, but are only accepted because of note 3 for micro-Deval test method.

- Aggregates produce by source #23 do not meet the requirement for micro-Deval test method according to Table 12, despite note 3.

2.2 DETERMINATION OF THE PROPERTIES OF THE AGGREGATES

Additional tests were carried out to determine the properties of the aggregates selected for the pursuit of the testing program. These tests included a petrographic analysis of aggregates and determination of relative density and absorption for each source of aggregates. Test results are presented in the following sections.

2.2.1 Petrographic examination of coarse aggregate

For every sources selected, a petrographic analysis was carried out according to method A of CSA standard A23.2-15A. Two (2) thin sections were also prepared and analyzed in order to complete the analysis. The petrographic analysis results are detailed in Appendix 2. Table 5 shows a resume of the main observations made. It is important to mention that inside a quarry, the geology may vary from one place to another. So, the petrographic analysis may vary as well depending on the sampling location.

Table 5: Petrographic examination of coarse aggregate

Source	Petrographic description	Petrographic number (PN)[1]	Mineralogy
5	Calcitic dolostone[2]	129	50% dolomite, 30% calcite, 5-10% opaque minerals (pyrite and iron oxide)
11	Calcilutite[2]	105	70% calcite, 30% dolomite, 2-5% opaque minerals (pyrite and iron oxide)
14	Fossil calcilutite[2]	106	Fine-grained carbonate matrix containing fossils fragments and trace of crystalline dolomite, 5-10% opaque minerals (pyrite only)
17	Calcitic dolostone[2]	106	85% dolomite, 10% carbonate clasts, 1-5% opaque minerals (pyrite and iron oxide)
19	Basalt	100	70-80% plagioclase, 10-15% pyroxene, 5% olivine, 1% opaque minerals (trace of pyrite and iron oxide)
22	Dolomitic limestone[2]	100	60% calcite, 35% dolomite, 1-3% opaque minerals (pyrite only)
23	Calcitic dolostone[2]	110	Crystalline dolomite in a fine-grained carbonate matrix with trace of fossils, 1-5% opaque minerals (pyrite only)
26	Basalt	100	75-85% plagioclase, 10-15% pyroxene, 1-2% olivine et chlorite, 2-10% opaque minerals (trace of pyrite and iron oxide)

[1] No petrographic quality factor of 6 (poor) or 10 (deleterious materials – argillaceous) was attributed during petrographic examination of each source of aggregates.

[2] Clayed and/or schisted veneer were observed on a certain amount of particles during petrographic examination of these aggregates.

Aggregates produce by sources #5, #17 and #23 are mainly composed of calcitic dolostone with a mineralogical composition slightly different from one source to another. Aggregates produce by sources #19 and #26 consist of basalt. Aggregates from sources #11, #14 and #22 are respectively a calcilutite, a fossil calcilutite and a dolomitic limestone.

The mineralogical composition of sedimentary rocks may vary depending on the conditions sediments have been deposited and then consolidated over time. Accordingly, the number of occurrences and the particularities of different facies observed on the polished thin sections, was establish and report in the petrographic examination report. Note that distinction between these different facies can only be achieved by an experienced professional using a microscope.

For each source, a petrographic number (PN) was attributed to the aggregates. For all aggregates selected for this testing program, a PN corresponding to a good physical and mechanical quality, from a petrographic point of view and for the production of concrete, was obtained. Referring to Table A2.2 of the CSA standard A23.2-15A (14), all of these aggregates can be used in the manufacture of exposure class F-1, C-1 and C-2 concretes, since their petrographic numbers are below the limit of 125 suggested for these concrete classes, with the exception of source #5.

During the petrographic examinations, a petrographic factor is attributed to each facies observed in the samples. A factor of 1 is normally used for a facies with a good petrographic quality while factors of 3, 6 and 10 are commonly used for facies with a fair, poor and deleterious petrographic quality, respectively. Intermediate factors can also be used if the aggregate has good petrographic qualities, but lower hardness, for example. The majority of the facies observed in the aggregates selected have a good petrographic quality. However, clayed veneers have been noticed on some aggregate particles. The number of aggregates presenting clayed veneers varies from one source to the other. By multiplying the factor with the weighted grain size distribution for each facies, a cumulative PN is obtained. The sum of these cumulative PNs gives the PN attributed to the coarse aggregate analyzed. In this case, the petrographic examinations show PNs between 100 and 129 depending on the source of aggregates. Based on the petrographic analysis, it is important to mention that aggregates have less than 2.0 % of deleterious (argillaceous) materials.

2.2.2 Relative density and absorption of coarse aggregate

Relative density and absorption of coarse aggregate were determined in accordance with CSA standard A23.2-12A. Tests were carried out on the aggregates of sieve fraction 5-14 mm selected. Test results are shown in Table 6.

Table 6: Relative density and absorption of coarse aggregate (sieve fraction 5-14 mm)

Source	Bulk relative density	Bulk relative density SSD*	Absorption (%)
5 – Calcitic dolostone	2.78	2.79	0.64
11 – Calcilutite	2.75	2.77	0.49
14 – Fossil calcilutite	2.71	2.72	0.47
17 – Calcitic dolostone	2.70	2.72	0.54
19 – Basalt	2.52	2.53	0.50
22 – Dolomitic limestone	2.70	2.71	0.37
23 – Calcitic dolostone	2.69	2.71	0.86
26 – Basalt	2.91	2.93	0.63

* Saturated surface-dry.

Test results show a bulk density ranging from 2.52 to 2.91 and absorption between 0.37% and 0.86% depending on the aggregates. Aggregates with high absorption are usually more likely to be affected by the action of freezing and thawing cycles. However, the nature and petrographic quality (mineralogical composition, degree of crystallinity, etc.) may also have an influence on the performance of aggregates under freezing and thawing conditions.

2.3 CONCRETE MIXING AND LABORATORY TESTING

A withdrawal of notes 1 to 5 could have a major impact for aggregates producers using their aggregates in exposure class F-1, C-XL, C-1 and C-2 concretes. It was therefore important to demonstrate the durability performance of concrete made from these aggregates. For this purpose, exposure class F-1 and C-2 concretes were manufactured and then assessed through durability performance tests in laboratory. Durability tests involved the evaluation of the resistance to scaling (BNQ 2621-905/2012) for exposure class F-1 and C-2 concrete mixtures and also the determination of the resistance to rapid freezing and thawing cycles (ASTM C666) for exposure class F-1 concrete mixtures.

A total of 12 concrete mixtures were manufactured, consisting more specifically of one exposure class C-2 concrete mixture for each source of aggregates, as well as 4 exposure class F-1 concrete mixtures. Regarding exposure class F-1 concrete mixtures, aggregates produced by sources #5, #14, #19 and #23 were selected. For selecting these four sources of aggregates, the following criteria were used:

- Source #5: Test results comply, but are close to the limit without note 2 for resistance of unconfined coarse aggregate to freeze-thaw. Micro-Deval test results meet the requirement of Table 12 without note 3.
- Source #14: These aggregates are accepted due to the extension of the limit permitted by note 2 for unconfined resistance to freeze-thaw. Aggregates comply with micro-Deval requirement without note 3.
- Source #19: These aggregates are used as control samples, because test results comply with the requirements of Table 12 without notes 2 and 3.
- Source #23: These aggregates are compliant, but close to the limit for unconfined resistance to freeze-thaw without note 2. However, these aggregates do not meet the requirement of micro-Deval test method, despite note 3.

The following sections present test results regarding the performance of concrete made from the aggregates selected. For exposure class C-2 concrete mixtures, four cylinder concrete specimens were made to evaluate the parameters of the air-void system in hardened concrete and to determine the compressive strength at 7 and 28 days. Two concrete slabs were also made to determine the resistance of concrete to scaling. This test was also carried out on exposure class F-1 concrete mixtures. For exposure class F-1 concrete mixtures, additional prism concrete specimens were made in order to evaluate the resistance of these concrete mixtures to rapid freezing and thawing cycles.

2.3.1 Properties of freshly mixed concrete

As mentioned previously, 12 concrete mixtures were made, including 8 exposure class C-2 concrete mixtures and 4 exposure class F-1 concrete mixtures. Some physical properties were verified on freshly mixed concrete, e.g. slump test value, air content, density and temperature of concrete during placement.

The concrete mix designs are detailed in Appendix 3. A water-to-cement ratio of 0.45 and 365 kg/m^3 of binder were used for exposure class C-2 concrete mix design. For exposure class F-1 concrete, a water-to-cement ratio of 0.50 and 335 kg/m^3 of binder were used. Table 5 on next page shows the constituents used for all concrete mixtures. The values are expressed for 1 m^3 of concrete. The properties of freshly mixed concrete are also presented in Table 7.

According to the specifications of the CSA standard A23.1-14, concrete of exposure classes C-2 and F-1 must have air content ranging from 5% to 8% and have a temperature between 10^{o}C and 35^{o}C during the placement of concrete. In addition, when a slump test value of 80 mm is prescribed, a tolerance of ± 30 mm is recommended.

Test results show that all of the concrete mixtures meet these acceptance criteria, with the exception of exposure class C-2 concrete mixtures made with the aggregates produce by sources #5 and #19. For these sources, a slump test value below 50 mm was obtained. These concrete mixtures were made for a second time in order to meet the CSA standard A23.1-14 requirements.

Furthermore, Table 7 shows that the temperature of concrete measured during the placement of concrete was relatively high. This can be explained by the fact that warm temperatures recorded in July may have affected the ambient temperature where the constituents were stored. The concrete specimens were stored for 24 hours under a wet burlap and a plastic sheet at ambient temperature (temperature ranging from 18.3^{o}C to 26.5^{o}C), before being removed from the moulds. According to the CSA standard A23.2-3C, the initial temperature of curing shall be between 15^{o}C and 25^{o}C. However, we judged that the difference between the ambient temperature in the laboratory and the temperature of curing prescribed in the CSA standard A23.2-3C was not considered to be significant, so the compressive strength development and the performance of concrete were not affected by this difference observed.

Table 7: Physical properties of freshly mixed concrete

Constituents	Exposure classification C-2										Exposure classification F-1			
	5	5 second trial	11	14	17	19	19 second trial	22	23	26	5	14	19	23
Coarse aggregate (5-14 mm)	988 kg	992 kg	988 kg	976 kg	976 kg	944 kg	936 kg	972 kg	976 kg	1 012 kg	960 kg	949 kg	917 kg	940 kg
Sand	812 kg	816 kg	808 kg	800 kg	800 kg	768 kg	772 kg	796 kg	796 kg	832 kg	886 kg	874 kg	843 kg	871 kg
Cement	365 kg	365 kg	365 kg	365 kg	365 kg	365 kg	365 kg	365 kg	365 kg	365 kg	335 kg	335 kg	335 kg	335 kg
Water	164 kg	156 kg	160 kg	164 kg	164 kg	156 kg	156 kg	168 kg	168 kg	164 kg	166 kg	166 kg	160 kg	169 kg
Air entrainers (Master Air AE 210)	208 ml	112 ml	168 ml	160 ml	160 ml	160 ml	112 ml	160 ml	160 ml	160 ml	120 ml	120 ml	120 ml	120 ml
Water-reducers (Pozzolith 210 BASF)	912 ml	920 ml	912 ml	912 ml	912 ml	912 ml	912 ml	912 ml	912 ml	912 ml	837 ml	837 ml	837 ml	837 ml
Tests	**Properties of freshly mixed and hardened concrete - Exposure classification C-2**										**Properties of freshly mixed and hardened concrete - Exposure classification F-1**			
Slump test (mm)	40	85	70	95	70	45	90	70	60	65	90	90	90	95
Air content (%)	5.8	7.0	5.5	5.5	6.5	5.5	7.4	7.6	6.0	6.5	5.6	7.2	6.6	6.8
Density (kg/m^3)	2,376	2,332	2,382	2,363	2,338	2,283	2,223	2,325	2,334	2,410	2,389	2,321	2,213	2,297
Temperature (^{o}C)	27.1	21.2	26.8	28.0	27.1	27.4	20.3	26.5	27.8	27.0	27.9	27.1	29.0	27.6

Note: Test results in red do not meet the tolerance of ± 30 mm recommended by the CSA standard A23.1-14 for a slump test value prescribed of 80 mm.

2.3.2 Compressive strength of concrete

The determination of the compressive strength was performed in accordance to the CSA standard A23.2-9C. Three concrete specimens were made to determine the compressive strength of concrete at 7 and 28 days. Test results are shown in Table 8 for each concrete mixture.

Table 8: Compressive strength

Exposure classification	Source	Compressive strength (MPa)			
		7 days	28 days (1)	28 days (2)	28 days (average)
C-2	5	30.7	37.9	37.5	37.7
	5 second trial	38.7	46.0	45.0	45.5
	11	29.2	33.3	34.1	33.7
	14	31.3	38.1	36.7	37.4
	17	30.9	36.4	35.3	35.9
	19	34.4	40.3	39.0	39.7
	19 second trial	37.4	43.0	44.6	43.8
	22	27.6	33.7	34.5	34.1
	23	30.1	36.4	35.5	36.0
	26	28.8	35.8	35.4	35.6
F-1	5	28.1	33.7	35.1	34.4
	14	28.0	32.1	33.1	32.6
	19	28.6	34.9	35.0	35.0
	23	26.0	32.2	32.6	32.4

According to the requirements of Table 2 of the CSA standard A23.1-14, exposure class C-2 concrete shall have a minimum compressive strength of 32 MPa at 28 days. Test results show that compressive strength of each concrete mixture meets this requirement, with results ranging from 33.7 MPa to 39.7 MPa.

Regarding the exposure class C-2 concrete mixture made for a second time with the aggregates produce by sources #5 and #19, a compressive strength of 45.5 MPa and 43.8 MPa were obtained at 28 days, respectively. These results are slightly higher than those previously obtained for the other exposure class C-2 concrete mixtures. The mixing procedure and the concrete mix design have been analysed and no key factor was identified to be responsible of the development of a higher compressive strength, with the exception of a water-to-cement ratio slightly lower for the second trial of exposure class C-2 concrete made with the aggregates of source #5. The increase of compressive strength may also be associated with lower ambient temperatures recorded during the second trial of exposure class C-2 concrete made with aggregates from sources #5 and #19.s However, we guarantee that the difference observed did not have a significant impact on the performance and durability of concrete.

For exposure class F-1 concrete, the compressive strength shall be greater than 30 MPa at 28 days. According to this, the compressive strength of exposure class F-1 concrete mixtures meets the requirement, with results ranging from 32.4 MPa to 35.0 MPa.

2.3.3 Determination of parameters of the air-void system in hardened concrete

The parameters of the air-void system were determined in accordance with the ASTM standard C457. For each concrete mixture, one concrete specimen was made and then stored in a temperature and humidity controlled chamber for a period of 28 days. The parameters of the air-void system were then evaluated following this curing period. Test results are shown in Table 9.

Table 9: Parameters of the air-void system in hardened concrete

Exposure classification	Source	Spacing factor (µm)	Air content (%)	Specific surface (mm^{-1})
C-2	5	111	3.9	50.4
	5 second trial	99	4.5	51.4
	11	104	3.9	53.5
	14	126	3.4	47.3
	17	115	4.1	47.2
	19	117	3.6	51.4
	19 second trial	140	5.8	32.6
	22	81	5.4	56.1
	22 second reading	99	4.6	52.2
	23	120	4.6	41.8
	26	143	5.0	33.7
F-1	5	93	3.5	60.5
	5 second reading	103	3.9	53.5
	14	96	4.0	52.6
	19	96	5.8	45.3
	23	116	4.7	45.2

According to article 4.3.3.4 of the CSA standard A23.1-14 – *Concrete materials and methods of concrete construction*, the air-void system should meet the following requirements: the average spacing factor determined on samples of the same mix design should not exceed 230 µm, with no single value greater than 260 µm; and air content should be greater than or equal to 3.0% in the hardened concrete. The spacing factor and air content measured for each concrete mix meet these criteria. This indicates that the concrete mixtures are composed of an air-void system able to withstand repeated cycles of freezing and thawing.

High specific surfaces were obtained for exposure class F-1 concrete made with aggregates from source #5 and also for exposure class C-2 concrete made with aggregates from source #22, with values of 60.5 mm^{-1} and 56.1 mm^{-1}, respectively. These results can be explained by a larger amount of small air voids have been observed in comparison to a concrete sample with a specific surface between 20 mm^{-1} and 40 mm^{-1}. Also, the air voids are very close to each other, which would explain that a spacing factor of 81 µm and 93 µm were measured for exposure class C-2 concrete made with aggregates from source #22 and exposure class F-1 concrete made with aggregates from source #5, respectively. In order to verify and confirm these test results, the concrete samples were analysed for a second time. Test results appear to be similar to those obtained from the first reading, which confirms that the air-void system consists of a larger amount of small air voids very close to each other.

2.3.4 Determination of concrete resistance to scaling

The resistance to scaling of concrete subjected to freeze-thaw cycles and in presence of de-icing salts was determined for all concrete mixtures in accordance with the procedure described in Appendix B of the BNQ standard 2621-905 / 2012 test method. Test results are shown in Table 10. For additional informations, photos of the concrete slab surfaces were taken for each measurement interval (e.g. after 7, 21, 35 and 56 freeze-thaw cycles). The photos are shown in Appendix 4.

According to the standard requirements, an exposure class C-2 concrete shall have a mass loss by scaling equal to or less than 0.50 kg/m^2 after 56 freeze-thaw cycles.

Table 10: Concrete resistance to scaling

Exposure classification	Source	Mass loss by scaling (kg/m^2)			
		After 7 cycles	After 21 cycles	After 35 cycles	After 56 cycles
C-2	5	0.03	0.05*	0.07	0.09
	5 second trial	0.08	0.10	0.12	0.12
	11	0.04	0.06*	0.07	0.07
	14	0.21	0.29*	0.34	0.40
	17	0.22	0.26*	0.29	0.32
	19	0.27*	0.31	0.34	0.37
	19 second trial	0.03	0.05	0.05	0.06
	22	0.06*	0.10	0.11	0.12
	23	0.15	0.19*	0.23	0.26
	26	0.16	0.19	0.20	0.21
F-1	5	0.29	0.31*	0.32	0.34
	14	0.11*	0.18	0.20	0.22
	19	0.21*	0.24	0.26	0.28
	23	0.12*	0.18	0.23	0.27

* Popouts were observed on the concrete surfaces.

The test results show that mass loss measured after 56 cycles was less than 0.50 kg/m^2 for all C-2 and F-1 exposure class concretes. Regarding the exposure class C-2 concretes, there appears to be a relationship between the unconfined freeze-thaw test results and those obtained for resistance of concrete to scaling. The mass loss is greater for concrete made with aggregates that showed a higher unconfined freeze-thaw test results.

In fact, the comparison between test results for concrete made with aggregates from sources #5, #11, #22 and #26 with those manufactured with aggregates from sources #14 and #17, show that the mass loss by scaling of the latter was two or three times greater. However, it is important to mention that popouts were observed after 7 cycles for concrete made with aggregates from source #22 and after 21 cycles for the other concretes, with the exception of concrete made with the aggregates from source #26, where the mass loss is only associated with surface scaling of mortar.

Regarding the exposure class F-1 concretes, no relationship was observed. A mass loss ranging from 0.22 to 0.34 kg/m^2 was measured depending on the concrete mixture. Popouts were also observed on concrete slab surfaces after 7 cycles for concrete made with aggregates from sources #14, #19 and #23, and after 21 cycles for concrete manufactured with aggregates from source #5.

According to the BNQ standard 2621-905/2012, test results after 56 cycles may be interpreted through a qualitative evaluation of concrete slab surfaces by using the rating system shown in Table 11. This rating system provides additional informations about the performance of the concrete subjected to this test.

Table 11: Rating system used for interpretation of scaling resistance of concrete

Index	Characteristic of concrete slab surfaces
0	No significant scaling observed
1-A	Very light scaling of surface mortar with no popouts observed
1-B	Significant scaling of surface mortar with no popouts observed
2-A	No significant scaling of surface mortar, but small number of popouts observed
2-B	No significant scaling of surface mortar, but several popouts observed
3	Combination of surface mortar scaling mainly with fragmented coarse aggregate
4	Combination of fragmented coarse aggregate mainly with scaling of surface mortar

* Popouts formation may be caused by the fragmentation of the surface of the aggregate or by sudden breakage of bonding between the surface mortar and the aggregate.

This rating system is intended make it possible to distinguish more easily a mass loss associated with the combination of surface mortar scaling and the presence of fragmented coarse aggregate from a mass loss caused by scaling of the surface mortar only. Table 12 presents the main observations made using this rating system. For additional informations, a visual assessment was performed to determine the proportions of scaling surfaces associated with scaling of surface mortar in comparison with scaling caused by the fragmentation of coarse aggregate.

Table 12: Interpretation of scaling resistance

Exposure classification	Source	Mass loss by scaling after 56 cycles (kg/m²)	Concrete slab surface interpretation according to standard	Visual inspection of the scaling surface	
				Scaling of surface mortar (%)	Scaling associated with the fragmentation of aggregates (%)
C-2	5	0.09	Index: 2-A	5 %	2-3 %
	5 second trial	0.12	Index: 2-A	10 %	< 1 %
	11	0.07	Index: 2-A	5 %	2-3 %
	14	0.40	Index: 3	60 %	25 %
	17	0.32	Index: 3	60 %	15 %
	19[1]	0.37	Index: 4	60 %	10 %
	19 second trial	0.06	Index: 2-A	10 %	2-3 %
	22	0.12	Index: 3	20 %	25 %
	23	0.26	Index: 3	60 %	15 %
	26	0.21	Index: 1-B	65 %	-
F-1	5	0.34	Index: 4	70 %	5 %
	14	0.22	Index: 3	15 %	10 %
	19	0.28	Index: 1-B	60 %	2-3 %
	23	0.27	Index: 3	15 %	15 %

[1] Note that test results for source #19 were possibly affected by a poor concrete surface finish.

Test results presented in Table 12 show that the mass loss as well as the scaling rating index are not completely representative of the durability performance of the concrete. In fact, the visual inspection performed on the concrete slab surfaces makes it possible to identify the extent of the damage associated with scaling of surface mortar in comparison with scaling due to the fragmentation of the coarse aggregate. The visual determination of the proportions of scaling surface revealed that there was a greater amount of popouts observed on the surface of exposure class C-2 concrete made with aggregates that do not meet the requirements of Table 12, if notes 2 and 3 are withdrawn. Concrete manufactured with aggregates from sources #14, #17, #22 and #23 shows a scaling surface caused by the fragmentation of the coarse aggregate (popouts) ranging from 30% to 50%, while only 3% has been estimated for concrete made with aggregates that meet the requirements of Table 12 without notes 2 and 3.

Furthermore, the concrete slabs showing a greater amount of popouts are those containing various proportions of clayed and/or schist carbonated facies, with the exception of concrete made with aggregates from source #22. These aggregates are a perfect example where the presence of clayed and/or schist veneers can be considered to be marginal. The probability to found these clayed and/or schist veneers near the concrete surface and even cause aggregates to pop out, is relatively low. The marginal presence of these clayed veneers is also a reason why a petrographic number of 100 have been attributed to the aggregates from source #22. Thus, there appears to be a relationship between the concentrations of clayed and/or schist carbonated facies and the presence of popouts. The strength, the hardness and the toughness of clayed and/or schist carbonated facies are normally weaker, so that zones of weakness may be generated, leading to the delamination of these facies under certain conditions.

Regarding the exposure class F-1 concrete, it was noticed that concrete made with aggregates meeting the requirements of Table 12 without notes 2 and 3 (e.g. aggregates from sources #5 and #19), shows a greater mass loss by scaling, but the lowest presence of popouts. Without a visual inspection of the concrete slab surfaces, it would have been difficult to demonstrate that this mass loss is exclusively associated with scaling of the surface mortar. For concrete made with aggregates from sources #14 and #23, it can be seen that the proportion of the surface showing damage by scaling is smaller than exposure class C-2 concretes, but the proportion of scaling associated with the fragmentation of coarse aggregate is more significant, according to the visual inspection of the slab surfaces (10% to 15%).

2.3.5 Concrete resistance to rapid freezing and thawing (ASTM C666 procedure A)

The resistance of concrete to rapid freezing and thawing was determined in accordance with procedure A of the ASTM standard C666. Note that procedure A is referring to a period of freezing and thawing in water. Tests were carried out by the concrete research group from the University of Sherbrooke. Four concrete mixtures of exposure class F-1 were tested, e.g. concrete made with the aggregates produce by sources #5, #14, #19 and #23. The technical report produced by the University of Sherbrooke is detailed in Appendix 5.

The resistance of concrete to freezing and thawing in water is normally evaluated according to the following criteria:

- Length change after 300 cycles of freezing and thawing;
- Durability factor after 300 cycles of freezing and thawing.

These two criteria are used to determine the presence or the absence of micro-cracking in concrete caused by the action of freezing and thawing cycles. As specified in standard test method ASTM C666, a concrete will be accepted only if the maximum length change is less than 1,000 µm/m. However, a maximum length change of 500 µm/m can also be used in order to be more conservative regarding the durability of concrete. The length change is evaluated using a length comparator for each measurement interval.

Moreover, according to the ASTM standard C666, concrete must have a durability factor greater than 60% after 300 cycles of freezing and thawing to be accepted, but some publications suggest that this limit value should be increased to 80% to ensure the resistance of concrete against freezing and thawing cycles. The durability factor is determined on the basis of wave propagation measurements in concrete specimens.

For additional informations, photos of the concrete specimens for each measurement interval are shown in Appendix 6. Table 13 shows the average test results obtained after more than 300 cycles of freezing and thawing in water.

Table 13: Resistance to rapid freezing and thawing in water

Exposure classification	Source	Mass variation (%)	Length change (µm/m)	Durability factor (%)
F-1	5	+ 0.3	123	100
	14	- 0.1	159	99
	19	0	103	97
	23	- 1.1	235	99

Test results demonstrate that the acceptance criteria were met for all concrete mixtures in terms of resistance to freezing and thawing cycles. It was also noted that the concrete made with aggregates that do not meet the requirements of Table 12 show a mass variation and a length change slightly greater than concrete manufactured with aggregates meeting the requirements. Indeed, the aggregates from source #23 show a mass variation equal to 1.1% and a length change of 235 µm/m. These aggregates do not meet the requirement for the micro-Deval test method despite note 3, but are close to the limit value for the unconfined freezing and thawing resistance test method without note 2. A mass variation close to zero and a length change of 123 µm/m and 105 µm/m were obtained for the concrete made with aggregates from sources #5 and #19, respectively. It is important to mention that these latter meet the requirements of Table 12 of the CSA standard A23.1 without notes 2 and 3.

Regarding concrete manufactured with aggregates meeting the unconfined freeze-thaw resistance test requirement because of note 2 (source #14), test results indicate that the mass variation and the length change are slightly greater than the results obtained for concrete made with aggregates from sources #5 and #19. In addition, it is important to note that despite a greater mass variation and length change for sources #14 and #23, a durability factor of 99% was obtained. Given these results, all concrete mixtures have shown a good durability performance against freezing and thawing cycles in water.

3 CONCLUSIONS

Based on test results obtained so far for this testing program, these results suggest that exposure class C-2 and F-1 concretes made from aggregates with physical properties normally accepted due to notes 2 and 3 of Table 12 of the CSA standard A23.1-14, are found to meet the acceptance criteria of performance tests under freezing and thawing conditions.

However, test results also show that the qualification of aggregates for use in concrete subjected to freezing and thawing conditions may be influenced by predominant factors different than the intrinsic properties of the aggregates. According to the resistance of scaling test results, the observed popouts are mainly caused by two key factors. The first one is the proximity of the aggregates to the concrete surface regardless of its physical and mechanical properties (e.g. unconfined freeze-thaw and micro-Deval test results) or its petrographic characteristics. The second factor is the presence of clayed and/or schist veneer observed on the aggregate surfaces. This latter is an important factor because these clayed veneers are found to be responsible for the popouts observed on the concrete specimen's surfaces. The evaluation of the scaling residues also suggests that the fragmented scaling residues are essentially associated with the clayed veneers.

Given these results and observations, we consider that there are some justifications for note 2 of Table 12, which allows an extension of the unconfined freeze-thaw test requirement from 6% to 9%, to be maintained in the next revision of the CSA standard A23.1. We therefore recommend that this note should be rewritten in order to clarify the statement and to include the exposure class F-2. Indeed, exposure class F-2 concrete (concrete in an unsaturated condition exposed to freezing and thawing, but not to chlorides) is normally less exposed to poor conditions than the exposure class F-1 (concrete exposed to chlorides and / or freezing-thawing in saturated condition), which is included in notes 2 and 3. Also, test results for the resistance of concrete to freeze-thaw cycles in water (ASTM C666 procedure A) have shown that exposure class F-1 concretes made with aggregates that whether or not meet the requirements of Table 12, have a good durability performance under freezing and thawing conditions. This distinction will allow a more appropriate use for better quality aggregates thus for a better management of aggregates sources.

Moreover, we think that it would be interesting, for a more or less short-term, that a standard technical committee be formed to further deepen the knowledge about the influence of these clayed veneers in order to possibly establish an acceptance limit of those clayed veneers for an acceptable concrete surface from an aesthetic point of view.

Regarding the micro-Deval abrasion test results, we believe that the influence is marginal on the concrete resistance to freezing and thawing. Thus, the extension of the limit from 17% to 19% allowed by note 3 does not represent a decisive factor in the appearance of popouts.

Consequently, we therefore recommend that note 3 must be maintained but redrafted in order to clarify the statement and also to include the exposure class F-2 for the same reasons as those mentioned previously.