ACI CRC 117
Recommendations for Unified Durability Guidance in ACI Documents

Final Report

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1 EXECUTIVE SUMMARY

The concrete design and construction industry needs uniform guidance on durability recommendations across ACI documents. As part of the preparation for this proposal, five ACI documents, where discrepancies exist, were examined: ACI 201.2R-16, 222R-19, 301-16, 318-19, 350.5-12, and 350-06 [1-6]. These five documents contain the main guidance that informs our current concrete design and building practice regarding durability requirements and exposure category descriptions. Unfortunately, there is disagreement between these documents in providing 1) allowable chloride limits in new concrete for corrosion resistance; 2) water to cementitious material ratio (w/cm), strength requirements and air content (volume %) for freezing-thawing resistance; and 3) w/cm and strength requirements for resistance to sulfate attack. Further complicating matters there is no standard language to define the various exposure category descriptions for each deterioration mechanism. A systematic and rigorous statistical analysis of existing data is anticipated to provide an excellent resource to establish unified guidance by the aforementioned ACI Committees. This effort would represent an important clarification to our current code and guidance documents and would be a welcome and needed advancement for the concrete design and construction industry.

Existing concrete durability data from field exposure sites, published literature, and Army Corps of Engineers infrastructure evaluations were analyzed to make recommendations on unified durability limits and exposure category descriptions. Project outcomes include: 1) Recommendations to ACI Committees 201, 222, 301, 318, and 350 on unified durability requirements and exposure category descriptions; 2) Identification of knowledge gaps and future studies to address deficiencies in existing data sets; and 3) A detailed research methodology that leverages modern field performance and lab testing to inform the development of performance-based standards. Furthermore, the completed work details the need for improved test methods to further validate the reliability of specified mixture proportions and material parameters for durability.

2 OBJECTIVES AND SCOPE

The objective of the proposed study is to analyze existing concrete durability data from field exposure sites, laboratory testing and published literature to make recommendations on unified durability and exposure class descriptions to ACI Committees 201, 222, 301, 318 and 350. At the end of the project the following outcomes are envisioned:

1) Make recommendations to ACI committees (201, 222, 301, 318, 350) on unified durability requirements and exposure class descriptions;
2) Identify any knowledge gaps in existing data sets and make the case for new studies investigating long-term testing to address these deficiencies and
3) Make recommendations to better link field performance and lab testing to standards development, specifically performance-based specifications. This will also inform the needs for development of new and or improved test methods.

Scope

The research and recommendations in this report are limited to the following specific durability related issues:

- Chemical sulfate attack from external sources of sulfate, delayed ettringite formation nor physical salt attack are covered;
- Allowable chloride limits for fresh concrete, chloride thresholds for corrosion initiation are not covered;
- Freezing-and-thawing of concrete, salt scaling is not covered.

3 OVERALL RESEARCH APPROACH

3.1 Project Tasks

To accomplish the objectives of this research project the following tasks were completed by the research team. The PI responsible for leading each task is also identified.

3.1.1 Task 1: Industry Advisory Panel

The Industry Advisory Panel was comprised of members from several organizations representing a diverse cross section of the concrete field: National Ready Mixed Concrete Association (Colin Lobo), Army Corps of Engineers ERDC (Robert Moser), Portland Cement Association (Paul Tennis) and ACI Committees: 201 (Tom Van Dam), 222 (David Trejo), 350 (John Ardahl), 318 (Doug Hooton) and 301 (Thomas, #220). The panel is under the direction of Anthony Bentivegna (Jensen Hughes).

3.1.2 Task 2: Procure and organize all relevant data from existing literature, laboratory testing and field exposure sites as well as field experience in real structures related to the following durability concerns:

1. Critical parameters for freeze-thaw performance including relative dynamic modulus changes, mass loss and visual inspection, and microscopic investigations; (Dr. Kurtis)
2. Critical parameters for external sulfate attack performance including length change, mass loss, visual inspection and microscopic investigations (Dr. Thomas and Dr. Drimalas) and;
3. Key performance indicators for available chloride content in concrete including relevant literature on chloride threshold data for corrosion initiation (Dr. Ideker)

3.1.3 Task 3: Do rigorous statistical analysis on the organized data from Task 2.

This may include regression analysis and/or Monte Carlo simulations to establish reliability parameters for ultimately establishing specification limits for guidance documents. Other statistical methods and data refinement were included where appropriate and are outlined in the respective sections for each durability issue.

3.1.4 Task 4: Establish the recommended specifications/performance indicators.

Based on the analysis in Task 3, recommended requirements for concrete mixtures and/or performance indicators for each type of deterioration are presented. Current specifications often rely just on w/cm and/or strength. Use of performance-based alternatives where appropriate, were also explored. Knowledge gaps or missing data sets where additional lab testing or field verification are needed were identified.

3.1.5 Task 5: Assess the recommended specifications/performance indicators.

In conjunction with the Industry Advisory Panel the established specifications/performance indicators and exposure classifications are proposed in this report.

4 FREEZING AND THAWING

4.1 Summary of Freeze-Thaw Exposure Classes, Specifications, and Recommendations

There are four general exposure classes found in ACI documents [1,4-6] that are concerned with concrete specification for freezing-and-thawing (FT) durability. Table 1 and 2 display specifications and recommendations for minimum compressive strength, maximum water-to-cementitious-materials (w/cm) ratio, and total air content as a function of nominal maximum aggregate size (NMAS) as established by ACI committees 201, 318, and 350 [1,4-6]. For all ACI documents, the F0 exposure class refers to no exposure to FT cycles. The remaining exposure classes (i.e., F1-F3) are distinguished by the expected saturation state of the concrete and potential exposure to deicing chemicals. The F1 exposure class refers to exposed concrete placements in with low likelihood of saturation and exposure to FT cycles. The F2 and F3 exposure categories apply to exposed concrete placements with high likelihood of saturation and exposure to FT cycles, where F3 is distinguished by additional exposure to deicing chemicals. It is important to note that ACI 201.2R-16 has two F3 categories (a and b) that are delineated by the method in which the surface of the concrete is finished. F3a is hand-finished concrete whereas F3b is formed and machine-finished [1]. As detailed in R26.4.3.1(a) of ACI 318-19, ACI 318 does not address differences in surface finishes throughout the document. Considering Table 1, it is apparent that ACI 350 has a singular recommendations for FT durability, differing from ACI 201's recommendations and ACI 318's specifications.

Table 1: Minimum strengths and maximum water to cementitious materials ratio based on FT exposure class from different ACI documents [1,4-6].

Exposure Class	ACI 201.2R-16		ACI 318-19		ACI 350-06	
	Min $\overline{f_c}$ # ksi (MPa)	Max w/cm	Min f_c'** ksi (MPa)	Max w/cm	Min f_c' ksi (MPa)	Max w/cm
F0	None	None	2.5 (17)	N/A	-	-
F1	3.5 (25)	0.5	3.5 (25)	0.55	-	-
F2	3.5 (25)	0.45	4.5 (32)	0.45	4.5 (32)	0.42
F3 (a/b)*	4.5 (32)	0.45§	5.0‡ (35)	0.40‡	-	-

*ACI 201.2R-16 delineates exposure class F3 based on surface finish. F3a is hand-finished concrete with maximal limits on the total amount of allowable supplementary cementitious materials content. F3b is concrete that is formed and machine-finished.

#Min $\overline{f_c}$ is defined in ACI 201.2R-16 as the minimum average compressive strength that should be achieved before initial exposure to freezing and thawing [1].
§A lower w/cm may be needed when corrosion is of concern (ACI 201.2R-16) [1].
** Min f_c' is defined by ACI 318-19 as the minimum specified compressive strength of concrete [4].
‡For plain concrete (i.e., non-reinforced concrete), the maximum w/cm shall be 0.45 and the minimum f_c' shall be 4.5ksi [4].

To date, the definitions for the strength requirements associated with ACI 318 and ACI 201 are fundamentally different. Exact definitions of the respective minimal strength requirements can be found in the footnotes of Table 1. The intent of ACI 201's recommended minimum $\overline{f_c}$ is to ensure a minimal compressive strength prior to initial exposure to FT cycling [1]. The values found in ACI 201.2R-16 for min $\overline{f_c}$ are based upon findings from a study conducted by P. Klieger in 1957 that evaluated the minimum required curing time to impart scaling resistance [7]. As detailed in Section R19.3 of ACI 318-19 [4], the minimal f_c' values are established to be consistent with the specified maximum w/cm, which is selected to ensure sufficient resistance to fluid penetration. Furthermore, Section R26.4.3.1(a) of ACI 318-19 states that, "the Code does not include provisions for…temporary freezing-and-thawing conditions during construction…" – identifying the fundamental departure between the ACI 201 and ACI 318 strength provisions [4]. Despite the difference in definition of f_c' and $\overline{f_c}$, when the ACI 318's min f_c' and max w/cm values for the F3 exposure category are relaxed to consider non-reinforced concrete the ACI 318 and ACI 201 design requirements are in agreement in terms of value.

As shown in Table 2:, ACI 201 recommends higher values for the total air content than those specified by ACI 318 [1]. ACI 201 obtains the design values, as shown in Table 2, by requiring 18% air content in the paste fraction of the concrete mixture with the associated maximal w/cm for each exposure category. The values recommended for total air content in ACI 201.2R-16 are based upon two studies [8,9] conducted by P. Klieger in 1952 and 1956 that evaluated the influence of NMAS and total air content in the paste fraction on net expansion after 300 FT cycles. Although the technical basis for which ACI 318 establishes their target air content values was not identified, the values are similar to those recommended in ACI 201.2R-16 and are likely informed by decades of design experience. Interestingly, Section R26.4.3 of ACI 318-19 clearly states, "it is not the responsibility of the licensed design professional to proportion concrete mixtures," despite that a requirement for total air content directly influences volumetric proportioning [1]. This finding, although non-technical, establishes the extent to which durability requirements can be developed. Lastly, restrictions on the use of supplementary cementitious materials (SCMs) are only required for F3a exposures per ACI 201.2R-16, whereas ACI 318-19 places restrictions on SCM use for its F3 exposure category due to its sole consideration of building structures as detailed in Table R19.3.1, which defines example structural members for FT exposure classes for the reference of design engineers. Allowable cementitious material replacement percentages, which are equivalent in definition and value amongst ACI 318 and ACI 201 documents, are shown in Table 3.

Table 2: Average total air content for concrete based on FT exposure class from different ACI documents [1,4].

Nominal Maximum Aggregate Size in (mm)	**Air Content (%)***			
	Exposure Class: F1		Exposure Class: F2 and F3	
	ACI 201.2R-16*	ACI 318-19	ACI 201.2R-16	ACI 318-19
3/8 (9.5)	7.0	6.0	7.5	7.5
½ (13)	7.0	5.5	7.0	7.0
¾ (RILEM Committee 119, #45)	6.5	5.0	7.0	6.0
1 (25)	6.5	4.5	6.5	6.0
1.5 (38)	6.0	4.5	6.5	5.5
2 (50)	6.0	4.0	6.0	5.0
3 (76)	5.0	3.5	5.5	4.5

*ACI 201.2R-16 and ACI 318-19 allow for a field tolerance of air content of ±1.5%.

Table 3: Limits on cementitious material replacement percentages for ACI 318 F3 and ACI 201 F3a exposure classes [1,4].

Cementitious Materials	**Maximum Percent of Total Cementitious Materials by Mass***
Fly ash or other pozzolans conforming to ASTM C618	25
Slag conforming to ASTM C989/C989M	50
Silica fume to ASTM C1240	10
Total of fly ash or other pozzolans, slag, and silica fume	50
Total of fly ash or other pozzolans and silica fume	35**

*The total cementitious materials also include ASTM C150/C150M, ASTM C595/C595M, ASTM C845/C845M, and ASTM C1157/C1157M cements.
The maximum percentage should include:
(a) Fly ash or other pozzolans in ASTM C595/C595M blended cement or ASTM C1157/C1157M cement
(b) Slag used in the manufacture of an ASTM C595/C595M blended cement, or ASTM C1157/C1157M cement
(c) Silica fume, ASTM C1240, or present in a ASTM C595/C595M blended cement ASTM C1157/C1157M cement
**Fly ash or other pozzolans and silica fume shall constitute no more than 25 and 10 percent, respectively, or total mass of the cementitious materials.

4.2 Background and Critical Review of Freezing-Thawing ACI Provisions

Due to the difference in strength requirement definitions, comparing the ability for one set of design requirements to yield more durable concrete is challenging. In terms of definition alone, it could be concluded that ACI 201's min $\overline{f_c}$ recommendation is more conservative than ACI 318's min f_c' requirement with respect to early-stage construction as it provides assurance that a sufficient strength is achieved prior to a single FT cycle. Such a conclusion does not consider that ACI 318-19 places the responsibility of cold-weather concreting on the contractor. Furthermore, current compliance requirements, found in Section R26.4.2 of ACI 318-19, only require that 28-day strength evaluations be met in addition to the required maximal w/cm, target air content, and potential SCM replacement limits [1]. Despite this, it is important to note that the minimum specified f_c' of ACI 318 could result in higher late-age strengths, which supports long-term durability.

As detailed in Section 4.1, the maximum w/cm values for F2 and F3 classes associated with ACI 318 and ACI 201 are equivalent but differ in value for the F1 class. As will be shown in Section 4.3, the w/cm of a concrete mixture is not correlated with its FT durability as assessed by the ASTM C666 durability factor (DF). Despite the difficulty in comparing which set of strength and maximum w/cm recommendations will ensure performance, it is well established that increased total air content, when properly distributed, improves FT durability [10-12]. This suggests that ACI 201's total air content requirement would yield more durable concrete mixtures for F2 and F3 FT environments based upon the values shown in Table 1 and Table 2 alone. When this conclusion is evaluated in terms of mixture design, a net increase in total air content has implications for achievable compressive strength, which is critical for mixture compliance.

In addition to reviewing the history of the current recommendations and specifications for FT durability, data and results from published literature, long-term exposure sites, and dam inspections were collected and assessed [13-21]. It was found that long-term exposure sites and dam inspections provided primarily qualitative data that was not supported with information regarding the design of the mixture [13-15]. Due to this finding, results from published studies that evaluated both the FT performance (ASTM C666) and microstructural characteristics (ASTM C457) were invaluable to critically assessing and providing evidence-based recommendations to unify the current FT provisions [16-21].

The state-of-the-art in terms of FT experiment and simulation was also reviewed to provide a holistic basis upon which to inform the unification of current FT recommendations and specifications. Since 1949, it has been known that the mean half-distance between entrained air voids, measured by Powers' spacing factor ($\bar{L}$), is critical to FT resistance [12]. Decades of laboratory testing has confirmed that the spacing factor is a strong predictor for FT durability [10-12] but is not currently specified by any ACI committee. Despite advancements in testing, the measurement of a mixture's spacing factor $\bar{L}$ per ASTM C457 requires a trained petrographer, making the assessment time-consuming and costly. Furthermore, because $\bar{L}$ is often conducted on concrete cores of hardened placements, it has been recommended that $\bar{L}$ be used as a means of mixture pre- or post-qualification [22].

In light of the complications associated with $\bar{L}$ testing and specification, Section 4.2.3.2.4 of ACI 201.2R-16 suggests that total air content is a stand-in for direct specification of the spacing factor

– i.e., it is assumed that the required air content will impart the necessary value of $\bar{L}$ to ensure a FT resistant concrete mixture [1]. ACI 201's conclusion does not coincide with decades of research and hundreds of concrete mixtures have shown that total air content does not guarantee a quality air void system and can be influenced by the combination of different admixtures [23-25]. The findings of these studies suggest that total air content does not fully represent the spacing of the entrained air void system. In effort to unify the current FT specifications and recommendations from ACI committees, it was apparent that current design variables needed to be quantitatively compared in terms of their ability to predict FT performance.

4.3 Evaluating Freeze-Thaw Performance

A database composed of 157 different ordinary portland cement (OPC) concrete mixtures containing no SCMs that were subjected to ASTM C666 Procedure A accelerated FT testing was assembled to assess the influence of mixture proportions and properties on FT performance [16-21]. Mixtures with SCMs were excluded because the current design provisions for allowable SCM replacement are equivalent in value within ACI 201.2R-2016 [1] and ACI 318-19 [4]. As detailed in Section 4.1 and 4.2, common design parameters for FT durability include: w/cm, compressive strength, total air content, air content in the paste fraction, specific surface, and spacing factor. When reported, these parameters were retained for each mixture in the database for comparison to the DF obtained by ASTM C666 testing. The comparison plots shown in Figure 1, Figure 2, and Figure 3 visually evaluate the ability for two variables to correlate with FT durability, as measured by the DF. Following [11,16], passing regions (i.e., regions where the DF is greater than 80) have been filled to display where design parameters can ensure FT performance. It is important to note that Figure 1, Figure 2, and Figure 3 collectively evaluate all possible combinations of the identified design parameters to predict performance and are representative of OPC concrete mixtures with NMAS ranging from ½" to 1". Furthermore, the design parameters associated with the 157 mixtures are representative of standard design space, where values of w/c, 28-day compressive strength, total air content, air content in the paste fraction, and spacing factor range from 0.2 to 0.7, 2ksi to 14ksi, 1% to 9%, 2% to 35%, and 0.004" (0.1mm) to 0.05" (1.27mm) respectively.

Since both ACI 318 and ACI 201 directly attribute concrete durability to resistance to fluid penetration, Figure 1 compares the ability for the water-to-cement (w/c) ratio for the 157 mixtures to predict FT performance as a function of the other design variables. Figure 1a clearly shows that w/c and 28-day compressive strength show no correlation for FT performance, with the exception of one region for very high compressive strengths (i.e. greater than 9ksi) and low w/c (i.e., less than 0.35). Figures 1b and 1c display that concrete mixtures with total air contents and total air content in the paste fraction greater than 4% and 14%, respectively, ensure FT performance – much lower values than currently specified by ACI 318 or ACI 201. Findings from Figure 1d align with commentary found in Chapter 4 of ACI 201.2R-16 (i.e., concrete mixtures with values of spacing factor less than 0.009" (0.23mm) ensure FT performance) [1].

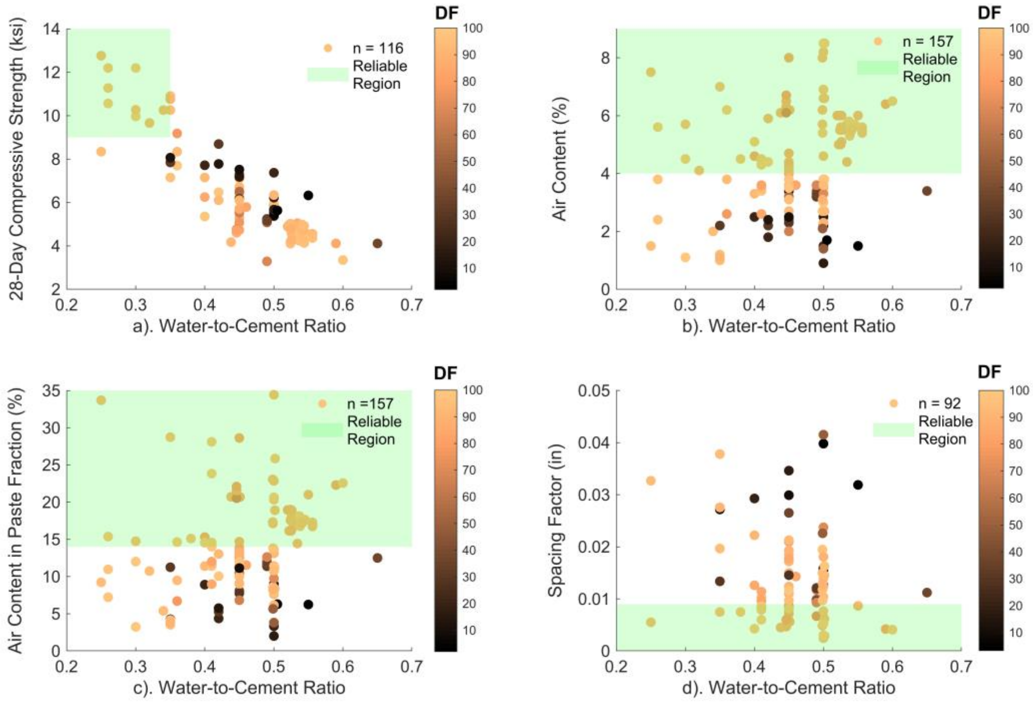

Figure 1: Comparison plot for water-to-cement ratio and a). 28-day compressive strength b). total air content c). total air content in paste fraction and d). spacing factor to ensure FT performance [16-21].

In terms of durability-based design, required compressive strength is discussed in ACI 318 and ACI 201 documents to ensure FT performance following resistance to fluid penetration [1,4]. ACI 318 views the compressive strength as a means through which an appropriate w/cm (or w/c) can be ensured [4]. ACI 201 sees compressive strength specification in terms of available tensile strength to resist the hydraulic and crystallization pressures subjected to the solid phase of concrete during a freezing event [1]. Figure 2, in addition to Figure 1a, compare the ability for 28-day compressive strength and other design variables to ensure FT performance. Similar to Figure 1b and Figure 1c , Figure 2a and Figure 2b, show that concretes with total air content and total air content in the paste fraction greater than 4% and 14% display high FT resistance. Furthermore, Figure 2a and Figure 2b show that concretes with 28-day compressive strength values greater than 9ksi appear to be are highly resistant to FT.

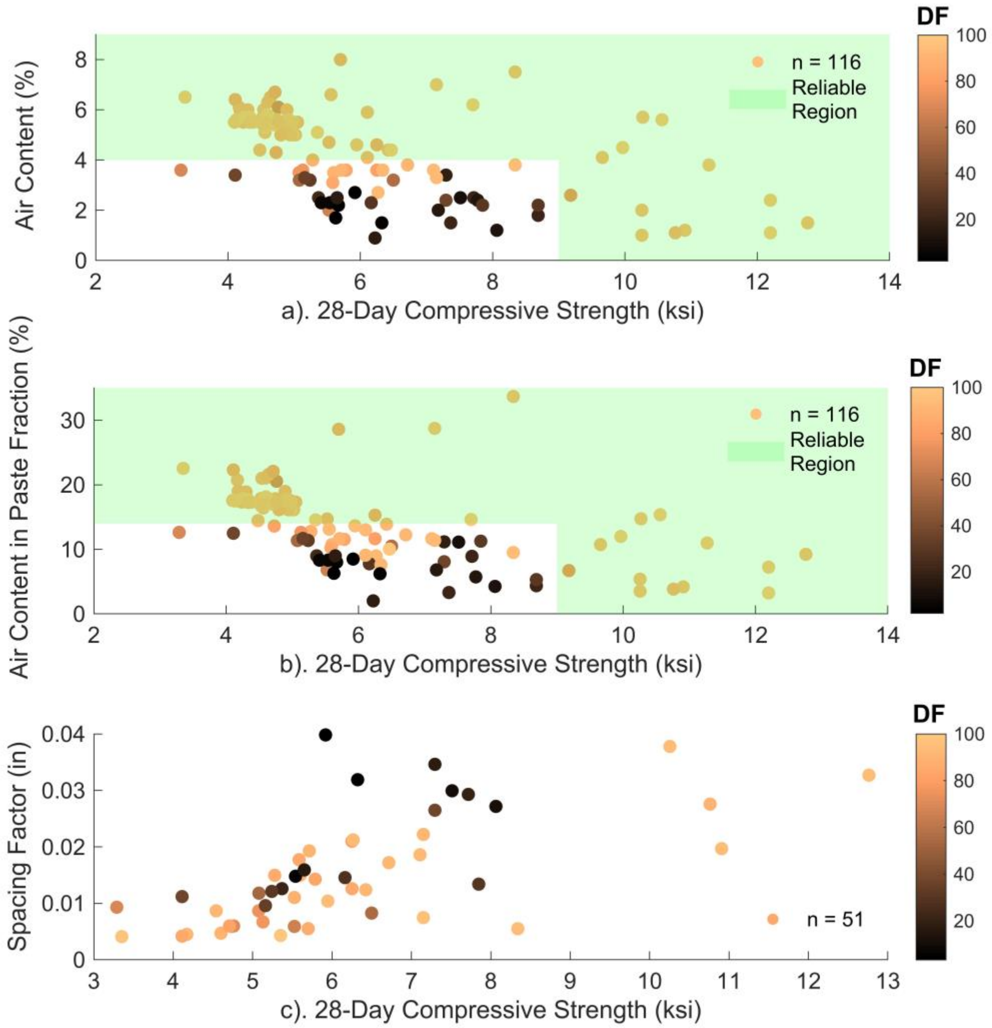

Figure 2: Comparison plot for 28-day compressive strength and a). total air content b). total air content in paste fraction and c). spacing factor to ensure FT performance [16-21].

Figure 3 evaluates the ability for different measures of the entrained and entrapped air void system to correlate with FT durability. Within the commentary found within ACI 201.2R-16 [1], it is clear that the committee sees the quantity (volume) and quality (spacing) of the air void system as the primary means of ensuring long-term FT performance. Figure 3a displays a linear relationship where values of the total air content and total air content in the paste fraction greater than 3.5% and 14% ensure FT durability. Figure 3b shows that concretes mixtures with spacing factors less than 0.02” (0.5mm) with total air content values greater that 3.5% perform well. Similarly, Figure 3c shows that concrete mixtures with spacing factors less than 0.015” (0.38mm) with total air content values in the paste fraction greater than 13% are highly FT resistant.

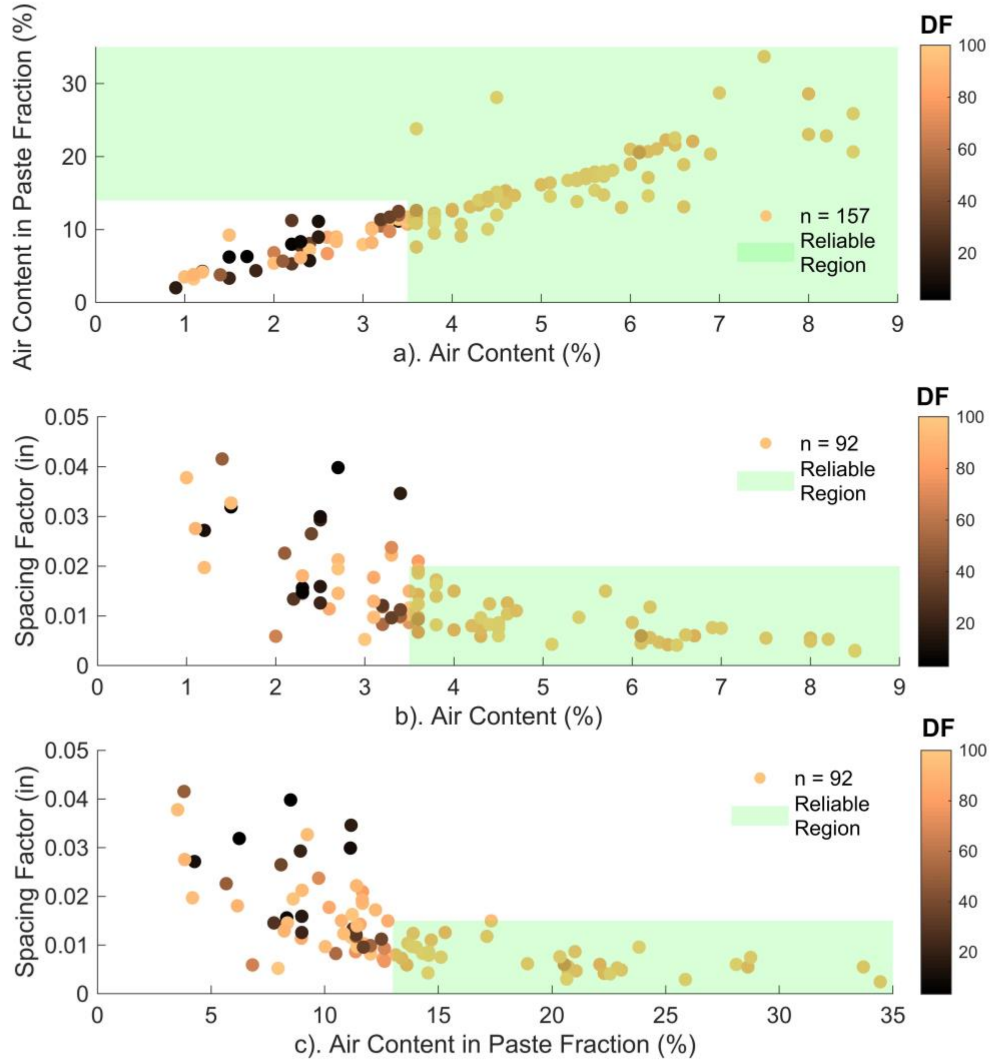

Figure 3: Comparison plot for various measurements to ensure FT performance: a). total air content and total air content in the paste fraction; b). total air content and spacing factor; c). total air content in the paste fraction and spacing factor [16-21].

Based upon the preceding multi-parameter assessment the following conclusion can be derived:

a). Despite ACI 318 and ACI 201's reference to w/cm and w/c as a primary way to control the FT durability of a concrete mixture, the data does not support such a claim. Based upon the comparison showed in Figure 1, total air content, total air content in the paste fraction, and spacing factor govern FT resistance.

b). The compressive strength of concrete does not ensure FT performance, unless greater than 9ksi.

c). ACI 201's practice of designing total air content by requiring 18% air content in the paste fraction is much larger than the 14% threshold found in the preceding assessment, yielding overly conservative recommended total air content values in ACI 201.2R-16 [1].

d). Figure 3b further suggests that total air content and spacing factor are not directly correlated (e.g., a spacing factor of 0.009" (0.23mm), which imparts reliable FT resistance, can be obtained through total air content measurements ranging from 3.0% to 7.0%).

4.4 Freezing-Thawing Recommended Limits

From the results of this investigation, the recommended unified design recommendations for FT performance are shown in Table 4-Table 7. Based on the completed work, the authors suggest the adoption of ACI 318-19's exposure class descriptions, maximum w/cm values, and minimum f_c' definition and values. Additionally, the use of the minimal total air content values, shown in Table 6, are suggested in comparison to the allowable ranges specified in ACI 201.2R-16 and ACI 318-19. Furthermore, pending the criticality of the concrete mixture, the authors see a need to clearly detail to licensed design professionals additional laboratory-based testing options for either pre-qualifying or post-validating that a concrete mixture will perform reliably in a FT environment.

The proposed guidance on the unification of FT durability specifications are detailed and justified as follows:

- Upon considering the commentary found in R19.3 of ACI 318-19, the FT exposure class definitions established by ACI 318 and ACI 201 are similar. The major difference is the discrepancy between exposure class F3 where ACI 201 distinguishes between the surface finish type, which has implications for allowable SCM replacement. Due to the fact that ACI 318-19 does not distinguish between surface-finish, as detailed in R26.4.3.1(a) of ACI 318-19, and all concrete mixtures must comply with the compliance requirements established by ACI 318, the SCM limits established for exposure class F3b by ACI 201 are suggested to be removed.
- The proposed adoption of ACI 318-19 maximum w/cm values is substantiated by the findings in Section 4.3 (i.e., w/cm and w/c of concrete were not found to substantially influence FT performance). Due to these findings, adopting lower values of w/cm would be unfounded.
- The design variable min f_c', defined by ACI 318-19, is adopted in comparison to the minimum average compressive strength prior to initial exposure, min $\overline{f_c}$, to ensure uniformity within the code (i.e., ACI 318-19 does not consider potential freezing-and-thawing conditions during construction). The suggested values of min f_c' shown in Table 1 are also retained. The authors additionally encourage the removal of the allowable reduction in air content for concretes with compressive strengths greater than 5ksi – in terms of the analysis conducted, this value is too low.
- It is proposed that the total air content values be expressed as a *minimum total content* (rather than an average total content with an allowable 1.5% tolerance, as is current practice). The values found in Table 6 are based upon 14% and 15% total air content in the paste fraction of the concrete mixture for F1 and F2/F3 exposure classes, respectively, where the absolute minimum total air content for any MSA is 3.5%. The intent of establishing the air content specification as a minimum is justified in two ways. First, the presented analysis displayed that concrete mixtures with greater than 14% air content in paste fraction and more than 3.5% total air content renders the concrete highly FT resistant. Secondly, the new values also yield total air contents near, or below, that were

already required considering the tolerance – implying the minimum specification relaxes the current requirements, especially those recommended in ACI 201.2R-16.

While reviewing the current design provisions and carrying out the presented analysis, it was identified that there is a significant need for improved on-site evaluation techniques of the quality of the entrained air void system and further assessments on the correlated influence of surface finish and SCM replacement on FT durability. To date, various gravimetric metrics are used to quantify the volume of entrapped and entrained air on construction sites, but do not accurately evaluate the quality (i.e., the size and spacing) of the air void system. Recent advancements in variable pressure [23,26], and ultrasonic [27,28], and shock physics techniques have been shown to rapidly quantify the quality of the air void system, but need further testing and evaluation by the construction industry prior to establishment within durability specification. Figure 4 displays that the specific surface of the air void system could serve as a direct way to ensure a sufficiently low value of spacing factor, pending its ability to be accurately predicted. Additionally, Figure 4 supports that concretes with air contents greater than 3.5% and a specific surface greater than 500 in^2/in^3 yields air void systems with spacing factors < 0.015", which are highly FT resistant. As on-site evaluation technologies are improved and the material parameters that dictate the response of concrete to FT are better understood, the presented unified durability specifications can become increasingly performance-based, allowing for furthered service life assurance in FT environments. Table 7 identifies values of specific surface, spacing factor, and durability factor that can be used to uniquely ensure FT durability.

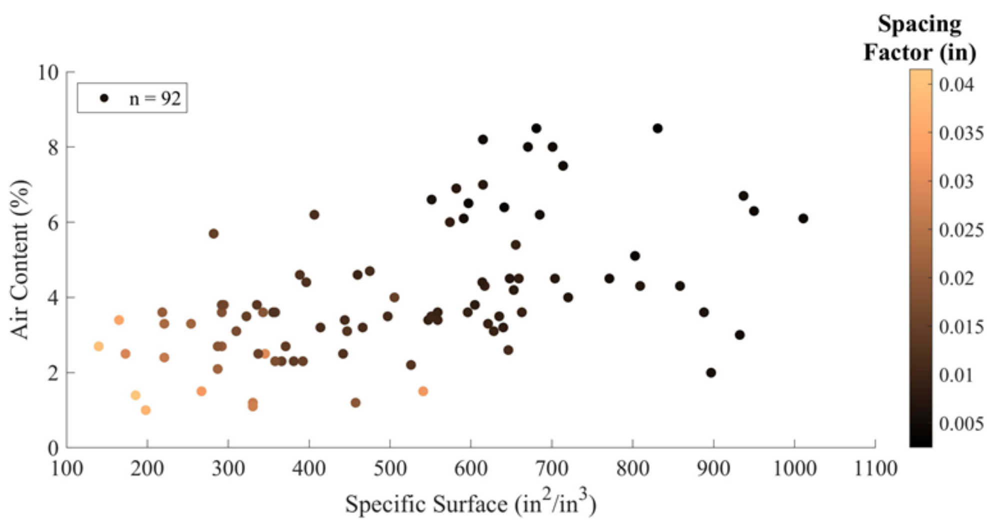

Figure 4: Influence of specific surface and total air content on spacing factor, $\bar{L}$.

Table 4: Unified exposure class descriptions for FT attack.

Exposure Class	Severity	Condition
F0	Not applicable	Concrete not exposed to freezing-and-thawing conditions
F1	Moderate	Concrete exposed to freezing-and-thawing cycles with limited exposure to water
F2	Severe	Concrete exposed to freezing-and-thawing cycles with frequent exposure to water
F3	Very Severe	Concrete exposed to freezing-and-thawing cycles with frequent exposure to water and exposure to deicing chemicals

Table 5: Unified design recommendations for FT performance.

Exposure Class	Min f_c' (psi)	Max w/cm	Min Total Air Content (%)	Limits on Cementitious Material Replacement
F0	None	None	None	None
F1	3500	0.55	See Table 6	None
F2	4500	0.45	See Table 6	None
F3	5000*	0.40*	See Table 6	See Table 3

* For plain concrete (i.e., non-reinforced concrete), the maximum w/cm shall be 0.45 and the minimum f_c' shall be 4500 psi [4].

Table 6: Unified minimum total air content recommendations for FT performance.

MSA	Total Minimum Air Content (%)*	
in	Exposure Class: F1	Exposure Class: F2 and F3
3/8	5.0	6.0
1/2	5.0	6.0
3/4	4.5	5.5
1	4.5	5.5
1-1/2	4.0	5.0
2	4.0	5.0
3	3.5	4.5

* Determined by an average of three assessments at the point of placement where no value is less than 0.5% of the required minimum.

Table 7: Individual FT durability assessment test metrics recommended for mixture pre-qualification or post-assessment for critical concrete placements.

Exposure Class	Min Specific Surface*, (in^2/in^3)	Max Spacing Factor**, (in)	Min Durability Factor#**
F0	None	None	None
F1	500	0.015	80
F2/F3	600	0.009	90

* Requiring that the air content is greater than or equal to 3.5% and measured by ASTM standard C457, or some other experimentally validated means, as the average of three assessments, where no assessment has a value is less than 20% of the mean of the samples.
** Measured by ASTM standard C457 as the average of three assessments from a single mixture during construction, where no assessment has a spacing factor greater than 0.018" or 0.012" for the F1 and F2/F3 categories, respectively. Note that with values of air content, paste fraction, and specific surface, the spacing factor may be calculated. The spacing factor values presented herein are denoted to ensure they align with previous performance based tests in the case that the previous values are not available.
#** Measured by ASTM standard C666, by either procedure A or B, as an average of three separate specimens that are representative of the same concrete mixture.

5 SULFATE ATTACK

5.1 Summary of Sulfate Attack Specifications and Recommendations

The minimum compressive strength and maximum w/cm for concrete in different sulfate exposure classes is shown in Table 9. ACI 201 does not specify any minimum design strength, only a maximum w/cm whereas ACI 318 and 350 specify both parameters. However, none of the strength or w/cm requirements agree across all sulfate exposure classes. This is interesting as this is one area where all the ACI documents define the sulfate exposure classes in a uniform manner based on sulfate in soil or water. No guidance on physical salt attack is given. This is another area where considerable attention is needed.

Table 8: Minimum design strengths and maximum water to cementitious materials ratio based on sulfate attack exposure category from different ACI documents [1,4,5]

Exposure Class	ACI 201.2R-16		ACI 318-19*		ACI 350-06	
	Min f'c (psi)	*Max w/cm*	*Min f'c (psi)*	*Max w/cm*	*Min f'c (psi)*	*Max w/cm*
S0	-	None	2500	N/A	-	0.45
S1	-	0.50	4000	0.50	4500	0.42
S2	-	0.45	4500	0.45	5000	0.40
S3	-	0.40	4500/5000	0.45/0.40	5000	0.40

* ACI 318-19 was recently revised to allow two options for S3 exposure. Option 1 maintained the requirements that were in ACI 318-14 which permits $f'_c \geq 4500$ psi and $w/cm \leq 0.45$ in combination with a Type V cement (ASTM C150) plus pozzolan or slag or a cement designated as Type HS (ASTM C595 or C1157) plus pozzolan or slag. The amount of pozzolan or slag has to be demonstrated by service record or testing (ASTM C1012) to improve the sulfate resistance when used with Type V cement. Option 2 requires $f'_c \geq 5000$ psi and $w/cm \leq 0.40$ used with a Type V cement (ASTM C150) or a cement designated as Type HS (ASTM C595 or C1157); there is no requirement for additional pozzolan or slag.

5.2 Factors Affecting Chemical Attack of Concrete in Sulfate Soils or Groundwaters

Problems with concrete structures in alkali soils and waters were first reported in the U.S.A. in 1908. However, numerous studies in Europe predated this discovery among the first being in 1887 [29-31]. Jewett (1908) [32] in the U.S. and Bied (1909) [33] in France were the first to report the beneficial use of pozzolans and this was later confirmed in the U.S. by testing of pozzolans (moler, trass and volcanic ash) and blast furnace slag at the United States Department of Agriculture [34]. Since this time, the phenomenon of sulfate attack on concrete has been extensively researched and there are thousands of publications reporting on measures to increase the resistance of concrete to attack by sulfate soils and groundwaters. There are many incongruities in the findings and conclusions from different studies and this can be explained, at least in part, to variations in the methodology employed. These variations include, but are not limited to, the following:

- *The nature of exposure whether static (e.g. continuous immersion in sulfate solution) or dynamic (e.g. exposure to fluctuations in humidity and/or temperature).* Continuous immersion of specimens limits the mechanisms of deterioration to chemical reactions between the species in solution and the products of hydration; also the mechanism of mass transport is limited to ionic diffusion. This type of attack is often referred to as "classical (or chemical) sulfate attack". Exposure to cycles of wetting and drying promotes degradation due to salt crystallization and, possibly, hydration-dehydration of salts (such as thenardite ↔ mirabillite; $NaSO_4 \leftrightarrow Na_2SO_4 \cdot 10H_2O$) in addition to the transport of mass by capillary suction. This is often referred to as “physical salt (or sulfate) attack” in the literature to distinguish this from “classical sulfate attack”. The nature of deterioration and the influencing factors may be quite different in these different exposure conditions. As a consequence ACI 201 *Guide to Durable Concrete* deals with these two phenomena separately. An example of the important role of static versus dynamic exposure can be seen in the findings of PCA’s long-term test program [35] where SCMs generally improved the durability of concrete continuously immersed in 6.5% Na_2SO_4 but reduced the resistance of concrete subjected to wetting and drying in the same solution.

- *The type and concentration of the sulfate solution.* In an attempt, presumably, to accelerate the deterioration, many researchers have employed very high concentrations of sulfate (such as 10% sodium sulfate, 67,606 ppm SO_4^{2-}) as employed in test method USBR 4908-86 Method B [36] which are many times higher than the highest sulfate concentrations found in soils and ground waters (e.g. around 15,000 ppm SO_4^{2-}). Furthermore it is well established that the nature of the cation associated with the sulfate has considerable impact on the rate, degree and form of deterioration; the aggressiveness of the cation of the sulfate solution increases in the order of calcium $\rightarrow$ sodium/potassium $\rightarrow$ magnesium.
- *The nature, size and age of the specimens used.* Cement pastes, mortars and concretes have been used and this can impact the nature of the deterioration. Also with larger samples early signs of damage are limited to mass loss at the surface with little or no bulk expansion of the sample being detected. The age or maturity of the specimen at the time of initial exposure can significantly impact the sulfate resistance of systems containing slowly-reacting pozzolans such as Class F fly ash. Furthermore, conditioning, such as air-drying of the sample, prior to exposure has been found to improve sulfate resistance [37].

Notwithstanding the significance of these issues on the outcome of test programs there does exist a general consensus among studies where specimens have been exposed to continuous immersion of “classical/chemical sulfate attack”; there are two main influences governing resistance; these are:

- *The composition of the binder*. In chemical sulfate attack the most vulnerable phases and the first to be attacked are the calcium aluminates (and calcium hydroxide, CH). Low-C_3A portland cement has a much higher resistance to attack resulting from continuous immersion in sulfate solution. Other factors, such as the C_3S/C_2S ratio, have also been found to have an impact on sulfate resistance of portland cement but to a much lesser extent than the C_3A content. Composition also has a significant impact on the efficacy of fly ash in terms of sulfate resistance. The ability of low-calcium (Class F) fly ash to increase sulfate resistance has been known since the earliest tests on fly ash in the 1930’s [38] and further long-term testing of concretes by the USBR [39,40]. Later work by the USBR [41] showed that this could not be extended to high-calcium (Class C) fly ashes which may actually reduce sulfate resistance. The reduced sulfate resistance is attributed to the formation of crystalline C_3A and calcium-aluminate glass in Class C fly ash [42].

The alumina content of slag has also been shown to influence the behavior of slag cements exposed to sulfates [43]. In general the use of SCMs enhance the chemical resistance of the binder by diluting the C_3A and CH available, by consuming CH by pozzolanic reaction and, possibly, the formation of more resistant C-A-S-H hydrates (such as Strätlingite). In addition, SCMs also reduce the permeability of the concrete.

- *The permeability of the concrete.* The permeability determines the physical resistance of the concrete to the penetration of sulfates. Reducing the *w/cm* of the concrete has a profound effect on the sulfate resistance of concrete [1,44]. The improved sulfate resistance observed when sufficient amounts of suitable SCMs are used is largely attributed to the significant reductions in concrete permeability that may be achieved with these materials.

Most specifications for concrete exposed to sulfates in service employ one or more of the following strategies to improve the resistance of the concrete to chemical attack:

- Impose maximum *w/cm* limits (often in association with minimum specified strengths)
- Require the use of sulfate-resistant cements
- Use a sufficient amount of SCM

These approaches are adopted by all of the ACI documents included in this study and the documents differ only in the finer details. The main points of contention are:

- What is the appropriate maximum *w/cm* for an S3 exposure (0.40 and 0.45 used by ACI 201 and 318, respectively)?
- Should blended cements meeting ASTM C595 with the -HS suffix or hydraulic cements meeting ASTM C1157 requirements of Type HS be permitted in S3 exposure? ACI 201 and 318 allow these cements as alternatives to Type V in S3 but ACI 350 requires the use of Type V (with pozzolans and slag) in S3.

5.3 Effect of *w/cm*

Monteiro and Kurtis presented an analysis of data from more than 100 concrete mixtures from the USBR's long-term study in which concrete cylinders (76 x 152 mm) were partially submerged in a solution of 2.1% Na_2SO_4 or 14,200 ppm SO_4^{2-} (S3 exposure) for periods of over 40 years [45]. Figure 5 shows the time-to-"failure" (age when expansion exceeds 0.05%) plotted against *w/cm* for concrete (without SCM) produced with cements with C_3A contents from 0 to 8%. Monteiro and Kurtis [46] proposed a safe zone with w/cm < 0.45 where deterioration does not occur provided the C_3A content is less than 8%. This would appear to support the requirements in ACI 318 for S3 exposure which are *w/cm* $\leq$ 0.45 and Type V cement ($C_3A \leq$ 5%) with pozzolans or slag.

It is not clear why there is no apparent increase in performance when the *w/cm* is reduced from 0.45 to 0.40 in Figure 5. This is somewhat counterintuitive as a significant reduction in permeability, and therefore increased physical resistance to the penetration of sulfates, would be expected as a result of this reduction in *w/cm*. It is possible that the effect of w/cm in this range may not become apparent until later ages.

Data from another long-term study [47] do indicate an increase in performance with reductions in *w/cm* below 0.45. In this study concrete prisms were immersed in a mixed solution of sulfates (0.15M Na_2SO_4 + $MgSO_4$) which yields a sulfate concentration of 14,400 ppm SO_4^{2-} (S3 exposure). The concrete was produced with *w/cm* in the range of 0.375 to 0.70 using CSA Type 10 and 50 cements, which are equivalent to ASTM C150 Type I and V, respectively. Some of the concrete mixes contained a high-alkali Class C fly ash (SiO_2 + Al_2O_3 + Fe_2O_3 = 62 to 70%). Figure 6 shows the time taken to reach an expansion of 0.05% and reveals that this time is significantly influenced by *w/cm* especially at values below 0.45.

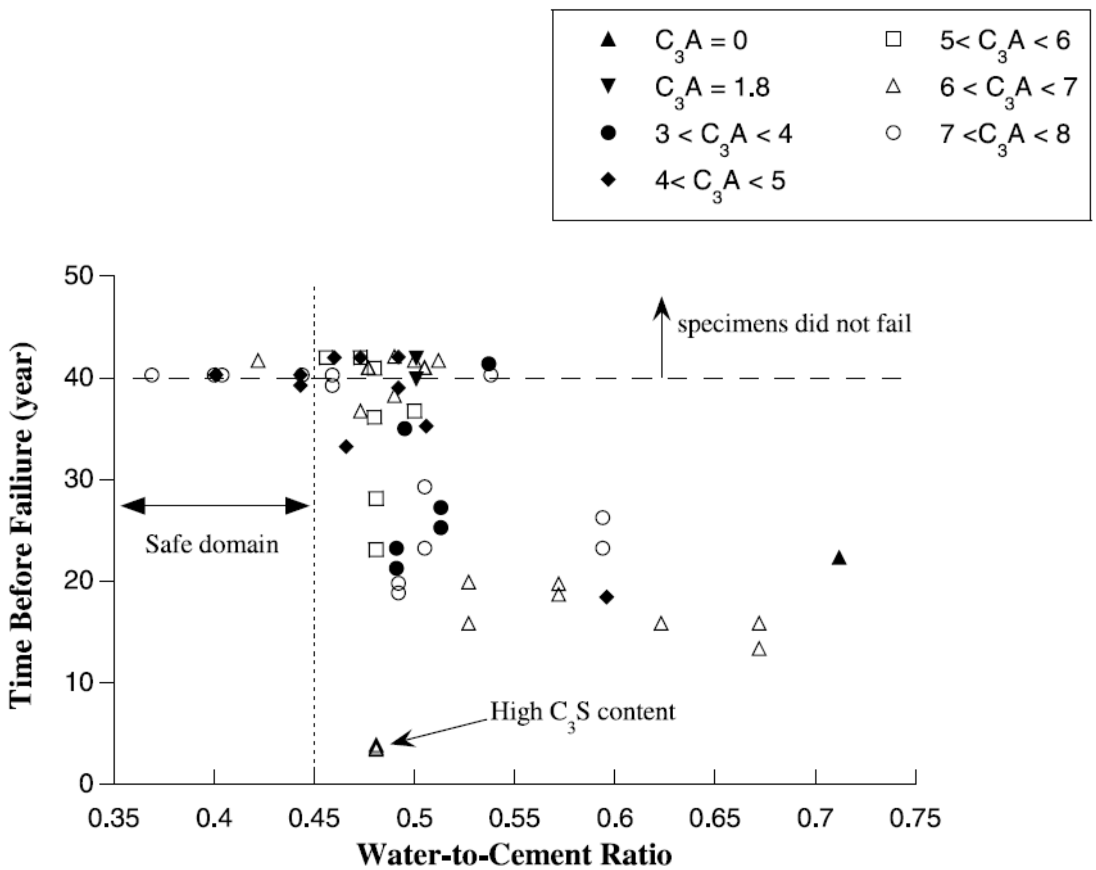

Figure 5: Effect of *w/cm* and C_3A content on time-to-failure in the USBR long-term study (from [46])

5.4 Equivalent performance of cement-SCM blends

There is a substantial body of literature that demonstrates many SCMs increase the sulfate resistance of concrete provided they are used at a sufficient amount of replacement. The level of replacement required with a given cement/SCM combination is frequently determined in the mortar bar method ASTM C1012 using expansion limits of 0.10% at 6 months to demonstrate moderate-sulfate (MS) resistance and either 0.05% at 6 months or 0.10% at 12 months for high-sulfate (HS) resistance. This same test and criteria are used in ASTM C595 and ASTM C1157 to qualify blended and hydraulic cements as being moderate-sulfate (MS) or highly-sulfate (HS) resistance. The question that needs to be addressed for specification purposes is whether blends of cementing materials that meet these requirements for MS and HS provide equivalent performance as, respectively, Type II and Type V portland cements.

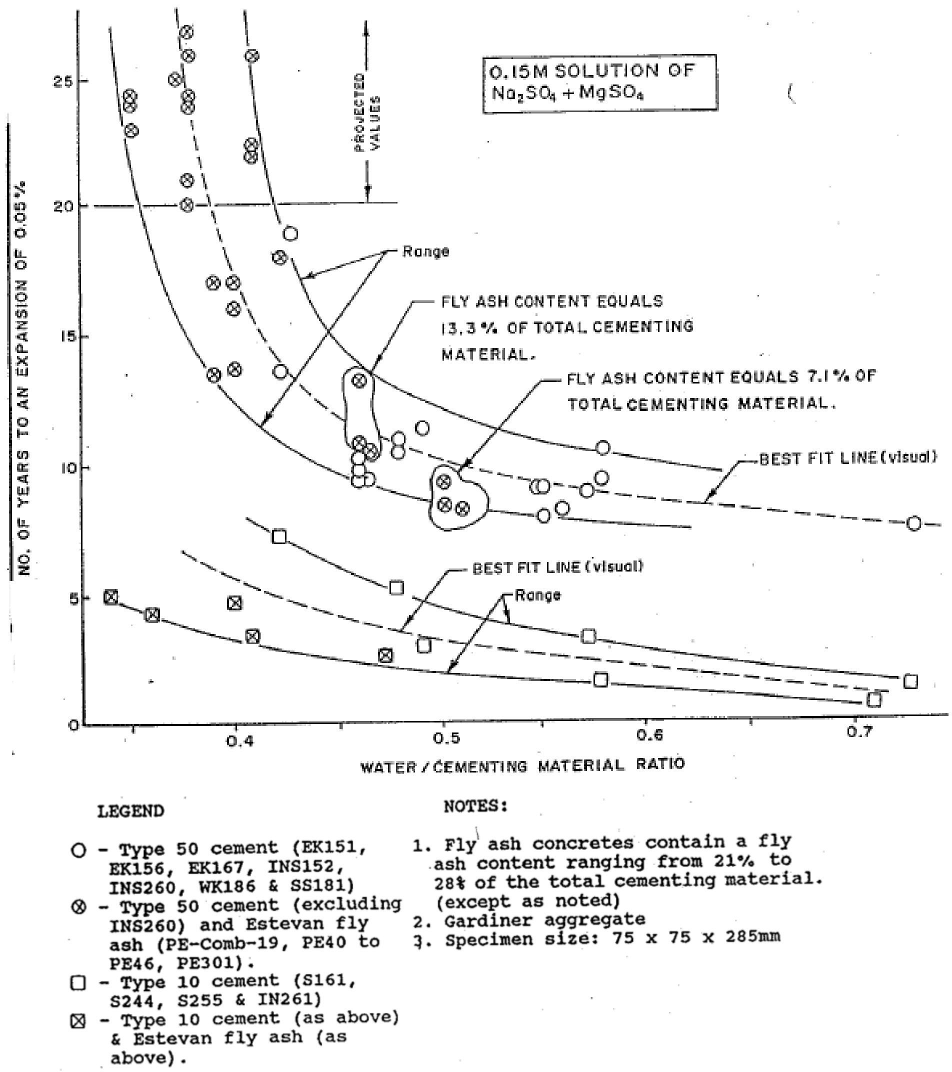

Figure 6: Effect of *w/cm* and cement type on time-to-failure (0.05% expansion) of concrete [47]
Note: Type 50 cement = ASTM Type V; Type 10 = Type I.

Figure 7 shows data from ASTM C1012 conducted on a range of portland cements (Types I, II and V) and blends of Type I portland cement with various types and amounts of SCM [48]. As expected, of the portland cement mortars, the best performance was provided by the Type V cement and the worst by the Type I cement. However, the mortar bars produced with Type V cement starts to expand between 12 and 18 months and actually fail the 18-month expansion limit of 0.10% imposed by ACI 318 and 201 for cements to be used in S3. Blends of Type I cement and sufficient levels of Class F fly ash, slag, silica fume and metakaolin can meet this requirement and show little evidence of expansion beyond this time period.

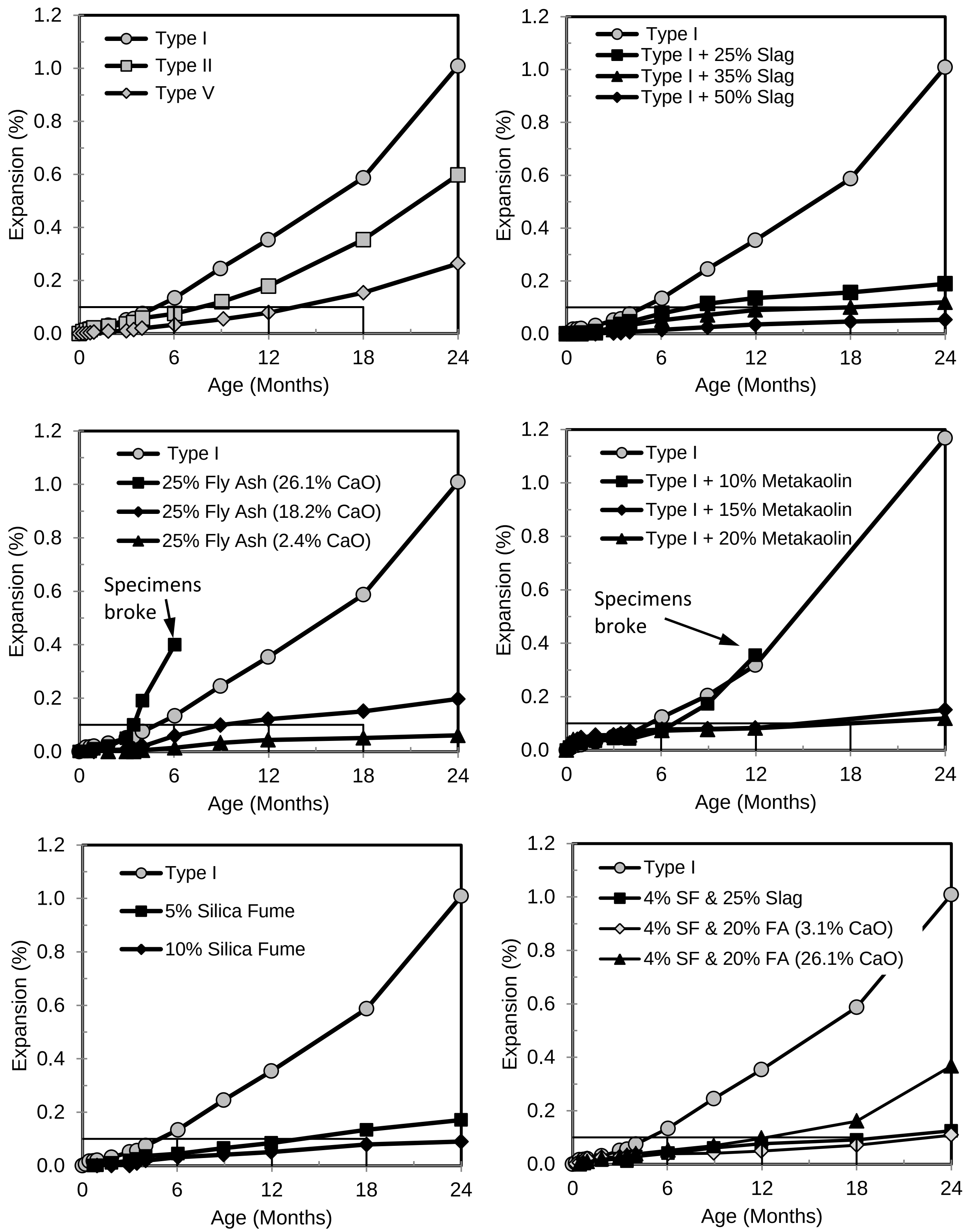

Figure 7: Effect of SCM on expansion of mortars stored in 5% Na_2SO_4 solution as per ASTM C1012 [48]

Figure 8 shows photographs of concrete cubes after 5 years immersion in 1.5% $MgSO_4$ solution (12,000 ppm SO_4^{2-} = S3) solution at the Building Research Establishment (BRE) in the U.K. Concrete produced with the high-C_3A portland cement shows severe deterioration whereas that produced with the same high-C_3A cement and 20% fly ash exhibits little deterioration and performs at least as well as concrete made with low-C_3A sulfate-resisting portland cement. Similar findings were reported by Hooton and Thomas based on field studies of concrete buried below grade in various sulfate solutions (including S3) [49]. The concretes were produced with portland cements (PC) and portland limestone cements (PLC) with up to 15% interground limestone, and with varying levels of different SCMs. Generally, it was found that PC-SCM and PLC-SCM blends with appropriate levels of SCM performed as well or better than concrete produced with ASTM C150 Type V cement.

Figure 8: Photographs of 100 mm (4 in.) cubes after 5 years exposure to $MgSO_4$ solution (1.5% SO_3). Concrete (W/CM = 0.43 to 0.46) produced with ordinary portland cement – 14.1% C_3A (left), sulfate-resistant portland cement – 1.2% C_3A (center), and a blend of ordinary portland cement (14.1% C_3A) plus 20% fly ash (right). [48]

Obla and Lobo recently reported data for a series of concretes produced with blends of portland cement and SCM (fly ash or slag), and compared the performance with concrete manufactured with Type V cement on its own [50]. Concrete with a range of *w/cm* was tested using 10% Na_2SO_4 solution (67,606 ppm SO_4^{2-} >> S3 exposure class) for up to 4 years. Concrete produced with *w/cm* = 0.45 and Type V cement failed (expansion > 0.05%) within the 4-year period. However, with one exception concrete produced with Type II or Type V cement with *w/cm* in the range of 0.40 to 0.60 and either 20% or 30% Class F fly ash, or 35% or 50% slag did not exceed the expansion limit within the 4-year test period. The exception was a mix with Type II cement and 35% slag with *w/cm* = 0.60 which expanded by 0.054% at 4 years. The authors concluded that the concrete produced with appropriate levels of SCM are more sulfate resistant than those produced with Type V cement.

Further analysis of the data by Obla and Lobo [50] indicate that all 15 cement-SCM blends that meet the (ASTM C1012) expansion requirement of ACI 318 Table 26.4.2.2(c) for S3 exposure class perform well when testing in concrete (expansion < 0.05% at 4 years in 67,606 ppm SO_4^{2-} solution) at w/cm of 0.40 and 0.50. Only one of the 15 cement-SCM blends (Type I cement with 25% slag) did not perform well when testing in concrete (“failed” at 19 months) with w/cm of 0.60.

5.5 Sulfate Attack Recommendations

The recommendations based on the findings in this project are summarized in Table 9 and Table 10. It is intended within each exposure class to provide options with regards to the *w/cm* and type of cementing system used. For example, in S2 exposure one option is to use *w/cm* = 0.45 together with either Type V portland cement, a blended cement with the HS designation (Types IP(HS), IS(HS) or IT(HS)) or a hydraulic cement meeting the requirements of ASTM C1157 Type HS. Alternatively, portland cement and SCM can be blended at the concrete mixer provided the blend meets the performance requirements for HS (expansion ≤ 0.05% at 6 months or 0.10% at 12 months in ASTM C1012). This is essentially what is currently recommended in ACI 201 and specified in ACI 318. A second option is to reduce the *w/cm* to 0.40 and permit the use of cementing materials that meet the requirements for moderate-sulfate (MS) resistance (as defined in ASTM C150 for portland cements, C595 for blended cements or C1157 for hydraulic cements); again there is a testing requirement using ASTM C1012 for “mixer blends”. A third option is to relax the *w/cm* to 0.50 and require the cementing system to meet the requirements for very-high sulfate (VHS) resistance; this would have to be demonstrated by testing in ASTM C1012 using more onerous expansion criteria (≤ 0.05% at 12 months or ≤ 0.10% at 18 months).

For S1 exposure the options include the requirements that are currently in ACI 201 and 318 (*w/cm* ≤ 0.50 with moderate-sulfate resistant cement (Type II or MS)) in addition to allowing a higher *w/cm* (0.55) with cementing materials that meet the performance-test requirements for highly-sulfate resistant (HS) or a lower *w/cm* (0.45) with no requirements on the cementing materials.

For S3 exposure the options are to limit the *w/cm* ≤ 0.40 and use a high-sulfate resistant (Type V or Type HS) cement or to allow a higher *w/cm* ≤ 0.45 with a very-high-sulfate (VHS) resistant cement. These options provide a compromise between the current guidelines in ACI 201 and the previous limits in ACI 318-14. This is similar to what is now required in the recent revision to ACI 318 (ACI 318-19) with the exception that in

Table 10 the performance requirements include a 12-month expansion limit (0.05%) for VHS cement in addition to the 18-month limit (0.10%) in Option 1 of ACI 318-19.

Table 9: Sulfate exposure class table

Exposure Class	Severity	Water-soluble sulfate (SO_4^{2-})* in soil, %	Sulfate (SO_4^{2-})* in water, ppm
S0	Not applicable	$SO_4^{2-} < 0.10$	$SO_4^{2-} < 150$
S1	Moderate	$0.10 \leq SO_4^{2-} < 0.20$	$150 \leq SO_4^{2-} < 1{,}500$
S2	Severe	$0.20 \leq SO_4^{2-} \leq 2.00$	$1500 \leq SO_4^{2-} \leq 10{,}000$
S3	Very severe	$SO_4^{2-} > 2.00$	$SO_4^{2-} > 10{,}000$

Table 10: Proposed durability requirements for concrete exposed to sulfates

Exposure Class	Max w/cm	Min f'_c psi	Cementing Materials (must meet either the cement designation or the performance requirements)			
			Cement Designation	Performance Requirements Expansion in ASTM C1012 (max %)		
				6 m	12 m	18 m
S0	No Requirements					
S1	0.55	3500	Performance requirement only →	0.05	0.10	
	0.50	4000	Type II or Type MS-designated blended cements	0.10	-	-
	0.45	4500	No restriction	-	-	-
S2	0.50	4000	Performance requirement only →	-	0.05	0.10
	0.45	4500	Type V or Type HS-designated blended cements	0.05	0.10	-
	0.40	5000	Type II or Type MS	0.10	-	-
S3	0.45	4500	Performance requirement only →	-	0.05	0.10

	0.40	5000	Type V or Type HS-designated blended cements	0.05	0.10	-

Notes to table:

Cement Types II and V refer to ASTM C150 cements

Type MS refers to hydraulic cements that meet the requirements for Type MS in ASTM C1157 or blended cements that meet the requirements for the MS designation in ASTM C595

Type HS refers to hydraulic cements that meet the requirements for Type HS in ASTM C1157 or blended cements that meet the requirements for the HS designation in ASTM C595

Where portland cement and SCMs are combined at the concrete mixer, it must be demonstrated that the blend meets the performance requirements listed when tested in ASTM C1012.

The type of cementing material used has to either meet the designations listed in the column "Cement Designation" or meet the required expansion criteria listed in the column "Performance Requirements" when tested in ASTM C1012.

If Type V cement is used as the sole cementitious material in an S3 exposure, the optional sulfate resistance requirement of 0.040 percent maximum expansion (when tested in ASTM C452) in ASTM C150 shall be specified.

6 ALLOWABLE CHLORIDE LIMITS

Several ACI documents report different limits on maximum allowable chlorides (C_A) in fresh concrete that leads to confusion for the practitioner. The C_A limits published by the ACI documents are mainly based on the ASTM C1218/1218M and ASTM C1152/1152M (herein C1218 and C1152) test methods. Nine ACI documents allow ASTM C1218 (water-soluble testing) only and three documents, including ACI 222R, ACI 201.2R, and ACI 212.3R, allow both test methods. Table 11 shows the variation in chloride limits established for new concrete across four ACI documents and CSA A23.1. In this table it can be seen that there is good agreement for allowable chloride limits for prestressed concrete. However, there is considerable discrepancy between the allowable chloride limits for reinforced concrete. In particular for concrete in dry or protected conditions (C0 exposure class) the range of water-soluble C_A by mass of cement is from 0.25% (ACI 201.2R-16/222R-19) to 1.00 % (ACI 318-19) [1,2,4]. It should be noted that ACI 201.2R-16 refers to ACI 222R-01. However, ACI 222 has revised the C_A limits in 2019 and ACI 201 has not revised its document since 2016. ACI 318 has a category for reinforced concrete exposed to chlorides in service whereas ACI 201.2R and 222R do not [1,2,5,4]. In addition, it should be noted that ACI 318 considers only one exposure class (C2) for chloride exposure regardless of severity of the exposure conditions. ACI 350 refers the user to a different table that specifies maximum w/cm and minimum strength for concrete exposed to chlorides in service [5]. Unlike ACI 318, ACI 222 do not provide requirements such as maximum w/cm and minimum fc' for the exposure classes. ACI 318 also has a requirement of minimum cover for reinforcement as an additional provision for C2 class. However, ACI 222R discusses the benefits of having lower w/cm ratio and higher cover depth.

The other ACI documents that give information on C_A are ACI 212.3R, ACI 221R, ACI 329R, ACI 332, ACI 349, ACI 362.1R, and ACI 506.2 [51-57]. ACI 201.2R refers to ACI 222R. ACI documents 301-10, 329R-14, 332-14, and 349-13 have the same maximum allowable chloride limits as ACI 318-19 for reinforced concrete. ACI 362.1R-12 limits allowable chlorides in reinforced concrete to 0.06% by weight of cementitious material (water-soluble chloride testing). ACI 212.3Rrefers to ACI 318. ACI 221Rrefers to ACI 201.2R or 318. ACI 349 refers to ACI 318. ACI 506.2refers to ACI 318 or 350.

ACI 318 has three exposure classes – dry conditions (C0), wet conditions (C1), and exposed to moisture and external chlorides in service (C2). ACI 201.2R and ACI 222R have only two exposure classes – wet or dry conditions. ACI 350.5 has only one C_A limit for all exposure conditions. It is very clear that there is no uniformity in number and type of exposure classes and C_A limits across ACI documents. Thus, there is a need to bring a uniform set of limits and exposure classes among various ACI documents.

C_A limits are reported as either acid-soluble chloride content or water-soluble chloride content relative to the weight of cement or total cementitious material. Determining acid-soluble chloride content is relatively straightforward, shorter, and considers all chlorides in the system. Acid-soluble chloride test may extract the majority of bound chlorides that maybe never unbound thus giving over-conservative values. Water-soluble chloride content has been used to represent the amount of free chlorides in pore solution, but, it could be higher than the amount of free chlorides in the pore solution. Trejo et al. reported that, on average, chloride concentration in pore solution is about 77% of the water-soluble chloride concentration [58]. If only water-soluble chloride limits are considered, there is a risk of ignoring corrosion due to bound chlorides that might be released at later ages due to factors such as reduction in pH owing to carbonation [59,60].

Table 11: Allowable chloride limits in various ACI documents and in the Canadian Standards Association (CSA) [1,2,5,4,61] (%bwc – percentage by weight of cement; %bwb – percentage by weight of binder or total cementitious material)

Category	**ACI 201.2R-16*/ACI 222R-19**		**ACI 318-19**	**ACI 350.5-12**	**CSA A23.1**
	Acid-soluble	Water-soluble	Water-soluble	Water-soluble	Water-soluble
Prestressed concrete	0.08 %bwb	0.06 %bwb	0.06 %bwb	0.06 %bwc	0.06 %bwb
Reinforced concrete exposed to chlorides in service	-	-	0.15 %bwb	0.10 %bwc	-
Reinforced concrete in wet conditions	0.20 %bwb	0.15 %bwb	0.30 %bwb	0.10 %bwc	0.15 %bwb
Reinforced concrete in dry or protected conditions	0.30 %bwb	0.25 %bwb	1.00 %bwb	-	1.00 %bwb

*ACI 201.2R-16 refers to ACI 222R-01. However, ACI 222 has revised the C_A limits in 2019 and ACI 201 has not revised its document since 2016

Looking at the history of these C_A limits in ACI documents revealed that these limits were first proposed in 1977 based on just three studies that were mainly funded by Federal Highway Administration [62]. It should be noted that these studies evaluated chlorides that were transported into the concrete and not admixed chlorides. These limits have been changed, over years, many times based on experience in the field and the discussions at the ACI committee meetings, but not based on well-defined scientific studies. One of the main reasons for the differences in the C_A limits could be extent of conservativeness of the particular ACI committee members or the importance of the structures that a particular document dealing with. For an example, ACI committee 222 mentions that it takes more conservative approach than most other ACI committees because of reasons such as highly variable and conflicting data on chloride threshold values, serious consequences of corrosion related damage, and difficulty in predicting the service environment throughout the life of a structure.

Table 12 shows the comparison of the chloride limits in the current ACI documents to the standards of several countries such as the Great Britain, Australia, New Zealand, and Japan. It can be seen that there is a difference in C_A limits among all these documents.

Table 12: C_A limits in various ACI Documents, Canadian Standards Associate (CSA), and other standards (%bwc – percentage by weight of cement; %bwb – percentage by weight of binder or total cementitious material)

Category	**ACI 201.2R-16*/ACI 222R-19 [2,1]**		**ACI 318-19 [4]**	**ACI 350.5-12 [5]**	**CSA A23.1 [61]**	**AS 1379-07/(NZS 3109) [63,64]**	**JSCE - 07 [65]**	**BS 8500-1: 2006 [66]**	**NS 3420-L (from ACI 222R-19) [2]**
	Acid-soluble	Water-soluble	Water-soluble	Water-soluble	Water-soluble	Acid-soluble	Acid-soluble	acid-soluble	acid-soluble
Prestressed concrete	0.08 %bwb	0.06 %bwb	0.06 %bwb	0.06 %bwc	0.06 %bwb	0.50 Kg/m3	0.08 %bwc	0.10 %bwb c	0.002 %bwc
Reinforced concrete exposed to chlorides in service (C2)	0.20 %bwb	0.15 %bwb	0.15 %bwb	0.10 %bwc	-	0.80 Kg/m3	0.30 Kg/m3a	0.30 %bwb	
Reinforced concrete in wet conditions (C1)	0.20 %bwb	0.15 %bwb	0.30 %bwb	0.10 %bwc	0.15 %bwb	0.80 Kg/m3	0.30 Kg/m3b	(0.40 %bwb)? d	
Reinforced concrete in dry or protected conditions (C0)	0.30 %bwb	0.25 %bwb	1.00 %bwb	0.10 %bwc	1.00 %bwb	0.80 Kg/m3 /(1.60 Kg/m3)	0.30 Kg/m3b	0.40 %bwb d	0.60 %bwc

*ACI 201.2R-16 refers to ACI 222R-01. However, ACI 222 has revised the C_A limits in 2019 and ACI 201 has not revised its document since 2016

a - Reinforced concrete that is used in an environment with chloride attack due to chloride ion intrusion or electrolytic corrosion and that is required to be highly durable, the chlorides quantity in the concrete should be made as small as possible in comparison with the specified value—0.30 kg/m3

b - Upper limit for the chloride ion content can be raised to 0.60 kg/m3 for structures with nonstructural reinforcement, if it is difficult to obtain low chloride content from materials

c – Prestressed by pre-tensioning; also for heat-cured reinforced concrete

d – 0.20% bwb, if sulfate resisting portland cement is used

In order to compare the limits, in Table 13, all the water-soluble C_A limits mentioned in Table 12 were converted to acid-soluble chloride limits and units were converted to either percentage by weight of cement or percentage by weight of binder. For the conversion, it was assumed that water-soluble chlorides are about 75% of the acid-soluble chlorides. Even though ACI 222R-19mentions that water-soluble chloride content is not a constant fraction of acid-soluble chloride content, it assumes that, on average, the water-soluble chlorides determined following ASTM

C1218 is 20 to 25% lower than the acid-soluble chlorides determined by ASTM C1152. This suggests chloride binding capacity of cementitious systems varies from 20 to 25%. A recent study shows that it can vary from 23% to 97% [67]. To convert kg/m^3 to %bwc, a cement content of 350 kg/m^3 is assumed. Acid-soluble C_A limits for prestressed concrete varied from 0.002 to 0.14 %bwc, with most of the documents specifying 0.08 %bwc. Acid-soluble C_A limits for C2 exposure conditions varied from 0.08 to 0.23 %bwc (or %bwb). Acid-soluble C_A limits for C1 exposure conditions varied from 0.08 to 0.4 %bwc (or %bwb). Acid-soluble C_A limits for C0 exposure conditions varied from 0.08 to 1.33 %bwc (or %bwb). Among all the documents, JSCE guidelines for concrete had the most conservative chloride limits.

Table 13: Acid-soluble C_A limits in various documents after the conversions

Category	ACI 201.2R-16*/ACI 222R-19 [2,1]	ACI 318-19 [4]	ACI 350.5-12 [5]	CSA A23.1 [61]	AS 1379-07/(NZS 3109) [63,64]	JSCE -07 [65]	BS 8500-1: 2006 [66]	NS 3420-L (from ACI 222R-19) [2]
Prestressed concrete	0.08 %bwc	0.08 %bwc[e]	0.08 %bwc[e]	0.08 %bwb[e]	0.14 %bwc[f]	0.08 %bwc	0.10 %bwb [c]	0.002 %bwc
Reinforced concrete exposed to chlorides in service (C2)	-	0.20 %bwc[e]	0.13 %bwc[e]	-	0.23 %bwc[f]	0.08 %bwc[a,f]	0.30 %bwb	
Reinforced concrete in wet conditions (C1)	0.10 %bwc	0.40 %bwc[e]	0.13 %bwc[e]	0.20 %bwb[e]	0.23 %bwc[f]	0.08 %bwc[b,f]	(0.40 %bwb)? [d]	
Reinforced concrete in dry or protected conditions (C0)	0.20 %bwc	1.33 %bwc[e]	0.13 %bwc[e]	1.33 %bwb[e]	0.23 %bwc[f] /(0.46 %bwc[f])	0.08 %bwc[b,f]	0.40 %bwb [d]	0.60 %bwc

*ACI 201.2R-16 refers to ACI 222R-01. However, ACI 222 has revised the C_A limits in 2019 and ACI 201 has not revised its document since 2016

a - Reinforced concrete that is used in an environment with chloride attack due to chloride ion intrusion or electrolytic corrosion and that is required to be highly durable, the chlorides quantity in the concrete should be made as small as possible in comparison with the specified value—0.30 kg/m3 (~0.08%bwc[f])

b - Upper limit for the chloride ion content can be raised to 0.60 kg/m3 (~0.16%bwc[f]) for structures with nonstructural reinforcement, if it is difficult to obtain low chloride content from materials

c – Prestressed by pre-tensioning; also for heat-cured reinforced concrete

d – 0.20% bwb, if sulfate resisting portland cement (conforming to BS 4027) is used

e- If water-soluble chlorides = 0.75 x acid-soluble chlorides

f – If cement content = 350 kg/m3 (0.8 kg/m3 ~ 0.23 %bwc)

Trejo and Weyers did a review on history of the C_A limits reported in ACI documents [62]. They also proposed standardized C_A limits that are classified according to exposure classes and structure importance (show in Table 14) based on their study. Structural importance was categorized at three levels – low, moderate or high. The study was not focused on defining the structure importance levels, but suggests that C_A limits should be dependent on the inherent risk to human life if a structure was to fail due to corrosion of the reinforcing steel. Examples of importance of structure were given as follows: high – higher hazard or risk to human life if the structure fails; moderate – minimal hazard to human life if the structure fails; low – low hazard to human life if the structure fails.

Table 14: C_A limits recommended by Trejo and Weyers [62]

Exposure class	Structure Importance	Max. acid-soluble chloride content in concrete, % by weight of cement	
		Reinforced concrete	Prestressed concrete
C0 - Concrete that will be dry and protected from moisture	Low	0.40	0.06
	Moderate	0.30	0.06
	High	0.20	0.06
C1 - Concrete exposed to moisture but not to external sources of chlorides	Low	0.15	0.06
	Moderate	0.10	0.06
	High	0.10	0.06
C2 - Concrete exposed to moisture and an external source of chlorides	Low	0.10	0.06
	Moderate	0.10	0.06
	High	0.08	0.06

Later, Trejo et al. did a review of how ACI documents define maximum chloride limits, and they highlighted inconsistencies in them [68]. They also highlighted the need of a consistent chloride limits across ACI documents.

The aforementioned review clearly states that there is a need of consensus among different ACI documents. However, to do that a better understanding of critical chloride threshold is needed, which is discussed in the following sub-section.

6.1 Critical chloride threshold

The risk of corrosion increases as the chloride content increases. When the chloride content exceeds the critical chloride threshold, corrosion can occur if oxygen and moisture exist to support the corrosion reactions. As the C_A must be significantly lower than C_T to prevent reinforcement corrosion, agreement on critical chloride threshold values is needed before uniform guidance can be given for allowable chloride limits.

A study done by Angst et al. in 2009 showed that there is a significant scatter (0.04 – 8.34% by weight of binder) of chloride threshold values reported in the literature [69]. In this study, most of the published critical chloride contents (C_T) until 2009 were reported. The data includes published C_T values under outdoor exposure conditions or from real structures, obtained from experiments with the steel directly immersed in solution and steel embedded in cement-based material (laboratory conditions). This large degree of variability is due to its dependency on several factors such as steel-concrete interface, electrochemical potential of the reinforcement steel, pore solution chemistry, etc. Other factors for this huge scatter are lack of an accepted or standardized test method to determine chloride threshold value (although many studies in the literature determined it in various ways) and clear definition for it [69].

Figure 9 illustrates the different factors influencing the determination of critical chloride threshold value

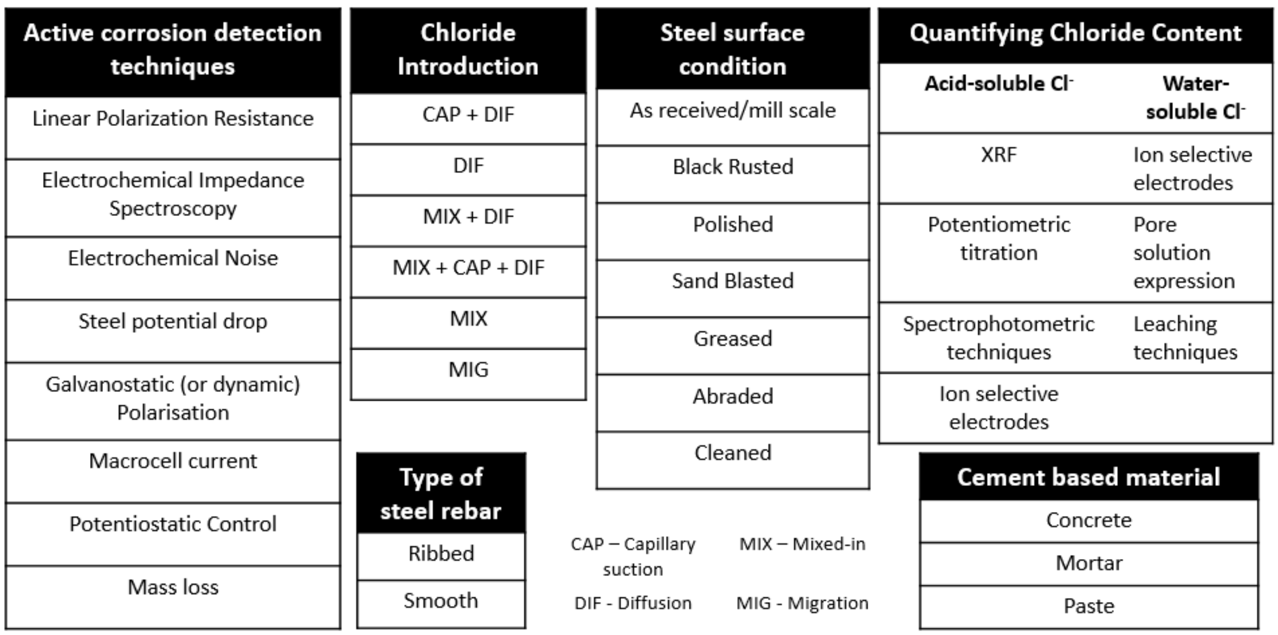

Figure 9: Different factors influencing C_T (CAP- capillary suction, DIF – diffusion, MIX – mixed-in)

Alonso and Sanchez analyzed the variability of C_T values reported in the literature available till 2009 [70]. For the analysis, data was collected from around 50 laboratory studies and 11 field studies. The data from laboratory studies include C_T determined from non-accelerated test methods (ponding or wet/dry cycling) and accelerated test methods (potentiostatic and migration). A wide variation of the values were observed, ranging from 0.3 to 4.0% bwb acid-soluble threshold chlorides values and 0.1 to 2.5% bwb water-soluble chloride threshold values. A large variation was observed in the data from non-accelerated tests and lower variation was observed in the data from migration tests. It was also observed that migration studies have the least C_T values. The authors analyzed the data from non-accelerated tests to see the effect of the way of introducing chlorides (chloride addition during mixing or later penetration of chlorides). A very similar range of distribution of values was observed for both the ways of introducing chlorides. The authors also reported that lower C_T values in field conditions than in the laboratory was observed [70].

As a part of this project, data on chloride threshold values were collected from various journals, conference proceedings, special publications, and research reports. Data from the tests that studied C_T by immersing steel in simulated pore solutions or alkaline solutions are not included in this study. It was observed that published acid-soluble critical chloride contents ranged from

0.02% to 8.34% bwb, water-soluble C_T values ranged from 0.07% to 4.00% bwb and from 0.045 to 3.22 mol/L. This data is widely scattered and ranges across two orders of magnitude.

The data were analyzed in order to reduce the scatter by excluding the data that were not close to the field conditions and sorting the data by different cementitious systems and exposure conditions. The studies that used paste samples, tests that used migration as the way of introducing chlorides, and the studies that used samples with more than 3% bwb admixed chlorides are excluded from the data as they do not represent field conditions. As C_T depends on whether chlorides are present in the mixture constituents or penetrates the hardened concrete from external sources, the data was sorted based on exposed categories as shown in Figure 10. Examples of ways of introducing chlorides are by ponding (CAP + DIF: capillary suction and diffusion), immersion in a salt solution (DIF: diffusion), mixed-in chlorides (MIX), and combination of mixed-in chloride and ponding or mixed in chlorides and immersion in salt solutions.

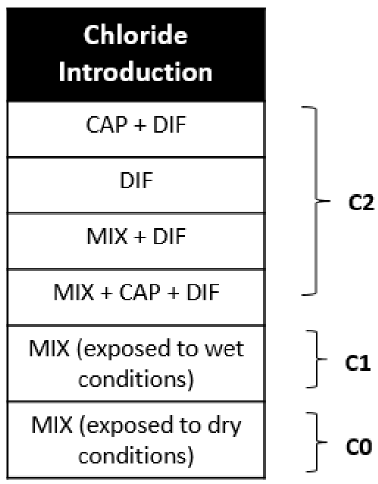

Figure 10: Classifying different types of chloride introduction to existing exposure classes (CAP- capillary suction, DIF – diffusion, MIX – mixed-in)

This method of analyzing the data reduced the scatter considerably as shown in Table 15 and Table 16. However, it is still large enough making it difficult to recommend a range of chloride threshold values.

Table 15: Range of C_T values for systems with portland cements (chlorides added during mixing)

	C1	C2
Acid-soluble chlorides (%bwb)	0.10-3.08	0.50-1.00
Water-soluble chlorides (%bwb)	0.11-1.16	-
Cl^-/OH^-	1.17-3.98	12.00-35.00

Table 16: Range of C_T values for systems with portland cements (chlorides penetration)

	C1	C2
Acid-soluble chlorides (%bwb)	-	0.20-2.15
Water-soluble chlorides (%bwb)	-	0.40-0.80
Water-soluble chlorides (mol/l)	-	0.04-1.83
Cl^-/OH^-	-	0.60-45.00

6.1.1 Presence of SCMs

Table 17 and

Table 18 summarize the range of measured C_T values (for blended cements) by the chloride addition during mixing and by the later penetration of chlorides respectively.

Table 17: Range of C_T values for systems with blended cements (chlorides added during mixing)

	C_T
Acid-soluble chlorides (%bwb)	0.25-1.80
Water-soluble chlorides (%bwb)	0.07-0.13
Cl^-/OH^-	0.19-0.27

Table 18: Range of C_T values for systems with blended cements (chlorides penetration)

	C_T
Acid-soluble chlorides (%bwb)	0.044-2.500
Water-soluble chlorides (%bwb)	0.283-0.746
Cl^-/OH^-	1.5-20.0

It is not very clear from the data analysis whether C_T increases or decreases by addition of SCMs to concrete due to the limited information available. Trejo and Tibbits observed that C_T decreases by addition of fly ash and slag to mortar mixtures of 0.45 w/cm [71]. Bouteiller et al. indicated that C_T might decrease with addition of SCMs to concrete mixtures [72]. They observed that corrosion initiation is a highly random phenomenon that also depends on time not just on chloride content. They concluded that proposing a range of critical chloride contents would be more realistic rather than proposing just a critical chloride value [72]. Presuel-Moreno and Moreno observed that C_T increases with low level replacements of portland cement with silica fume but C_T decreases at high level replacements of portland cement with silica fume [73]. Azad and Isgor showed that at higher SCM replacement levels Cl^-/OH^- increased but not affected up to a certain SCM replacement level, which could affect the C_T [74].

In the current study, lack of sufficient comparable data and inconsistencies in reporting of the variability of C_T values in the literature made it challenging to analyze the data statistically. It is observed that not many studies are done on C_T of concrete or mortar with water to cement ratio less than 0.40 and concrete with SCMs. More research is needed to understand the influence of SCMs on C_T. Another reason for this variability in C_T is lack of a standard method to determine C_T. Therefore, recommendations on allowable chloride limits using C_T data reported in the literature could not be made here. However, this section emphasizes the need of developing a commonly accepted reliable test method to determine critical chloride threshold value which in turn helps us to determine maximum allowable chloride limits in new concrete. The following section details the current efforts to develop commonly accepted test method.

6.2 Current research efforts

The status on the current research work across the world related to studying the parameters that influence chloride induced corrosion initiation, developing a test method to determine critical chloride threshold value, revising the exposure classes in ACI documents, and revising allowable chloride limits are discussed as follows:

- RILEM TC235-CTC (Corrosion initiating chloride threshold content in concrete): One of the main objectives of this technical committee was to give recommendations on test methods to determine C_T. A test method was proposed and a round-robin test was organized to evaluate it. However, the technical committee was not able to achieve its objective. Regardless of that, their research experiences in developing the test method is useful and recommendations were made for future research and its published by Tang et al. in RILEM technical letters [75,76].
- RILEM TC262-SCI (Characteristics of the steel-concrete interface and their effect on initiation of chloride-induced reinforcement corrosion): One of the four aims and objectives of this committee is to summarize the state of the art concerning different conditions at the steel-concrete interface and their possible effect on chloride-induced corrosion initiation. The other objectives are to find the knowledge gaps in research to improve understanding the conditions leading to chloride induced corrosion initiation, summarize existing methods to characterize the steel-concrete interface, and make recommendations on methods to characterize the steel-concrete interface conditions [77].
- ACI 222 TG1 – Developing standardized test methods to determine C_T: This task group has been recently formed by ACI 222 committee. The objective of this group is to develop a test protocol to quantify critical chloride thresholds for corrosion of reinforcing steel in cementitious systems. Currently, two different test methods are being investigated. One of the methods has a test set-up that mimics a macro corrosion cell that was developed by Dr. David Trejo and his research group at Oregon State University. The other test method is based on the work performed by Berke et al. as a part of ASTM

subcommittee G1.14, Corrosion of Metals in Cement, Mortar or Concrete [78]. A round robin test is currently undergoing (as of March 2020) and the experimental program for this testing is expected to conclude by the end of Summer 2020.

- Angst et al. developed an experimental protocol to determine the chloride threshold value for corrosion in samples taken from reinforced concrete structures [79]. The authors proposed an experimental protocol to determine C_T for individual structures. One of the main advantages of the proposed method is that C_T is determined under real conditions rather than from laboratory-produced samples. This method yields the statistical distribution of C_T that can be used to predict the time to corrosion initiation for structures in the field [79].
- NRMCA: Evaluation of chloride limits for reinforced concrete. This work is divided into two phases. The objective of the first phase is to establish a relationship between calculated total chlorides from the concrete ingredients and water-soluble chlorides measured on concrete specimens at an age between 28 and 42 days. The objectives of the second phase are to propose chloride limits and evaluate the validity of current ACI 318 chloride limits for reinforced concrete. At the time of writing this report, the second phase results are not available [80].
- It should be noted that the current ACI documents do not consider a different exposure class for reinforcement corrosion due to carbonation. In addition, it should be noted that ACI 318 does not clearly states if the corrosion due to wind-borne salt exposure is included in C2 exposure class. ACI 201 committee has recently started a discussion on expanding or revising the current exposure classes.
- Hooton recently discussed the limitations of current durability-based codes and standards and issues that need to be addressed in development of future codes and standards for durability in design and construction [81]. To address some of the current deficiencies, ACI 201 has recently started developing mandatory code and specifications for durability in design and construction.

6.3 Proposed recommendations

From the literature review concluded in the current study, it is understood that substantial research needs to be undertaken to determine more well-informed C_A limits in concrete. In the interim, chloride limits and exposure classes among the ACI documents can be unified to avoid further confusion to the practitioner and to bring clarity among ACI documents. Based on the analysis of the collected data and review of the current research work related to this topic, the following recommendations are made on how to bring unified specifications among the ACI documents.

The following are some of the recommendations adopted from the research work by Dr. David Trejo and his research group to provide further support for defining consistent C_A limits across ACI documents:

- ACI documents need to provide more information on determination of chloride contents of concrete having aggregates with bound chlorides [68]

- All ACI documents need to refer to the same chloride tests [68,82] and the recent research has shown recommending only water-soluble chloride test [83]
- If water-soluble chloride limits are provided by the documents, they need to specify that bound chlorides might be released and available as free chlorides during the structure life and this might lead to corrosion earlier than expected [68]
- ACI committees that provide chloride limits but not mentioning any exposure conditions should include exposure classes similar to ones included in ACI 318 [68]
- C_A limits could be classified according to structure importance/risk and exposure conditions. Defining importance/risk levels can be done by discussions at ACI conventions among ACI committees [68]

It should be noted that this report's scope is limited to provide recommendations on unifying C_A limits and exposure classes among the current ACI documents. This report does not discuss the additional requirements including maximum water to cementitious material ratio, minimum f_c', and minimum cover depths, or any performance requirements. The structure importance is divided into three categories – T1, T2, and T3. The possible definitions of them are given in Table 19. It should be noted that the only intention of Table 19 is to initiate further discussions on defining structure importance or risk levels. The proposed framework for unified water-soluble C_A limits and exposure classes from this study are shown in Table 20.

Table 19: Proposed structure importance/type classification

Structure importance/type category	Definition[a]	Examples[b]
T1	Minor risk of reinforcement corrosion can be tolerated	Non-load-bearing elements, highway barriers, retaining walls, culverts, low-volume rural highway pavements, slab-on-grade
T2	Reinforcement corrosion cannot be tolerated; Non-environmental structures	Bridges, multi-story buildings, tunnels, critical elements that are difficult to inspect or repair
T3	Reinforcement corrosion cannot be tolerated; Environmental structures	Environmental engineering concrete structures, water or wastewater treatment facilities

a – These could be the possible definitions. The only intention of defining the structure importance here is to initiate further discussions on this.
b – These are only meant to serve as examples. The only intention of giving examples here is to initiate further discussions on this.

Table 20: Proposed framework for unified water-soluble C_A limits

Exposure class	Structure importance /type	Chloride limits for new construction (% by mass of cementitious material[a]) — Water-soluble (ASTM C1218/C1218M): Reinforced concrete*	Chloride limits for new construction (% by mass of cementitious material[a]) — Water-soluble (ASTM C1218/C1218M): Prestressed concrete
Note: The purpose of this proposed framework is to unify the current C_A limits across the ACI documents before statistical data from a standardized test method to determine the critical chloride threshold are available			
C0 - Concrete in a dry[b] environment	T1	1.00 (ACI 318-19)	0.06
	T2	0.25 (ACI 222R-19)	0.06
C1 - Concrete exposed to moisture but not to external sources of chlorides in service	T1	0.30 (ACI 318-19)	0.06
	T2	0.15 (ACI 222R-19)	0.06
C2 - Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater, or spray from these sources in service[c]	T1, T2	0.15 (ACI 222R-19, ACI 318-19)	0.06
C3 – Concrete exposed to severe exposure conditions include concentrated chemicals, wetting and drying cycles, freeze and thaw cycles	T2, T3	0.10 (ACI 350-06)	0.06

*Reinforced concrete with conventional black steel (ASTM A615/A615M and A706/A706M)
a – Total cementitious material includes portland cement and SCM; however, the SCM content cannot exceed the portland cement content
b – A dry environment corresponds to a maximum relative humidity of 60%, normally found in the interior of buildings
c - Reinforced concrete that is used in an environment with chloride attack due to chloride ion intrusion or electrolytic corrosion and that is required to be highly durable, the chlorides quantity

in the concrete should be made as small as possible in comparison with the specified value in the table

7 CONCLUSION

In this research project several concrete durability issues were investigated including chemical sulfate attack, allowable chloride limits in fresh concrete and freezing-and-thawing of concrete. The research presented in this report has produced several important outcomes including: 1) Unified durability requirements and exposure class descriptions for ACI committees 201, 222, 301, 318 and 350; 2) Identification of knowledge gaps and future studies to address deficiencies in existing data sets and to develop relationships between concrete field performance and laboratory testing; and 3) Detailed need for improved test methods to further validate the reliability of specified mixture proportions and material parameters for durability. The results of this study can be used as a starting point for ACI Committee work that will ultimately provide unified guidance across ACI documents to provide clarity to the end user on durability requirements and exposure class descriptions.

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