Materials-Related Distress of PCC
Introduction

• Concrete pavements are inherently durable, having a history of exceptional long-term performance

• In some instances, pavement service life has been adversely affected by the concrete’s inability to maintain its integrity in the environment in which it was placed

• These distress manifestations are categorized as materials-related distress (MRD)
What is Materials-Related Distress?

• MRD is commonly associated with the “durability” of the concrete

• Durability is not an intrinsic material property
  
  ➢ Concrete that is durable in one application may rapidly deteriorate if placed in another application

  ➢ “Durability” can not be measured directly. Potential durability can be inferred

• It is not related to loading, although loading can exacerbate the distress
MRD Manifestations

• Fine pattern cracking
  ➢ Isolated to joints or over entire surface
  ➢ Progressive in nature, getting worse over time

• Degradation such as spalling or scaling

• Often accompanied by staining and/or exudate

• Evidence of expansion is also common with some types of MRD
Are These MRDs?
And This?
What About This One?
Important Considerations

• The concrete constituents, proportions, and construction all influence MRD

• Water is needed for deleterious expansion to occur

• Severe environments (e.g. freezing and thawing, deicer applications, high sulfate soils, etc.) are major contributors

• Strength does not equal durability
Materials Basics - Hardened Cement Paste (HCP)

- Combination of cement, supplementary cementitious materials (SCMs), water, and admixtures to form hydration products and entrained air

  - HCP provides cohesion to the concrete mixture

  - Comprised (primarily) of calcium silicate hydrate (CSH) and calcium hydroxide (CH)

  - CSH - provides strength and is desirable

  - CH - provides little strength and plays a role in many MRD mechanisms

    - CH is soluble in water; solubility maximum at 0 °C (32 °F)

    - Pozzolans (e.g., fly ash) consume CH and yield CSH
Materials Basics - Aggregates

• Natural gravel, quarried rock, and sand, or manufactured mineral filler (e.g., air-cooled blast furnace slag and manufactured sand)
  
  ➢ 65 – 80 % volume of the concrete
  
  ➢ Less expensive than components of the HCP
  
  ➢ Provide the load-carrying capacity of concrete
  
  ➢ Provide dimensional stability
  
  ➢ Generally considered inert but not always the case
  
  ➢ Aggregate quality significantly impacts concrete quality
  
  ➢ Local sources used (shipping considerations) so poor quality is often accepted but needs to be mitigated
Mechanisms of MRD Common in Pavements

• Physical Mechanisms
  ➢ Freeze-thaw deterioration of hardened cement paste
  ➢ Deicer scaling/deterioration
  ➢ Freeze-thaw deterioration of aggregate

• Chemical Mechanisms
  ➢ Alkali–aggregate reactivity
    ➢ Alkali–silica and alkali–carbonate reactivity
  ➢ Sulfate attack
    ➢ External and internal sulfate attack
  ➢ Corrosion of embedded steel
Freeze-Thaw Deterioration of Hardened Cement Paste

• Overview

- Deterioration of saturated HCP due to repeated freeze-thaw cycles
- Manifests as scaling, spalling, or map cracking
- Typically appears within 1 to 5 years after construction
- Prevented through the use of air entraining admixtures which create a protective air-void system
Freeze-Thaw Deterioration of Hardened Cement Paste

• Mechanism

- Results when the HCP becomes “critically saturated” and concrete undergoes F-T cycles

- The expansion of ice causes tensile forces that crack concrete

After CEB 1957
Freeze-Thaw Deterioration of Hardened Cement Paste

- **Prevention**
  - Entrained air-void system
    - Delays the time to critical saturation
    - Provides space for ice formation
    - Allows for water movement (minimizing hydraulic pressure)
  - Volume and spacing of voids critical

**Diagram:**
- Partially Saturated Capillary and Gel Pores
- Entrained Air
- Critically Saturated Capillary and Gel Pores

*When d exceeds some critical distance, water in the capillary and gel pores cannot travel to an air void and freeze, resulting in hydraulic pressure and cracking*
Air-Void System
Air-Void System

• Testing

- ASTM C231 (pressure meter), ASTM C173 (roller meter), and ASTM C138 (unit weight bucket) all measure **ONLY** the air content (A)

- Air content is the volumetric air content, typically 4 – 8 % volume by specification

- For QC purposes it is sufficient to measure the volume of air

- To fully establish F-T durability other air-void system (such as spacing factor) parameters are more important
Air-Void System

- F-T durability requires adequate air void system

  ➢ Spacing Factor ($\bar{L}$):
    - Approximates the constant distance from the surface of each air void surface, which would encompass some large fraction of the paste\(^\dagger\).
    - Maximum: less than 0.2 mm (0.008 in.) (ACI 201)

  ➢ Specific Surface ($\alpha$):
    - Surface-to-Volume Ratio
    - Defines the surface area associated with a unit volume of void space – smaller voids (i.e., more voids per unit volume) result in a higher specific surface
    - Minimum: 24 mm\(^2\)/mm\(^3\) (600 in.\(^2\)/in.\(^3\))

  ➢ Air Content ($A$)
    - Measured on hardened & fresh concrete
    - $L$ and $\alpha$ assumed proportional to $A$ in fresh concrete

\[ r_{an} = \text{radius of air void } n \]
\[ r_{pn} = \text{radius of FT protection} \]

Air-Void System

• Testing

➢ To measure spacing factor, specific surface, and other air-void system parameters, it is necessary to analyze hardened concrete using ASTM C457

➢ Requires special equipment and skilled operator (petrographer)

➢ Because only hardened concrete can be analyzed, can only be used as a QA tool

➢ AASHTO TP 118 (SAM meter) is an emerging technique that measures a combined parameter (SAM number) on fresh concrete that correlates to F-T performance in a manner similar to ASTM C457
Air-Void System

• Testing

➢ Measurement of F-T performance of a concrete mixture is measured using ASTM C666 *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing*

➢ Measures the loss in dynamic modulus after undergoing up to 300 cycles of freezing (either in air or water) and thawing (in water)

➢ Test is severe and does not always correlate with field performance, but serves as a mixture screening test

➢ Performed as part of mixture design, not as part of a QC/QA program (typically)
Deicer Scaling
Deicer Scaling/Deterioration

• Overview

- Deicer chemicals can amplify paste freeze-thaw deterioration and may also chemically react with or degrade hydrated paste constituents

- Manifests as scaling, spalling, or map cracking with possible staining near joints

- Typically appears within 1 to 5 years after construction

- Prevented through the use of air entraining admixtures and a relatively low water-to-cement ratio

- Minimize finishing, which can reduce air content at surface and can increase w/cm ratio (resulting in weaker, more porous hardened paste).
Deicer Scaling/Deterioration

• Mechanism

- Not well understood
- Current research indicates scaling is due to tensile forces developed in the surface layer of concrete due to expansion of the ice layer†
- The expansive forces of the ice are at a maximum when the solution freezing on the surface contains ~3% dissolved salt, and the type of salt is not a factor†
- Often seen when the concrete is over finished, working the air out of the surface layer

Impact of Poor Finishing

0.5 to 1 inch
Deicer Scaling/Deterioration

• Prevention

➤ Do not over finish

➤ Good air-void system to begin with

➤ Avoid salts – especially at early ages (i.e., one year or less)

➤ Proper curing

➤ Penetrating sealers to reduce water ingress
Deicer Scaling/Deterioration

• Testing

Scaling performance of a concrete mixture is measured using ASTM C672 *Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals*

- Assesses (by visual inspection only) the loss of material from the concrete surface after ponding dilute salt solution on the surface and exposing the samples to 50 cycles of F-T

- Test is widely criticized for its subjectivity; modifications include measuring the mass loss from the sample

- Test very susceptible to how specimens are finished
F-T Deterioration of Aggregate (D-cracking)

• Overview

- Caused by the fracturing or dilation of coarse aggregate under freeze-thaw cycling in a saturated state
- Manifests as cracking and staining parallel to joints and cracks that may eventually spall
- Commonly manifests within 10 to 15 years
- Prevented through the use of non-susceptible aggregates or reduction in the top size of the coarse aggregate
Aggregate Freeze-Thaw Deterioration
F-T Deterioration of Aggregate

• Mechanism

➢ Aggregates are porous and absorb water

➢ The rate of water absorption and the rate of water expulsion is a function of the pore size distribution of the aggregates

➢ D-cracking aggregates have a pore size distribution that does not allow for rapid expulsion of water as a freezing front moves through an aggregate

➢ The resulting hydrostatic pressure forces can exceed the tensile strength of the aggregate
F-T Deterioration of Aggregate

• Prevention

  ➢ Avoid the use of susceptible aggregates

  ➢ D-cracking aggregates are best identified by experience

  ➢ When it is necessary to use susceptible aggregates, limit the aggregate top size

  ➢ A smaller aggregate particle diameter allows for more rapid expulsion of absorbed water
F-T Deterioration of Aggregate

• Testing

➢ Iowa Pore Index Test (IPIT)

➢ Measures the amount of water absorbed by aggregates under pressure (241 kPa, 35 psi) in a prescribed time
  – Water absorbed 0-1 minutes macropores (Primary Load)
  – Water absorbed 1-15 minutes supermicropores (Secondary Load)
  – Secondary Load > 27 mL is associated with D-racking susceptible aggregates

➢ IPIT results are more representative of the parent rock because of the large sample volume used (9000 g)

➢ Variable/erroneous results for aggregates with rapid rates of early absorption

➢ No discernible trends in the results from gravels
F-T Deterioration of Aggregate

- Testing
  - Hydraulic Fracture Test (Washington, Indiana)
    - Measures the amount of particle fracture that occurs after pressurizing aggregate (7930 kPa, 1150 psi) submerged in water
      - Start with n particles retained on a 12.5 mm (1/2 inch) sieve. After 10 cycles of pressure and release, report the increase in the number of particles retained on a #4 sieve as a percentage of n (percent fracture)
      - From these measurements determine the number of pressurization cycles required to create a percent fracture of 10% - low values indicate more D-cracking susceptibility
    - Can identify aggregates likely to fail due to hydraulic pressures
    - Does not simulate the confining effect of the HCP
Alkali–Silica Reactivity
Alkali–Silica Reactivity

• Overview

- Caused by a reaction between alkali hydroxides in the paste pore solution and reactive silica in aggregate that forms expansive reaction product

- Manifests as map cracking over the entire slab area. Exudate is common, as is expansion related distress

- Commonly occurs within 5 to 15 years

- Prevented through the use of non-susceptible aggregates, limiting total mixture alkalinity, and addition of pozzolans
Alkali–Silica Reactivity

• Mechanism

- In the presence of pore solution (i.e., H₂O, Na⁺, K⁺, Ca²⁺, OH⁻, and H₃SiO₄⁻ ions), reactive silica undergoes depolymerization, dissolution, and swelling.

- Depends on pH of the solution, not on alkalis per se, although they control the pH.

- The higher the pH, the more soluble the silica.

- Attack is more aggressive when the silica is not crystalline (i.e., amorphous or crypto-crystalline) as in cherts or opaline shales.
Alkali–Silica Reactivity


Solubility Curve of Amorphous Silica
(Tang & Su-fen, 1980)
Alkali–Silica Reactivity

• Mechanism

- Silicon and oxygen prefer to combine in “tetrahedral coordination”
- Four (4) oxygens surround each silicon – open surface structure

† http://www.quartzpage.de/gen_struct.html
Alkali–Silica Reactivity

- Charged surface attracts ions due to unsatisfied bonds
Alkali–Silica Reactivity

- Charged surface attracts $H^+$ and $OH^-$ ions from alkali pore water
Alkali–Silica Reactivity

• With alkalis in pore water, Na⁺ substitutes for H⁺
Alkali–Silica Reactivity

• If any excess alkalis still exist, the associated hydroxyl (OH⁻) breaks the Si-O-Si bond and free an oxygen
Alkali–Silica Reactivity

- Hydrogen dissociates from OH\(^-\), combines with O\(^{2-}\) to form water. Na\(^+\) substitutes for H\(^+\) and the attack has progressed.
Alkali–Silica Reactivity

- \( \text{Si-O-Si} + 2\text{OH}^- + 2\text{Na}^+ \rightarrow 2(\text{Si-O-Na}) + \text{H}_2\text{O} \)
Alkali–Silica Reactivity

• Prevention
  ➢ Avoid use of reactive aggregates
  ➢ Limit alkalis in the concrete mixture

• Mitigation
  ➢ Use of SCMs
    ➢ Fly Ash
    ➢ Slag Cement
  ➢ Use of Lithium
**Alkali–Silica Reactivity**

- **Guidance**
  - ASTM C1778 - *Standard Guide for Reducing the Risk of Deleterious Alkali-Aggregate Reaction in Concrete*
  
  - AASHTO PP-65 - *Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction*

  - Performance Approach
    - Demonstrate performance (ASTM C1260 and ASTM C1293)

  - Prescriptive Approach
    - Limit the alkali content of the concrete mixture
    - Mitigate with SCMs
Alkali–Silica Reactivity

• Testing

➢ Various Screening Methods

➢ ASTM C295 - *Standard Guide for Petrographic Examination of Aggregates for Concrete*


➢ ASTM C1293 - *Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction*
Alkali–Silica Reactivity

• Testing

➢ ASTM C295

➢ Outlines procedures for the petrographic examination of materials proposed for use as aggregates in cementitious mixtures or as raw materials for use in production of aggregates

➢ Can identify, and approximate the quantity of, potentially ASR reactive aggregates

➢ Useful way to screen potential sources

➢ If an aggregate is identified as potentially reactive by petrography, expansion testing is still required to identify if the aggregate exhibits deleterious ASR or to identify mitigation strategies
Alkali–Silica Reactivity

• Testing

➢ ASTM C1260

➢ Used to detect the potential for deleterious alkali-silica reaction of aggregate in mortar bars

➢ Samples are exposed to NaOH solution for 14 days (cement alkali content not a factor) and expansion measured

➢ Accepted expansion limit (0.1 at 14 days) chosen empirically to correlate with an ASTM C1293 expansion of 0.04

➢ Performing the test with modifications (e.g., exposure time other than 14 days, expansion limit other than 0.1 - or any other changes – undermines the validity of the test
Alkali–Silica Reactivity

• Testing

➢ ASTM C1567

➢ Used to detect the potential for deleterious alkali-silica reaction of combinations of cementitious materials and aggregate in mortar bars

➢ Samples are exposed to NaOH solution for 14 days and expansion measured

➢ Accepted expansion limit (0.1 at 14 days) chosen empirically to correlate with an ASTM C1293 expansion of 0.04

➢ Mitigation of expansion can be evaluated by the test, but the actual SCM replacement level required must be confirmed by ASTM C1293 or field experience
Alkali–Silica Reactivity

• Testing

  ➢ ASTM C1293

    ➢ Estimates the susceptibility of an aggregate, or combination of an aggregate with pozzolan or slag, for participation in expansive ASR by measurement of length change of concrete prisms

    ➢ Samples are spiked with alkali to create any possible ASR and expansion is monitored for 1 – 2 years, depending on the test intent - accepted expansion limit (0.04 at 1 year)

    ➢ Pozzolan replacement levels established by ASTM C1293 should be treated as the minimum required

    ➢ The “gold standard” of ASR testing – not without incorrect results but the best test available

    ➢ Concerns with the test include alkali leaching from the specimens over time
Alkali-Carbonate Reactivity

• Overview

➢ Caused by a reaction between alkalis in the paste pore solution and *specific* carbonate/dolomitic aggregates that undergo dedolomitization and brucite formation

➢ Manifests as map cracking over entire slab area with accompanying expansion related distress

➢ Commonly manifests in 5 to 15 years

➢ Only sure prevention is to avoid susceptible aggregates, although blending and sizing aggregates and/or significantly reducing total alkalinity may be effective

➢ Mitigation is not possible
Alkali-Carbonate Reactivity

• Mechanism

Dedolomitization

\[ \text{CaMg(CO}_3\text{)}_2 + 2(\text{Na,K})\text{OH} \rightarrow \text{Mg(OH)}_2 + \text{CaCO}_3 + (\text{Na,K})\text{CO}_3 \]

Dolomite          Alkali Hydroxide          Brucite          Calcite          Alkali Carbonate

Recycle Alkalis – leads to carbonate halos

\[ \text{Na(K)}_2\text{CO}_3 + \text{Ca(OH)}_2 \rightarrow \text{CaCO}_3 + 2\text{Na(K)}\text{OH} \]

Alkali Carbonate          CH          Calcite          Alkali Hydroxide

Recycling of alkalis makes mitigation ineffective
Alkali-Carbonate Reactivity

• Mechanism – Possible Reasons For Expansion
  
  ➢ Swelling of clay minerals in dolomite
    ➢ Dedolomitization increases permeability, more water ingress, possible swelling
  
  ➢ Crystal growth in dolomite matrix
    ➢ Brucite formation
  
  ➢ Increased alkalis leading to ASR
    ➢ Microcrystalline silica is present in many aggregates reported to be ACR susceptible

  ➢ **IMPORTANT**: Requires a specific dolomite rock type – Only a very slight fraction of dolomite rocks are ACR reactive – specific geologic deposits/locations (i.e., argillaceous dolomitic limestone, fine grained matrix with dolomite crystals embedded)

• Prevention – Avoid ACR susceptible aggregates
Alkali-Carbonate Reactivity

- Testing – Reject aggregates based on chemistry
Alkali-Carbonate Reactivity

• Testing

➢ ASTM C586 Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks as Concrete Aggregates (Rock-Cylinder Method)

➢ Used to screen rock materials for potential ACR reactivity

➢ ASTM C1105 Standard Test Method for Length Change of Concrete Due to Alkali-Carbonate Rock Reaction

➢ Determines the susceptibility of cement-aggregate combinations to expansive alkali-carbonate reaction for certain calcitic dolomites and dolomitic limestones

➢ Aggregates passing ASTM C1105 should be evaluated for ASR potential using ASTM C1260 or C1293, as appropriate
External Sulfate Attack

• Overview

➢ Expansive formation of sulfate minerals resulting from an external source of sulfate ions

➢ Fine cracking and deterioration near joints and slab edges and on underside of slab

➢ Usually manifests within 1 to 5 years

➢ Mitigated through the use of low w/c, minimizing aluminate phases in cementitious materials, or using pozzolans or ground slag
External Sulfate Attack

• Mechanism

➤ Sulfate-laden solution enters concrete

➤ Sulfate reacts with alumina and calcia to form ettringite

➤ Monosulfate – normally present in HCP is converted to ettringite

➤ Ettringite can form in HCP or at the HCP-aggregate interface

➤ Conversion to ettringite results in volume increase – expansion – and cracking

➤ Ettringite that forms in air voids does not exert expansive pressures but may compromise the air-void system
External Sulfate Attack

• Mechanism – Example Reactions

Symbols Used

\( \bar{S} = \) sulfate ion \((SO_4^{2-})\)
\( CH = \) calcium hydroxide \((Ca(OH)_2)\)
\( AFm = \) monosulfate \((3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 12H_2O)\)
\( AFt = \) ettringite \((3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O)\)

Ettringite Formation with Calcium Hydroxide Dissolution

\[ 4(Na,K)^+ + 2\bar{S} + 2CH + AFm \rightarrow 4(Na,K)^+ + AFt + 4OH^- \]
**External Sulfate Attack**

- Mechanism – Example Reactions

**Symbols Used**

\[ S = \text{sulfate ion (SO}_4^{2-}) \]

\[ CH = \text{calcium hydroxide (Ca(OH)}_2) \]

\[ C\bar{S} = \text{calcium sulfate, gypsum (CaSO}_4 \cdot 2\text{H}_2\text{O)} \]

**Gypsum Formation with Calcium Hydroxide Dissolution**

\[(\text{Na,K})^+ + \bar{S} + CH \rightarrow 4(\text{Na,K})^+ + C\bar{S} + 4\text{OH}^-\]
External Sulfate Attack

• Mechanism – Example Reactions

Symbols Used

\( \tilde{S} = \) sulfate ion \((\text{SO}_4^{2-})\)

\(\text{CSH} = \) calcium silicate hydrate \((1.7\text{Ca} \cdot \text{SiO}_2 \cdot 4\text{H}_2\text{O})\)

\(\text{CSH}_D = \) decalcified calcium silicate hydrate \((<1.7\text{Ca} \cdot \text{SiO}_2 \cdot 4\text{H}_2\text{O})\)

\(\text{AFm} = \) monosulfate \((3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{CaSO}_4 \cdot 12\text{H}_2\text{O})\)

\(\text{AFt} = \) ettringite \((3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 3\text{CaSO}_4 \cdot 32\text{H}_2\text{O})\)

Decalcification of CSH Due To Ettringite Formation

\[ n\text{CSH} + \text{AFm} + 4(\text{Na,K})^+ + 2\tilde{S} \rightarrow n\text{CSH}_D + \text{AFt} + 4(\text{Na,K})^+ + 4\text{OH}^- \]
External Sulfate Attack

- Mechanism – Example Reactions

Symbols Used

\( S = \) sulfate ion \( (SO_4^{2-}) \)

\( CSH = \) calcium silicate hydrate \( (1.7Ca\_2SiO_3 \cdot 4H_2O) \)

\( CSH_D = \) decalcified calcium silicate hydrate \( (<1.7Ca\_2SiO_3 \cdot 4H_2O) \)

\( CS = \) calcium sulfate, gypsum \( (CaSO_4 \cdot 2H_2O) \)

Decalcification of CSH Due To Gypsum Formation

\[
nCSH + 2(Na, K)^+ + S \rightarrow nCSH_D + CS + 2(Na, K)^+ + 2OH^- 
\]
External Sulfate Attack

• Prevention

➢ Reduce concrete permeability

➢ Lower w/c – see ACI 201.2R-08 Guide to Durable Concrete for guidance on w/c determination based on sulfate exposure

➢ Use pozzolans or slag to densify the CSH and reduce the available CH for reaction – see ACI 201.2R-08 Guide to Durable Concrete for guidance on replacement level

➢ Reduce aluminate phases in cementitious materials that can react

➢ Use ASTM C150 Type II or Type V cement and limit aluminate phases in SCMs – see ACI 201.2R-08 Guide to Durable Concrete for guidance on cement selection based on exposure class and SCM composition
Internal Sulfate Attack

• Overview

➢ Expansive formation of ettringite in paste due to an internal source of sulfates
  ➢ Most commonly associated with high curing temperatures (delayed ettringite formation)
  ➢ Can also be associated with an internal source of sulfates (e.g. aggregate, fly ash, etc.)

➢ Fine cracking concentrated at joints but may effect entire slab area

➢ Usually manifests in 1 to 5 years

➢ Prevented by controlling internal sources of sulfate and avoiding high curing temperatures
Internal Sulfate Attack

• Mechanism

  ➢ Internal attack is most commonly associated with steam cured concrete

  ➢ If concrete is cured at temperatures above 70-80 ºC (158-176 ºF) the formation of ettringite during the initial set is “delayed” and monosulfate is formed

  ➢ The monosulfate later converts to ettringite after the concrete hardens resulting in expansion and cracking

  ➢ Other forms of internal sulfate attack involve sources of sulfate from the constituent materials leading to the same reactions observed with external sources of sulfate
Internal Sulfate Attack

• Prevention

➢ Fortunately not a distress associated with cast in place concrete

➢ For steam cured concrete observe proper curing temperatures

➢ For all concrete limit sulfate contents in constituent materials

    ➢ Portland cement is typically sulfate balanced and not an issue

    ➢ SCMs and aggregates can be likely sources of sulfate

➢ Problems with sulfide-bearing aggregates have been reported

    – Pyrrhotite \([\text{Fe}_{(1-x)}\text{S} \ (x = 0 \text{ to } 0.2)]\) weathering/oxidizing to provide sulfate internally
Sulfate Attack

• Testing

- Internal sulfate attack can be addressed by monitoring the chemical composition of the concrete materials

- ASTM C1012 *Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution*

  - Used to evaluate combinations of materials for susceptibility from external sulfate attack by monitoring the length change of mortar bars immersed in a sulfate solution

  - Test duration a minimum of twelve (12) months, or eighteen (18) months in the case of ACI 201 defined Class 3 exposure

  - Test exhibits considerable variation
Corrosion of Embedded Steel

• Overview

- Associated most often with chloride ingress, which destroys passivity film protecting steel
- Appears as cracking and spalling above and around embedded steel
- Rust staining
- Commonly manifests in 3 to 10 years
- Prevented by providing adequate concrete cover, reducing concrete permeability, protecting steel, or using corrosion inhibitors
Corrosion of Embedded Steel
Corrosion of Embedded Steel

• Mechanism
  - At the anode iron is oxidized and combines with OH\(^-\) to form corrosion product
  - At the cathode electrons, oxygen, and water combine to form OH\(^-\)
  - Both reactions must occur simultaneously
Corrosion of Embedded Steel

• Mechanism

- Under normal conditions the high pH of the concrete creates a corrosion layer on the steel that protects it from corrosion – passivation layer

- Passivation layer blocks the diffusion of water and oxygen to the steel and the cathode reaction is shut down, stopping the overall corrosion reaction
Corrosion of Embedded Steel

- Mechanism
  - Carbon dioxide (CO₂) lowers the pH of the concrete and the passivation layer breaks down
  - Chloride (Cl⁻) attacks the passivation layer directly – mechanism unclear
  - Limiting fluid ingress (CO₂, O₂, Cl⁻, and H₂O) limits corrosion

![Diagram of corrosion process](image-url)
Corrosion of Embedded Steel

• Prevention

- Provide adequate cover (concrete) over steel to reduce/slow the ingress of fluids

- Reduce concrete permeability (low w/c, penetrating sealers)

- Use corrosion inhibiting chemicals in the concrete to counteract the effects of chlorides or CO₂

- Minimize cracking in the concrete cover
Final Thoughts

• Many manifestations of MRD look similar, and therefore identification **cannot** be made based on visual assessment alone

• Water is necessary for deleterious expansion to occur

• Severe environments (freezing and thawing, deicers, sulfates, etc.) exacerbate the problem

• Durable materials are typically low shrinkage and relatively impermeable