

Reinforced Concrete Deterioration Mechanisms

Most reinforced concrete bridge deterioration traces back to a short list of mechanisms: corrosion of reinforcing steel (driven mainly by chlorides from deicing salt and seawater, and less often by carbonation), alkali-silica and alkali-carbonate reaction, freeze-thaw damage to paste and aggregate, deicer scaling, and external and internal sulfate attack. Each needs particular materials and a particular exposure, and each leaves a recognizable pattern of cracking, staining and spalling. This course explains how each mechanism works, how concrete cover and permeability control the time to corrosion, and how to read field findings back to a probable cause and the tests that confirm it. Its main sources are FHWA's Corrosion Protection: Concrete Bridges (FHWA-RD-98-088, Virmani and Clemena, 1998), FHWA-HRT-12-067 on corrosion-resistant reinforcement (2012), FHWA's ASR diagnosis and prognosis report (FHWA-HIF-09-004, January 2010) and FHWA's guidelines on materials-related distress (FHWA-RD-01-163, March 2002). The ACI 345.1R and ACI 546 references in the catalog are paywalled and were not reviewed for this course, so no requirement is quoted from them. Worked numbers in the examples are illustrative and computed for this course, not from a real bridge.

Learning objectives

- Explain chloride-induced corrosion
- Recognize ASR, freeze-thaw and sulfate attack
- Relate cover and permeability to durability
- Connect mechanisms to inspection findings

Why concrete bridges deteriorate

FHWA-RD-98-088 describes how reinforced concrete bridges performed reasonably well until the late 1960s. Then delamination and spalling, which used to be seen mainly in coastal areas, became common on decks in the snow belt, and some decks needed maintenance after as little as 5 years. The timing matched states' adoption of "bare pavement" policies and heavy use of sodium and calcium chloride deicers. By the mid-1970s, the report says, the cause was recognized: chloride from deicing salt reaching the reinforcing steel and corroding it.

Corrosion is the dominant mechanism on most bridge inventories, but it is not the only one. FHWA-RD-01-163, written for pavements but describing the same chemistry, classifies materials-related distress (MRD) into physical mechanisms (freeze-thaw of paste, deicer scaling, freeze-thaw of aggregate or D-cracking) and chemical mechanisms (ASR, ACR, external sulfate attack, internal sulfate attack and corrosion of embedded steel). It warns that more than one mechanism often acts at once, which complicates diagnosis.

A useful way to organize the mechanisms is by what each one needs:

Mechanism	What it needs	Where the damage starts
Chloride-induced corrosion	Chloride above a threshold at the steel, plus moisture and oxygen	At the bar, then cracks and delaminates the cover
Carbonation-induced corrosion	CO ₂ lowering the pH of the cover concrete to the bar depth	At the bar, usually where cover is thin or concrete is porous

ASR	Reactive silica in the aggregate, high-alkali pore solution, and moisture	Inside the concrete, at reacting aggregate particles
Freeze-thaw of paste	Saturated paste, freezing cycles, an inadequate air-void system	Near exposed surfaces, joints and edges
D-cracking	Susceptible coarse aggregate, saturation, freezing	Where moisture collects, often near joints and edges
Sulfate attack	External sulfate (soil, groundwater, some deicers) or internal sulfate and high curing temperatures, plus reactive aluminates	External: from exposed surfaces inward. Internal (DEF): throughout the paste

Durability is therefore not a fixed property of the concrete. FHWA-RD-01-163 quotes TRB's observation that the same concrete can last centuries in one environment and fall apart within a few years in another. An inspector or engineer diagnosing deterioration has to read both the material and its exposure.

How steel corrodes in concrete

FHWA-RD-98-088 explains the protective mechanism. Calcium hydroxide and smaller amounts of sodium and potassium give concrete a pore solution with a pH of about 12 to 13. At that alkalinity, a thin, tightly adhering oxide film forms on the steel and keeps it passive. As long as the film stays intact, the steel does not corrode in any meaningful way. FHWA-HRT-12-067 frames the same point with passivity at pH above about 13.2 and loss of protection when carbonation drops the pH below about 10.

The passive film can be destroyed in two ways:

- **Chlorides.** Chloride from deicing salt, salt spray or seawater moves slowly into the concrete, mostly through the pores in the cement paste. Once the chloride concentration at the steel exceeds a threshold, the film breaks down, and the steel corrodes wherever moisture and oxygen are available.
- **Carbonation.** CO₂ from the air forms carbonic acid, which converts calcium hydroxide to calcium carbonate and lowers the pH. FHWA-RD-98-088 says the pH can drop as low as 8.5. It also notes that carbonation is slow in good concrete: with a w/c of 0.45 and 1 in. of cover, carbonation is estimated to take more than 100 years to reach the steel. The report calls carbonation more of an issue in Europe than in the United States.

Once the film breaks down, corrosion proceeds as an electrochemical cell. Iron oxidizes at anodic sites and releases electrons. The electrons travel through the steel to cathodic sites, where they combine with oxygen and water to form hydroxide. The concrete pore solution is the electrolyte. FHWA-RD-98-088 describes both **micro-cells**, where anode and cathode are on the same bar, and **macro-cells**, where the top mat of a deck becomes the anode (more chloride and moisture) and the bottom mat becomes the cathode. Cracks speed both processes by giving chloride, water and oxygen a direct path to the steel.

The report also explains why chloride corrosion is self-sustaining. In low-oxygen conditions at the anode, iron-chloride complexes form and then hydrolyze, releasing the chloride to act again and producing acid. The local pH at the anode can drop to about 5 or 6. That anode becomes very different from the surrounding high-pH steel, which acts as a powerful macro-cathode feeding it. Chloride is not used up by the reaction.

Corrosion damages the concrete because rust occupies more volume than the steel it replaces. FHWA-RD-98-088 cites estimates of the resulting expansive pressure from 32 to 500 MPa, and estimates of the corrosion product needed to crack the cover ranging from 0.1 to 20 mils. The concrete first cracks along the bar, then delaminates in a plane at the level of the steel, and finally spalls. Section loss reduces bar area and bond. In prestressed members the report warns that the consequences are

more serious, because a small corrosion pit can fracture a highly stressed strand, whereas a mild-steel bar will "literally rust away before breaking."

The chloride threshold

The chloride threshold is the concentration at the steel at which corrosion starts. FHWA-RD-98-088 gives its history:

- A 1962 study reported a threshold of **0.15% soluble chloride by weight of cement**. For typical deck concrete with a cement factor of 7 sacks, the report converts this to about **0.025% soluble chloride by weight of concrete**, or 0.59 kg/m³ (about 1.0 lb/yd³).
- Later FHWA laboratory research estimated the threshold at **0.033% total chloride by weight of concrete**.
- The report notes that the threshold varies between bridges with cement type and mix design (C₃A and hydroxide content). It cites work suggesting a ratio of chloride to hydroxide, [Cl⁻]/[OH⁻], between 2.5 and 6.

FHWA-HRT-12-067 gives the commonly used figure for conventional carbon-steel reinforcement as about **1.2 lb/yd³**, roughly 0.03% by weight of concrete or 0.2% by weight of cement. It reports thresholds for stainless reinforcement at least 12 times higher. In FHWA-RD-98-088's summary of epoxy-coated bar field surveys, 0.6 kg/m³ (1.0 lb/yd³) is described as the level "considered by many" to be the threshold.

These figures are consistent with each other once the basis is matched. Watch the basis when reading a test report:

- **Soluble vs. total.** FHWA-RD-98-088 explains that only water-soluble chloride is believed to drive corrosion, but practice usually measures total (acid-soluble) chloride because the methods are simpler and more precise. The report cites AASHTO T 260 and ASTM C1218 for water-soluble chloride and AASHTO T 260 and ASTM C1152 for acid-soluble chloride.
- **By concrete vs. by cement.** Results are usually reported as a percentage of concrete mass because that does not require knowing the cement content of the hardened sample.
- **lb/yd³ vs. percent.** Multiply the percentage by the concrete unit weight.

Illustrative conversion. Take normal-weight concrete at about 3,900 lb/yd³ (roughly 145 lb/ft³). Then 0.025% × 3,900 ≈ 0.98 lb/yd³ and 0.033% × 3,900 ≈ 1.29 lb/yd³. A 7-sack mix has 7 × 94 = 658 lb of cement per cubic yard, and 0.15% × 658 ≈ 0.99 lb/yd³. All of these round to the same practical band of about 1 to 1.3 lb/yd³ for black bar. These numbers are illustrative; use the measured unit weight and cement content when you have them.

A threshold is a band, not a sharp line. Moisture, temperature, oxygen, cracks and the steel surface all affect whether corrosion actually starts at a given chloride level. FHWA-RD-98-088 reports field data from epoxy-coated bar decks with little or no corrosion in uncracked concrete with chloride as high as 7.6 kg/m³ (12.8 lb/yd³). It also reports most of the corrosion found was in cracked concrete with high chloride. Treat the threshold as an index for ranking risk, and combine it with half-cell and delamination data.

Cover, permeability and time to corrosion

FHWA-RD-98-088 treats cover and concrete quality as the first two lines of defense in new construction. It argues that adequate cover is "absolutely necessary" in a corrosive environment, and models chloride entry with Fick's second law of diffusion. For a semi-infinite slab exposed on one face, such as a deck or a pier face, the solution the report gives is:

$$C(x,t) = C_0 [1 - \operatorname{erf}(x / (2\sqrt{Dt}))]$$

Here $C(x,t)$ is the chloride concentration at depth x after time t , C_0 is the surface concentration, and D is the chloride diffusion coefficient of the concrete. The report explains that the required cover can be found by setting C at the bar depth equal to the threshold at the end of the target service life, using the measured diffusivity of the mix and the expected salt exposure.

What the equation says

The argument of the error function is $x/(2\sqrt{Dt})$. For a given ratio C/C_0 that argument is fixed, so $x^2/(Dt)$ is constant. Two practical consequences follow:

- **Time to initiation grows with the square of cover.** Increasing cover from 2 in. to 3 in. multiplies the initiation time by $(3/2)^2 = 2.25$, all else equal.
- **Time to initiation is inversely proportional to D .** Halving the diffusion coefficient through a lower w/c, supplementary cementitious materials or better curing doubles the initiation time.

Illustrative example

These numbers are illustrative only and are not from a real bridge. Assume a deck surface chloride of $C_0 = 15 \text{ lb/yd}^3$, a threshold of 1.2 lb/yd^3 for black bar, and an apparent diffusion coefficient of $D = 0.05 \text{ in.}^2/\text{yr}$. Ignore initial chloride in the mix and assume D and C_0 are constant.

- $C/C_0 = 1.2/15 = 0.080$, so $\text{erf}(z) = 0.920$, and $z \approx 1.24$ from an erf table.
- With 2.5 in. cover: $t = (x/2z)^2/D = (2.5/2.48)^2/0.05 \approx 1.016/0.05 \approx 20$ years.
- With 3.0 in. cover: $t = (3.0/2.48)^2/0.05 \approx 1.463/0.05 \approx 29$ years.
- With 2.5 in. cover and D halved to $0.025 \text{ in.}^2/\text{yr}$: $t \approx 41$ years.

The model ignores cracking, changes in D with age and moisture, and chloride binding. Real decks with cracks can initiate much sooner. Its value is in ranking options and showing how strongly cover and permeability drive service life.

Initiation is not the end of service life. FHWA-HRT-12-067 describes a propagation period after initiation before cracking and spalling require repair, which it puts at roughly 20 years for its model conditions. The engineering choice is between delaying initiation (cover, low permeability, sealers) and slowing propagation once chloride arrives (corrosion-resistant reinforcement, inhibitors, cathodic protection).

Specifying cover that is actually built

FHWA-RD-98-088 makes a point that is often missed: the design cover must allow for construction tolerances. Its example assumes a required minimum cover of 50 mm and measured as-built covers that follow a normal distribution with a standard deviation of about 10 mm. The specified cover then has to be about 67 mm for 95% compliance or 73 mm for 99% compliance. The same logic explains why many corrosion problems cluster where cover is thinnest, over chairs that sank, at construction joints, and near drains and curbs.

The report also warns that adequate cover alone is not enough. Concrete eventually cracks, and variations in cover and density create micro-cells along the bars. It recommends a system approach combining cover, quality concrete, corrosion inhibitors and corrosion-resistant reinforcement.

Concrete quality and permeability

FHWA-RD-98-088 says concrete contains interconnected pores through which water and oxygen move. A low w/c and good consolidation reduce the number and size of pores, which lowers permeability and electrical conductivity. A low w/c also

raises strength, which delays cracking from corrosion pressure. In an FHWA study of 30 mixes covering cement type, mineral admixture, aggregate type, w/c (0.3, 0.4 and 0.5) and air content, w/c, cement type, mineral admixture and fine aggregate had the largest effect on corrosion behavior. Coarse aggregate and air content had less effect. Low-C₃A Type I cement, quartz aggregates and silica fume performed best in that study.

The same report notes that blended cements can reduce alkalinity, but says this is likely more than offset by large reductions in permeability and conductivity, especially when they allow a lower w/c.

FHWA-RD-98-088 also reports that environment matters as much as the mix. In its test program, chloride from 0.59 to 5.9 kg/m³ (1 to 10 lb/yd³) produced a wide range of corrosion behavior, and temperature and humidity significantly affected rates. High temperature (38°C) with intermediate humidity (75%) was identified as producing high corrosion rates. Intermediate moisture was more aggressive than very dry or fully saturated conditions, because saturated concrete limits oxygen supply and dry concrete has high resistance. This is one reason splash and tidal zones and the leaking undersides of joints corrode faster than continuously submerged or sheltered concrete.

Protection systems and what they imply for inspection

Knowing the protection system used when a bridge was built helps explain what you see in the field. FHWA-RD-98-088 summarizes the main ones:

- **Epoxy-coated reinforcement (ECR).** The first U.S. bridge with epoxy-coated bars was built in 1973 in West Conshohocken, Pennsylvania. By the time of the report, ECR had been used in about 20,000 bridge decks. In FHWA's review of 92 decks studied by 11 states and Canadian SHRP, 81% of the extracted bar segments had no corrosion, and most of the corrosion found was in cracked concrete with high chloride. A West Virginia deck had about 0.25% delaminated area after 19 years, against 5% to 20% typically expected for a black-bar deck of the same age and design. Performance was better when both mats were coated. Coating defects, holidays and loss of adhesion reduce performance. The report describes the Florida Keys substructure failures in the marine splash zone that raised concerns about ECR in hot, wet, salty exposure.
- **Galvanized bars.** The report describes conflicting laboratory and field results. One panel concluded that zinc tolerates more chloride before corrosion starts, but that once corrosion begins, distress may progress faster than for black steel. A 10-year FHWA slab study found no extra benefit at w/c 0.40.
- **Stainless and clad bars.** Solid 316 stainless bars and stainless-clad bars performed very well in FHWA screening tests aimed at 75 to 100 years of corrosion-free life, at higher first cost. FHWA-HRT-12-067 reports stainless thresholds at least 12 times that of carbon steel.
- **Corrosion-inhibiting admixtures,** such as calcium nitrite, combined with ECR in some systems.
- **Overlays, membranes and sealers.** The report says FHWA research and field data show these slow corrosion but do not stop it once chloride is at the steel.
- **Cathodic protection and electrochemical chloride extraction** for rehabilitation, covered in the repair course.

For inspection, these points matter. A deck built with black bar in a salt state in the 1960s or 1970s is expected to have chloride-induced delamination. A deck with ECR in both mats that shows widespread delamination warrants a closer look at cover, cracking, coating quality or another mechanism. Splash-zone substructure in warm seawater is a known high-risk location for any bar type.

Alkali-silica reaction

Alkali-silica reaction (ASR) is a chemical reaction between the alkaline pore solution and certain reactive forms of silica in the aggregate. FHWA-RD-01-163 describes the product as an alkali-calcium-silica-hydrate gel that expands significantly as it absorbs moisture, damaging the aggregate particle and the surrounding paste. It lists three conditions that must all be present:

- Reactive forms of silica in the aggregate
- A high-alkali pore solution
- Sufficient moisture

Alkalinity is usually reported as sodium oxide equivalent, $\text{Na}_2\text{O} + 0.658 \text{ K}_2\text{O}$. The same source lists common reactive aggregate types, including opaline or chalcedonic cherts, siliceous limestones, rhyolites, dacites, andesites and their tuffs, and phyllites. It notes that some aggregates react within weeks and others over many years.

Moisture and timing

FHWA-HIF-09-004 (Sec. 3.1) states that ASR typically develops or is sustained in concrete with internal relative humidity above 80 to 85%. ASR is not expected to develop significantly where average ambient RH is below about 60%, which the report says is normally only found inside buildings. Between 60% and 80%, the extent depends on the aggregate's reactivity, and expansion rates are lower. The report adds that expansion and cracking are generally most severe where an element has an external, constantly renewed moisture supply. Sun exposure, wetting and drying, freeze-thaw and saline water (tidal zones, or barriers and abutment walls splashed with deicer) usually make the cracking and deterioration worse, even if they do not drive the expansion itself.

The same report (Sec. 4.1) says ASR may take from 3 to more than 25 years to develop significantly, depending on aggregate reactivity, moisture, temperature and concrete alkali content. FHWA-RD-01-163's pavement table puts typical appearance at 5 to 15 years.

What ASR looks like

FHWA-HIF-09-004 lists the common visual symptoms, while noting that none is exclusive to ASR:

- Expansion causing deformation, relative movement and displacement
- Cracking
- Surface discoloration
- Gel exudations
- Occasional pop-outs

The report explains why ASR cracking appears on the surface. The interior of a member stays moist and keeps expanding, while the surface layer, which dries and leaches alkalis, expands less and cracks in tension. In unrestrained or lightly reinforced concrete this produces **map cracking**. In members with strong reinforcement or prestress, cracks tend to align with the main reinforcement or the direction of compressive stress. For example, longitudinal cracks may run along a prestressed girder or a heavily reinforced column.

Table 2 of FHWA-HIF-09-004 (modified from CSA A864-00) classifies the condition-survey features as low, medium or high potential for ASR:

Feature	Medium potential	High potential
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Expansion or displacement	Some evidence, such as closed joints, spalls, misalignment between members	Fair to extensive volume increase, spalling at joints, displaced or misaligned members
Cracking	Some cracking typical of ASR (map cracking or cracks aligned with main reinforcement or stress)	Extensive map cracking or cracking aligned with major stress or reinforcement
Discoloration	Slight discoloration at some cracks	Many cracks with dark discoloration and an adjacent zone of light-colored concrete
Exudations	White exudations at some cracks, possibly colorless gel	Colorless, jelly-like exudations readily identifiable as ASR gel at several cracks

The report pairs this with exposure. Elements in dry, sheltered locations have low potential. Elements outdoors but sheltered from constant wetting have medium potential. Elements frequently exposed to rain, groundwater or water from the structure's function have high potential. When potential is low, further work waits for the next survey. When it is medium or high, the report recommends a preliminary diagnosis program.

Confirming ASR

FHWA-HIF-09-004 sets out a staged program:

- **Level 1: condition survey.** Record type, extent and location of distress with photos (including scale) and sketches mapping low, medium and high severity zones, and note restraint and moisture sources. A SHRP field test for gel using uranyl acetate fluorescence exists, but the report warns that results need careful interpretation (ASTM C856).
- **Level 2: preliminary diagnosis.** Gather documents, including construction records, mix designs, aggregate sources, and inspection history and dates. Check other structures built with the same materials. Measure a Cracking Index on representative surfaces. Take cores for petrographic examination.
- **Level 3: detailed investigation and prognosis.** Estimate expansion to date, the current expansion rate (in-situ monitoring and core expansion tests at 38°C and RH above 95%), and the potential for further expansion, then use these to choose mitigation.

The report stresses that field observation alone often cannot separate ASR from other causes such as internal sulfate attack or plastic or drying shrinkage. Petrography is what usually distinguishes ASR (reaction rims, gel in cracks and voids, cracked aggregate particles) from alkali-carbonate reaction. FHWA-RD-01-163 adds that gel alone does not prove destructive ASR, because gel can be present without significant distress.

Alkali-carbonate reaction

FHWA-RD-01-163 describes alkali-carbonate reaction (ACR) as a reaction between alkalis and certain dolomitic carbonate aggregates with a characteristic texture of dolomite rhombs in a clayey fine matrix. Dedolomitization and brucite formation cause expansion. Surface symptoms resemble ASR: map cracking and pressure-related distress. Unlike ASR, the source says pozzolans are not effective against ACR. Prevention relies on avoiding the aggregate, diluting it, reducing its size, or limiting alkali to a lower level than is used for ASR.

Prevention and existing structures

For new work, FHWA-RD-01-163 lists non-reactive aggregates, low-alkali cement (below 0.60% Na₂O equivalent, successful with slightly to moderately reactive aggregates), fly ash, slag, silica fume and other pozzolans (effectiveness depends heavily on type and dosage), limits on total alkali, and lithium compounds for highly reactive aggregates. For in-service concrete, it says no method has been shown to stop ASR completely. Silane sealers, methacrylate and lithium have been tried. The

practical lever on an existing bridge is usually moisture: fix drainage, joints and leaks, and consider breathable sealers. Structural assessment should consider whether expansion has caused loss of bond, cracked anchorage zones, or displaced bearings and joints.

Freeze-thaw damage to the paste

FHWA-RD-01-163 describes freeze-thaw deterioration of hardened paste as damage to **saturated** cement paste under repeated freezing and thawing. It says there is no consensus on the exact mechanism. The two leading theories are:

- **Hydraulic pressure** (Powers, 1945). Water expands about 9% as it freezes, pushing unfrozen water away from the freezing sites. If the pore system resists that flow, the pressure can exceed the paste's tensile strength.
- **Osmotic pressure** (Powers, 1975). As pure water freezes in the larger pores, the remaining liquid becomes more concentrated, drawing water from the surrounding paste toward the freezing sites. If there is not enough air-void space nearby to receive it, the resulting pressure can fracture the paste.

Both theories point to the same cure: a system of small, closely spaced entrained air bubbles that gives water somewhere to go within a short distance. Air entrainment is described in FHWA-RD-01-163 as an effective way to prevent paste freeze-thaw damage.

The air-void system

FHWA's Tech Brief on air entrainment (FHWA-HIF-17-009) explains that total air content alone does not guarantee protection, because the size and spacing of the voids matter. ASTM C457 measures the hardened air-void system, including:

- **Spacing factor**, an index of the distance between voids, recommended at 0.008 in. (0.200 mm) or less
- **Specific surface**, the surface area of the voids divided by their volume, recommended at 600 in.²/in.³ (24 mm²/mm³) or more

The tech brief notes that ASTM C457 treats 0.008 in. as appropriate for moderate exposure. A higher value may be acceptable for mild exposure, and a lower value is needed for severe exposure, especially with deicing chemicals. It also describes the Super Air Meter (AASHTO TP 118), which estimates air-void quality in fresh concrete. A SAM Number of 0.2 psi is roughly correlated with a 0.008 in. spacing factor and a durability factor of 70% in AASHTO T 161.

For forensic work on an existing bridge, ASTM C457 on cores tells you whether the concrete was ever properly air-entrained. FHWA-RD-01-163 adds a complication: secondary ettringite can fill air voids over time in permeable, saturated concrete, reducing their effectiveness. Concrete that looks like it failed from poor air may originally have had an adequate system.

How it appears

FHWA-RD-01-163 describes paste freeze-thaw damage as scaling, map cracking, or severe cracking, spalling and deterioration that commonly starts at joints and free edges, where moisture is more available. It is typically top-down, with fractures roughly parallel to the surface that decrease with depth. The source puts typical appearance at 1 to 5 years in pavements. On bridges, look first where water collects and freezes: under leaking deck joints, on pier caps and abutment seats below joints, on barrier and parapet faces splashed by plows, and on the fascias of decks with poor drips.

Deicer scaling and D-cracking

Deicer scaling

FHWA-RD-01-163 describes deicer scaling as scaling or crazing of the surface from repeated deicer application. The proposed physical mechanisms are high thermal strain when deicer melts ice, osmotic pressure between dilute surface water and concentrated salt solutions in the concrete, and salt crystallization pressure. The source also cites studies suggesting chemical changes: dissolution of calcium hydroxide, coarsening of the pore system and possibly expansive products, often seen as staining and deterioration near joints.

The source notes that over-vibrated or poorly finished concrete is more vulnerable because it leaves a weak paste or mortar layer at or just below the surface. Even well air-entrained concrete can scale. It cites ACPA (1992) recommendations: minimum cement content of 335 kg/m³, maximum w/c of 0.45, adequate curing, and at least 30 days of environmental exposure before deicers are applied. On a bridge deck, scaling is a surface defect that reduces cover and ride quality. On its own it is not usually a structural issue, but it brings chloride closer to the steel.

D-cracking

FHWA-RD-01-163 describes freeze-thaw deterioration of aggregate, commonly called D-cracking, as damage from susceptible coarse aggregate particles that fracture or dilate when they freeze saturated. Key aggregate properties are composition, pore structure, sorption and size. Most susceptible aggregates are sedimentary, commonly limestone, dolomite or chert.

In pavements, it first appears as a series of fine cracks running parallel to joints, cracks or free edges, often with dark staining in an hourglass shape at affected joints. It commonly starts near the bottom of the slab, where moisture accumulates. Typical appearance is 10 to 15 years. Spalling and disintegration follow.

The key point for diagnosis: **air entrainment of the paste does not prevent D-cracking**. The source says prevention relies on rejecting susceptible aggregate, or on reducing the maximum size of the susceptible aggregate. On bridges, the same mechanism can appear in decks, approach slabs and other elements where saturated concrete freezes, especially near joints. The repair implication is that patching with the same aggregate source can reproduce the problem.

Sulfate attack and delayed ettringite formation

External sulfate attack

FHWA-RD-01-163 describes external sulfate attack as chemical attack by sulfate ions entering from outside, from groundwater, soil or some deicing chemicals. Two reactions are involved:

- Sulfate combines with calcium to form **gypsum**. The expansion may not be destructive, but gypsum has little cementing value, so the concrete loses integrity.
- Sulfate combines with hydrated calcium aluminate to form **ettringite**, which can exert very destructive expansive pressure.

In pavements, the source describes fine cracking near joints and edges, or map cracking, eventually leading to joint or surface deterioration, typically within 3 to 10 years. Prevention includes minimizing C₃A in the cement, reducing calcium hydroxide with pozzolans, and keeping w/c below 0.45.

On bridges, the elements most exposed to sulfate are foundations, footings, piles and abutment walls in sulfate-bearing soil or groundwater, and in some regions elements in contact with sulfate-rich water. Look for softening, loss of paste and cracking at and below the ground or waterline. Confirmation needs sampling of the soil or water for sulfate and petrographic and chemical testing of the concrete.

Internal sulfate attack and DEF

Internal sulfate attack involves sulfate from inside the concrete. FHWA-RD-01-163 distinguishes:

- **Secondary ettringite formation (SEF).** Ettringite dissolves and reprecipitates in voids and existing microcracks in permeable, saturated concrete. Most experts consider it not expansive enough to crack healthy paste, but it can fill air voids and reduce freeze-thaw resistance. Whether it is a cause or a symptom is debated.
- **Delayed ettringite formation (DEF).** At high curing temperatures, primary ettringite does not form properly. After the concrete cools, sulfate and aluminate in the paste react to form expansive ettringite, causing microcracking and gaps around aggregate particles. The source cites a minimum temperature of 65°C to 80°C in current research, with many citing 70°C. DEF is most closely associated with steam curing.

The source describes internal sulfate attack in structures as closely spaced, tight map cracks with wider cracks at regular intervals. Microscopically, DEF shows gaps in the paste around aggregates with widths proportional to particle size.

DEF matters for bridges because the high-temperature condition can occur in steam-cured precast elements and in massive cast-in-place placements that heat up from hydration. Many owners now cap curing or internal temperatures in their specifications for these reasons. Check the governing specification. Because DEF and ASR cracking can look alike, and the two often occur together, petrography is needed to tell them apart.

Reading field findings back to a mechanism

An inspector records defects. An engineer has to work back from defects to a mechanism, because the mechanism determines the repair. The table below pairs common bridge findings with the mechanisms they suggest. It is a starting point for deciding what to test, not a diagnosis.

Finding	Most likely mechanism(s)	Next step
Cracks running along bars, rust staining, hollow sound, spalls exposing corroded bar	Chloride-induced corrosion (or carbonation where cover is thin)	Delamination survey, cover survey, chloride profile, half-cell
Map cracking over a whole face, with dark crack edges, gel or white exudations, closed joints or displaced members	ASR (or ACR), possibly with DEF	Cores for petrography, records review, Cracking Index, expansion monitoring
Surface scaling and loss of paste at joints, edges and plow-splash zones	Paste freeze-thaw or deicer scaling	Cores for ASTM C457 air-void analysis
Fine cracks parallel to joints or edges with dark staining, deterioration starting from the wet face	D-cracking	Petrography of coarse aggregate, aggregate source history
Softened, crumbling concrete at or below grade or waterline	External sulfate attack (or acid attack)	Soil and water sulfate testing, petrography, chemistry
Tight map cracks with wider regularly spaced cracks in steam-cured precast or mass concrete	DEF (internal sulfate attack), often with ASR	Petrography, curing records

Several of these mechanisms often act together. ASR cracks let in chloride and water, which speeds corrosion and freeze-thaw damage. Freeze-thaw scaling reduces cover. Corrosion cracks give sulfate and chloride a path. FHWA-RD-01-163 makes this point repeatedly, and FHWA-HIF-09-004 warns against attributing all distress to ASR on visual evidence alone.

Element-level defects

Under the AASHTO Manual for Bridge Element Inspection, concrete defects are recorded as defect elements within each structural element and assigned condition states. The concrete defects typically include delamination, spall or patched area; exposed reinforcing; efflorescence and rust staining; cracking; and abrasion and wear. A CS3 or CS4 delamination quantity tells you how much concrete is affected, but not why. Pair the element data with the mechanism-specific tests below before scoping repairs.

Field and laboratory evaluation methods

FHWA-RD-98-088 lists the methods states used in field evaluations of decks:

- Visual examination for cracking, spalling and patches
- Chain dragging to locate delamination
- Pachometer (cover meter) surveys for cover and bar location
- Drilled powder samples and cores for chloride content
- Coring to assess concrete quality
- Half-cell potential surveys
- Concrete resistivity surveys
- Corrosion rate by linear polarization resistance

Laboratory work on cores included visual examination, cover over the bar, condition of extracted bar segments, coating thickness, total or water-soluble chloride, permeability, pH next to the bar, compressive strength and unit weight.

What each method tells you

- **Chain drag and hammer sounding** find delaminations, the planes of separation at bar level caused by corrosion pressure. They map damage that has already happened.
- **Cover surveys** explain why damage clusters where it does and give the x in the diffusion model.
- **Chloride profiles**, taken at several depth increments (for example, 1/2 in. steps down past bar depth), give surface concentration, chloride at the bar and an apparent diffusion coefficient. Comparing chloride at the bar with the threshold tells you how much of the deck is contaminated but not yet delaminated. That hidden quantity often decides whether patching or a larger rehabilitation is economical.
- **Half-cell potential** (ASTM C876) maps the probability of active corrosion. More negative potentials indicate a higher likelihood of active corrosion. Readings are affected by moisture, temperature, coatings and overlays, so interpret them as a map of relative risk, preferably with chloride and delamination data.
- **Resistivity and linear polarization** give an indication of corrosion rate.
- **Petrography** (ASTM C856) identifies ASR, ACR, DEF, freeze-thaw damage, air-void deficiencies and paste quality. It is the main confirmation test for every non-corrosion mechanism in this course.
- **Air-void analysis** (ASTM C457) checks spacing factor and specific surface.

Sampling plan

Plan sampling to answer specific questions. Take cores from both distressed and apparently sound areas, so you can compare them. Locate cores using the delamination and half-cell maps. Avoid cutting prestressing strand or primary reinforcement, using a cover meter or GPR first. Record core location, orientation, cover, visible cracking and bar condition before shipping. For ASR, FHWA-HIF-09-004 recommends documentation before coring: construction records, aggregate sources, mix designs, and the history of when distress was first observed.

Illustrative case: diagnosing a 1970s deck

This case is illustrative and composed for this course. It is not a real bridge.

Structure. A three-span continuous steel girder bridge built in 1974 in a northern state, with an 8.5 in. cast-in-place deck, black bar in both mats, and 2 in. specified top cover. The deck has a history of patching.

Findings. Chain drag shows 9% of the deck area delaminated or patched, concentrated in the right wheel paths and along the curb lines. A cover survey shows median top cover of 1.9 in. with about 15% of readings under 1.5 in., mostly near the curbs. Chloride samples at bar depth average 3.1 lb/yd³ in delaminated areas and 1.8 lb/yd³ in sound areas. Half-cell readings are more negative across much of the right lane than the left. Two cores from the left lane, which shows map cracking with no delamination, show a petrographer some gel in voids and cracked chert particles in the coarse aggregate.

Interpretation.

- The primary mechanism is **chloride-induced corrosion**. Chloride at the bar exceeds the roughly 1.2 lb/yd³ black-bar threshold even in sound areas, so the contaminated area is larger than the 9% currently delaminated. The concentration near the curbs matches the thin cover.
- A secondary **ASR** component is present in the left lane. It needs Level 2 follow-up per FHWA-HIF-09-004 (records review for the aggregate source, more cores, Cracking Index) before it is weighted in the decision.
- Using the Fick relationship, the engineer can estimate how quickly the sound-but-contaminated areas will progress. That points to removal of contaminated concrete over a larger area, or electrochemical options, rather than patching only the delaminations. Patching only the delaminated areas in chloride-contaminated concrete risks new corrosion around the patches, which the repair course covers as the ring-anode or halo effect.

Lessons. Visible damage understated the problem, chloride data found the hidden part, and a second mechanism was found only because cores were taken from an area that looked different. Each test answered a specific question.

Exposure zones on a typical bridge

The same concrete deteriorates differently in different parts of a bridge, because moisture, chloride and freezing vary from place to place. A useful habit is to map the structure by exposure before looking for defects.

- **Deck top surface.** Direct deicer application, traffic wear and freeze-thaw. Expect chloride-induced corrosion of the top mat, scaling, and, with susceptible materials, ASR or D-cracking.
- **Deck underside, fascias and overhangs.** Leakage through cracks and joints and runoff around the fascia carry chloride to the bottom mat and the overhang edge. Missing or blocked drips concentrate damage here.
- **Below joints.** Pier caps, abutment seats, bearing areas and girder ends under leaking joints get salt water repeatedly and often stay wet. These areas commonly combine corrosion, freeze-thaw and, where present, ASR.

- **Barriers and parapets.** Plow spray and splash wet them with salty slush. They are thin elements with many exposed faces, so they are frequently among the first to show scaling and corrosion.
- **Splash and tidal zones of marine substructure.** FHWA-RD-98-088 lists the marine conditions: higher year-round temperature, a continuous supply of moisture and chloride, and more wetting and drying cycles. These lower concrete resistivity and drive corrosion faster. FHWA-HIF-09-004 also identifies tidal and splash zones as places where ASR damage is worse.
- **Footings, piles and walls in soil or groundwater.** Sulfate exposure where soils or water carry sulfate, plus wetting from groundwater.
- **Sheltered interior faces,** such as box girder interiors and the insides of hollow piers, are usually the least aggressive, unless drains leak into them.

Prestressed concrete and other considerations

FHWA-RD-98-088 gives prestressed members particular attention. It notes that slightly more than 10% of the bridges in the National Bridge Inventory at that time had prestressed concrete superstructures. It also says prestressed members, although usually made with higher-strength concrete, are subject to the same corrosion as reinforced concrete. Because members rely on the tensile strength of the strands, the loss of even a few strands could be serious, and high stress accelerates corrosion effects. The report also mentions the sudden collapse of two post-tensioned bridges in the United Kingdom and one in Belgium as the reason FHWA evaluated nondestructive methods for detecting voids in post-tensioning ducts, leading to field use of impact-echo.

For inspection, this means:

- Longitudinal cracks along the strand lines on the soffit or bottom flange of a prestressed beam, with or without rust staining, deserve close attention. They may indicate strand corrosion, though they can also reflect ASR, DEF or fabrication cracking.
- Spalls at beam ends under leaking joints can expose strand at the anchorage region.
- For post-tensioned structures, grout voids and water in ducts are hidden defects. Inspection relies on targeted NDE and invasive openings rather than visual signs alone.

The prestressed damage course goes into these in detail.

Acid and other chemical exposure

FHWA-RD-98-088 also notes that sulfur dioxide and nitrogen oxides from burning fossil fuels form acids in rain and snow that are more aggressive than carbonic acid. It reports data suggesting deicing salts combined with acid precipitation are more corrosive than either alone, and calls for more study of the effect on concrete. In practice, acid attack is a local issue on bridges, for example near industrial discharges, in mine drainage, or in some streams and soils. It shows as loss of paste with exposed aggregate. Confirm it with water or soil chemistry before attributing damage to it.

Abrasion

FHWA-RD-01-163 sets abrasion aside as a load-related rather than environmental problem, noting that tire chains and studded tires can wear even good concrete. On bridges, abrasion from traffic on decks and from debris and sediment on piers in streams reduces cover and can expose reinforcement. It should be recorded and taken into account when evaluating cover.

Inspector's checklist for concrete deterioration

Use this list on any reinforced concrete element where deterioration is suspected:

1. Map the exposure: where water, salt and freezing reach the element, and where it stays wet.
2. Record defects by type, extent and location with photos (including scale) and sketches. Note cracking patterns: along bars, map pattern, parallel to joints or edges, or aligned with prestress.
3. Note staining: rust (corrosion), white efflorescence (water moving through cracks), dark crack margins (ASR) or hourglass staining at joints (D-cracking).
4. Sound the surface and drag chain on decks to find delaminations.
5. Note signs of expansion: closed joints, displaced bearings, misaligned members or crushed joint edges.
6. Check drainage, joints, drips and deck drains, since most mechanisms need water.
7. Gather records: year built, mix design, aggregate source, reinforcement type (black, epoxy-coated, galvanized or stainless), cover, curing method for precast, and prior inspections and repairs.
8. Recommend targeted testing: chloride profiles, cover, half-cell, cores for petrography and air-void analysis, and soil or water sulfate where relevant.
9. Report the probable mechanism and confidence level, and say what testing would confirm it.

Diagnosing the mechanism correctly is the most important step in a concrete repair. Patching corrosion spalls on a deck that also has active ASR, or replacing D-cracked concrete with the same aggregate, spends money without stopping the deterioration.

Key takeaways

- Concrete's high-pH pore solution (about 12 to 13) keeps a passive film on the steel. Chlorides or carbonation destroy that film.
- The classic carbon-steel chloride threshold is about 1 to 1.3 lb/yd³ (about 0.025 to 0.033% by weight of concrete, or about 0.15 to 0.2% by weight of cement). Always check whether a result is soluble or total, and by concrete or by cement.
- Corrosion products expand, causing cracking along the bar, then delamination at the bar level, then spalling. Prestressing strand can fracture from small pits.
- Cover and permeability set the time to corrosion. Diffusion depth grows with the square root of time, so added cover buys time roughly in proportion to its square.
- ASR needs reactive silica, high alkali and moisture (internal RH above about 80%). Map cracking, gel, discoloration and expansion are its signs, and petrography confirms it.
- Freeze-thaw paste damage is prevented by a proper air-void system (spacing factor 0.008 in. or less). D-cracking is an aggregate problem that air entrainment does not prevent.
- External sulfate attack forms expansive ettringite and gypsum from outside sulfate. DEF is internal sulfate attack linked to high curing temperatures, about 70°C and above.
- Field findings point to a probable mechanism, but confirmation needs testing: chloride profiles, half-cell, cover survey, cores and petrography.

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8. FHWA, Tech Brief: Air Entrainment and Concrete Durability, FHWA-HIF-17-009

Study text only; the final quiz is taken online. Verify all references against the edition your agency has adopted.