COMMENTARY ON AASHTO R 101, Developing Performance Engineered Concrete Pavement Mixtures

June 2024

National Concrete Pavement Technology Center

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Iowa State University
Institute for Transportation

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About This Manual

This manual presents commentary explaining the context and rationale for the recommendations provided in AASHTO R 101, Developing Performance Engineered Concrete Pavement Mixtures. The intent of the commentary is to provide transportation agencies with tools for preparing a specification for concrete pavement mixtures that moves closer to measuring and basing acceptance on parameters that are critical to the long-term performance of the pavement system.

This manual is a product of the National Concrete Pavement Technology Center (CP Tech Center) at Iowa State University's Institute for Transportation. For more detailed information about the topics covered in this manual, readers are encouraged to consult the CP Tech Center's performance-engineered mixtures (PEM) website and the final report for the Performance-Engineered Concrete Paving Mixtures Transportation Pooled Fund (TPF-5(368)), *Performance-Engineered Concrete Paving Mixtures*.

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Commentary on AASHTO R 101, Developing Performance Engineered Concrete Pavement Mixtures

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Introduction

This document provides commentary on the requirements for concrete mixtures discussed in AASHTO R 101, Developing Performance Engineered Concrete Pavement Mixtures. This commentary is intended to provide agencies with tools to prepare a specification for concrete pavement mixtures that moves closer to measuring and basing acceptance on parameters that are critical to the long-term performance of the pavement system.

Background

The assumption inherent in concrete pavement structural design is that if the concrete possesses the required mechanical properties, it will achieve design expectations. This assumption supposes that the concrete is durable, but durability is not an intrinsic, measurable property of concrete. Instead, durability depends on a set of concrete material properties that enable the concrete to resist the environment in which it serves (TRB 2013). For instance, a concrete placed in a mild, dry environment may remain wholly intact for decades, yet this same concrete may rapidly disintegrate if exposed to chemical deicers in a wet, freeze-thaw environment. Both the environment and the materials must be considered together to specify and construct durable concrete pavements.

A review of current specifications has shown that many are largely based on strength, slump, thickness, and total air content, which provide limited correlation with the mechanisms of pavement failure due to commonly observed durability-related distresses. The need for a change in the way concrete is specified, especially concrete for paving mixtures, is becoming increasingly apparent as mixtures become more complex due to a growing range of chemical admixtures and supplementary cementitious materials (SCMs), traffic loadings continue to increase, more aggressive winter maintenance practices are implemented, and demand increases to build systems more quickly, more cheaply, with increased longevity, and with a reduced amount of embodied carbon.

A significant barrier to pure performance-based specifications, or even performance-related specifications, has been the lack of effective test methods that assess the ability of a concrete mixture to resist the environment to which it is exposed. New testing methods that measure performance-related parameters have been developed and are being evaluated in the field, while other advances are emerging.

A group of senior experts representing agencies, industry, and academia met at two events sponsored by the Federal Highway Administration (FHWA) and agreed that the following are the critical parameters that should be addressed in a materials specification:

- Strength
- Susceptibility to cracking and warping due to drying shrinkage
- Durability (freeze-thaw resistance)
- Durability (resistance to chemical deicers)
- Durability (low absorption, diffusion, and other transport-related properties)
- Durability (aggregate stability)
- Workability

Workability, while of limited direct interest to agencies, is included because many failures in the field have been attributed to inappropriate workability of the mixture for the placement technique used, leading to segregation, poor consolidation, edge slump, and/or inadequate air content in the completed slab.

Scope

This commentary supplements the sections of AASHTO R 101 that address the critical parameters listed above, providing background information on and the rationale behind the approaches recommended in the specification. For each critical parameter, relevant quotes from AASHTO R 101 are presented along with the numbered clause in which the quote appears.

The means through which desired performance requirements are typically sought in a prescriptive specification are also discussed to provide a comparison between the recommendations in this document and current practice. It is recommended that performance-based and prescriptive approaches not be mixed, to the degree possible, because doing so may make it impossible to achieve all of the demands on the pavement system.

It is recognized that some of the recommended tests are slow, expensive, and/or relatively insensitive to workmanship issues. Therefore, this document separates the testing necessary to accept concrete during production from the testing necessary for mixture qualification. Note that a key element of the practice described in AASHTO R 101 is the requirement to provide some test data during qualification that have no limits. These data either tie back to the parameters used in the mechanistic-empirical (ME) pavement design process or point forward as a baseline for QC and acceptance tests to follow in future versions of the specification.

It is also recognized that some of the test methods are relatively new, and therefore only limited correlation between test results and long-term field performance has been achieved to date. However, there is considerable value in applying the tests to seek information on batch-to-batch uniformity and to establish an extensive data set as a foundation for future correlation. Limits are therefore suggested based on the science available at present, and percent within limits (PWL) plans can be imposed based on local practices to ensure that the mixtures delivered are consistent.

At this juncture, the recommended limits are established for an average concrete pavement life (i.e., 30 years).

However, as is true with any test method, there is always some risk that inherent variability in materials and testing will result in a shorter or longer life. As the use of the test methods and associated service life prediction models increases over time, the risks associated with their predictive ability will be reduced and more definitive limits can be established.

This document is limited at present to discussing the production and delivery of a desirable mixture and does not address placement issues such as consolidation, finishing, and curing.

Strength

Minimum concrete flexural strength of 4.1 MPa (600 psi) at 28 days using T 97 ..... 6.3.1

A concrete strength value that meets the specific project design requirements may be substituted here because it is not uncommon for agencies to select other values for strength at other ages. Agencies may elect to only require either flexural strength or compressive strength. Acceptance criteria are addressed in Section 7 of AASHTO R 101

Minimum concrete compressive strength of 27.5 MPa (4,000 psi) at 28 days using T 22 .....6.3.2.

Concrete pavement designs are often based on flexural strength, but consideration may be given to conducting acceptance testing based on compression cylinders. The compressive strength requirement can be established based on a correlation to flexural strength tests conducted during the mixture development and qualification stage. Acceptance criteria are addressed in Section 7 of AASHTO R 101.

Susceptibility to Slab Warping and Cracking

The susceptibility to moisture warping in concrete slabs is related to the volume change (shrinkage and expansion) resulting from changes in the moisture content of the concrete. As hardened concrete dries, the loss of moisture results in shrinkage (known as drying shrinkage). Moisture gradients naturally occur in slabs on the ground, in which the bottom of the slab tends to be wetter than the top, which can result in differential drying shrinkage that is responsible for upward warping. In turn, this produces increased stress and roughness, especially in more arid climatic zones (Asbahan and Vandenbossche 2011, Karamihas and Senn 2012, and Van Dam 2015).

In addition to drying shrinkage, concrete can also undergo volume change due to chemical shrinkage when water is consumed in hydration. However, chemical shrinkage is normally only of importance in concrete having a water-to-cementitious materials (w/cm) ratio of less than 0.40, which is not common in paving-grade concrete. If drying and autogenous shrinkage are restrained by embedded reinforcement, the frictional drag of a slab on grade, or load transfer devices at the joints, this restraint will result in the development of stress in the concrete. If this stress exceeds the tensile strength of the concrete, cracking will ensue.

Various strategies can be utilized to assess and minimize the shrinkage of concrete and its tendency to crack due to water-related volume change. These include the use of prescriptive measures based on minimizing paste volume or measuring unrestrained drying shrinkage using AASHTO T 160 or the use of performance measures based on an assessment of cracking tendency through ring testing such as that described in AASHTO T 334.

The volume of the paste shall not exceed 25 percent..... 6.4.1.1

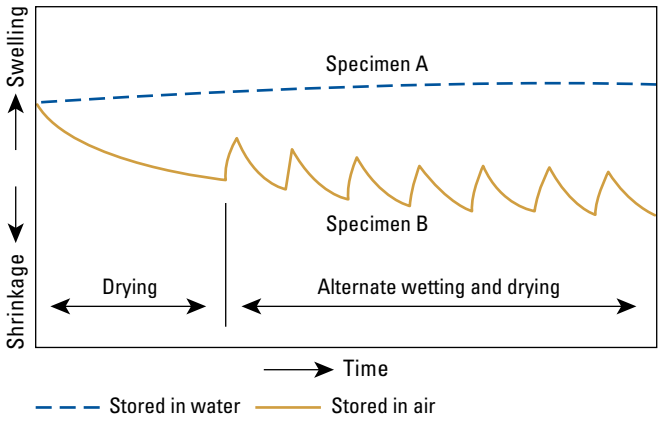
The volume of paste is determined by computing the volume of the liquid and binder phases in a cubic yard of concrete. The volume of the aggregate and air are not considered in this calculation.

Hardened concrete is primarily composed of aggregates held together by hydrated cement paste (HCP). The HCP is composed of hydration products created through chemical reactions between the cementitious materials (cement and SCMs) and mix water. These hydration products do not occupy all of the space that was once occupied by the cementitious materials and mix water, and therefore open space remains in the form of pores of various sizes. The largest pores (~1 mm to 10 mm or more) are created by entrapped air, while the next largest pores (~70 microns to 1 mm) are spherical air bubbles purposely entrained in the concrete to protect it against freeze-thaw damage. The pores between 0.01 and 5 microns in size, known as capillary pores, are left by unreacted water, and the smallest pores (≤ 0.5 to 10 nm), called gel pores, are intrinsic to the main hydration product, calcium silicate hydrate. The capillary pores have been proven to have a significant influence on concrete strength and permeability, and their volume and sizes also have an influence on the water-related volume changes that concrete undergoes with changes in moisture content. The gel pores also play a key role in these volume changes (Mindess et al. 2003).

As concrete dries, the pores begin to empty, starting with the largest pores first. As the capillary pores transition from being full to partially filled with water, menisci form due to surface tension. The menisci in millions of pores pull on the pores' sidewalls, drawing them inward ever so slightly. As drying continues, water empties from increasingly smaller pores, including the gel pores, leaving the pores partially filled. The menisci that form in smaller pores have much higher surface tension than the menisci that form in larger pores, resulting in higher stress pulling the pore walls inward. The net result of the water loss in pores of all sizes is that concrete shrinks as it dries, leading to the need for joints in jointed concrete pavements to accommodate the shrinkage.

As dry concrete is rewetted, the opposite occurs. Empty or partially empty gel and then capillary pores take up water, with the smallest accessible pores filling up first, until all pores are full. The stresses in the pores decrease and the concrete swells, but not to its initial size because permanent changes have occurred to the pores and microstructure of the HCP. Therefore, only a portion of the shrinkage is reversible. The net result is that once concrete has dried, it will never return to the original volume it occupied when placed as a result of resaturation alone.

Further, as the concrete undergoes repeated cycles of wetting and drying, the shrinkage due to drying continues to increase, as illustrated in Figure 1 (Kosmatka and Wilson 2016). In contrast, a specimen that is continually soaked actually undergoes slight expansion. Taking both shrinkage and expansion into account, it can be seen how concrete slabs that undergo drying and wetting at the surface over time but remain largely saturated on the bottom can develop upward curvature that will continue to increase for years to come (Asbahan and Vandenbossche 2011, Karamihas and Senn 2012).



Wilson and Tennis 2021, with information from Roper 1960; Courtesy of Portland Cement Association (PCA), used with permission

Figure 1. Effects of drying and wetting cycles on concrete shrinkage

As the description of drying and rewetting suggests, the key material property influencing the water-related volume changes of concrete is the nature and volume of the pores within the HCP. This property, in turn, is controlled by the volumes of the cementitious materials and mix water present in the concrete mixture. While many believe that the amount of mix water added to the concrete is the critical factor (Kosmatka and Wilson 2016), the volume of mix water is related to the overall cementitious materials content of the concrete through the w/cm ratio, and the combined volumes of the mix water and cementitious materials define the total volume of HCP in the concrete. It has been found in many concretes used for applications such as bridge decks that keeping the total paste volume below 28 percent can significantly reduce the cracking tendency of concrete (Zollinger et al. 2014). However, slipformed concrete pavements do not require the same fresh properties as concretes designed for pumping or finishing bridge decks or other commercial concretes. As such, it is recommended that the volume of paste in concrete pavement mixtures be no more than 25 percent, with lower volumes being desirable.

For a given w/cm ratio, reducing the cementitious materials content through increasing the aggregate volume will reduce the ultimate shrinkage of the concrete, not only because of the reduction in volume of the HCP but also because aggregates provide internal restraint against shrinkage. For paving-grade concrete, a total cementitious content of less than 325 kg/m³ (550 lb/yd³) is typically desirable for many reasons, including to reduce shrinkage. (Note that some state departments of transportation [DOTs] have reduced the cementitious content of their concrete paving mixtures below 300 kg/m³ [500 lb/yd³] with great success.) The mixture's workability can be maintained by optimizing the aggregate gradation. One way to do this is with a gradation technique called the Tarantula Curve (Cook et al. 2013). This technique provides guidance in grading the coarse and intermediate aggregate and the coarse and fine sand for a mixture. Guidance is also provided regarding the shape of the aggregate. Both the Box Test (AASHTO TP 137) and the Vibrating Kelly Ball (VKelly) Test (AASHTO T 403) may be useful for evaluating the workability of trial mixtures.

Water can also be reduced for a given cementitious materials content by reducing the w/cm ratio, but concerns exist that if the w/cm ratio drops significantly below 0.40, chemical and autogenous shrinkage (due to chemical shrinkage and self-drying of the paste as water is consumed in hydration) may become prominent (Kosmatka and Wilson 2016, ACI Committee 201 2016). Typically, a good range for the w/cm ratio of paving-grade concrete is between 0.40 and 0.42, although higher values may be suitable for paving in areas not subjected to freeze-thaw cycles or chemical deicers.

The unrestrained volume change should be less than 420 microstrain at 28 days drying as determined from T 160 .....6.4.1.2

Free concrete shrinkage is commonly measured in accordance with AASHTO T 160, Standard Method of Test for Length Change of Hardened Hydraulic Cement Mortar and Concrete. This relatively simple test measures the length change of concrete specimens due to various causes other than applied stress or temperature change and is particularly useful as a prescriptive test for comparative evaluation of concrete mixtures (Goodwin 2006). Three concrete prisms (76.2 by 76.2 by 280 mm [3 by 3 by 11 in.] with a maximum aggregate size of 25 mm [1 in.] and a gauge length of 254 mm [10 in.]) are cast and tested to obtain a single test result. After casting, the specimens are kept in molds and moist cured for 23.5 ± 0.5 hours and then demolded, at which time the initial length measurements are taken. This procedure ensures that any early-age volume changes due to chemical and/or autogenous shrinkage that may have occurred within the first 24 hours are not assessed (see Goodwin 2006, Kosmatka and Wilson 2016). Following the initial length measurement, the specimens are moist cured for 28 days.

To assess the drying shrinkage, the specimens are then stored in air at a temperature of 23.0 ± 1.7°C (73.4 ± 3°F) and a relative humidity of 50 ± 4 percent, and comparative length readings are taken at 4, 7, 14, and 28 days and after 8, 16, 32, and 64 weeks, unless otherwise specified.

The two major limitations of this test are that (1) the volume change within the first 24 hours (early age) is not assessed and (2) the specimens are unrestrained and therefore the effect of restraint on cracking is not measured. Regardless, the test provides valuable comparative information on the drying shrinkage characteristics of concrete and is commonly used as a prescriptive test to specify concrete for slab-on-grade applications and bridge structures.

The prescriptive limit of 420 με at 28 days drying is a conservative value for pavement applications because it is consistent with simulations for bridge decks by Radlińska and Weiss (2012). This number may be adjusted to reflect restraint values consistent with pavement applications and local climatic conditions.

The unrestrained volume change as determined from T 160 at an age of 91 days should result in a probability of cracking of less than 5, 20, or 50 percent, depending on the application. Three concrete prisms (76.2 by 76.2 by 280 mm [3 by 3 by 11 in] for 25 mm [1 in.] maximum aggregate size having a gauge length of 254 mm [10 in.]) are tested for a single test result. Suggested limits are 360 microstrain, 420 microstrain, or 480 microstrain, respectively 6.4.2.2.1

AASHTO T 160 is used to assess free concrete shrinkage due to drying that occurs after the first 24 hours from the time of casting. The suggested limits for the performance specification are based on a Monte Carlo simulation analysis to predict the probability of cracking due to drying shrinkage for bridge decks (Radlińska and Weiss 2012) and are therefore conservative for pavement applications. AASHTO T 160 provides an insufficient measure of total shrinkage for concrete with a w/cm ratio of less than approximately 0.40 because, as previously described, the test does not measure early-age chemical and autogenous shrinkage, which can be significant at lower w/cm ratios (Sant et al. 2006, Kosmatka and Wilson 2016).

Crack free at 180 days as determined using T 334 ..... 6.4.2.1.1

AASHTO T 334, Standard Method of Test for Estimating the Cracking Tendency of Concrete, is used to determine the cracking tendency of restrained concrete specimens. Unlike AASHTO T 160, in which the concrete prisms are not restrained, in AASHTO T 334 the concrete is cast around a steel ring that provides restraint as the concrete undergoes shrinkage while drying. The restraint generates stress, and if this stress exceeds the strength of the concrete, the ring will crack. As stated in AASHTO T 334, the procedure can be used to determine the effects of variations in the properties of concrete as these variations relate to the time to cracking of concrete when restrained. However, the procedure is comparative and is not intended to determine the time of initial cracking of concrete cast in a specific type of structure, such as a pavement.

The height of the AASHTO T 334 ring is 152 mm (6 in.). The inside steel ring has an outside diameter of 305 mm (12 in.) and a wall thickness of 12.7 ± 0.4 mm (½ ± 1/64 in.). Alternatively, the method allows for the use of a structural steel pipe conforming to ASTM A501 or ASTM A53/A53M for the inside steel ring. These specifications describe an extra-strong 12 in. pipe having an outside diameter of 324 mm (12¾ in.) and a wall thickness of 13 mm (½ in.). Four strain gauges are mounted equidistantly at mid-height along the inner surface of the inner steel ring. The outer ring can be made of a 6.4 mm (¼ in.) thick cardboard form tube (e.g., Sonotube) that has an inside diameter of 457 mm (18 in.). The base forms can be made of 0.6 by 0.6 by 0.016 m (24 by 24 by ⅝ in.) plywood or resin-coated (or polyethylene-coated) plywood sheets.

The outer forms are removed after 24 ± 1 hours of moist curing, and the specimens are moved to a room with a constant air temperature of 23 ± 2°C (73.5 ± 3.5°F) and a relative humidity of 50 ± 5 percent. Data acquisition equipment is used to automatically record measurements from each strain gauge independently, and the time and strain data from the strain gauges are recorded every 30 minutes. Every 2 to 3 days, the strain data are reviewed and the specimens are visually assessed for signs of cracking. A sudden decrease of more than 30 microstrain in one or more strain gauges usually indicates that cracking has occurred. The time of cracking, the length of cracking, and the width of the crack on the exterior radial face of the specimen are recorded.

The limit provided in AASHTO R 101 is based on limits set for bridge decks and is considered conservative for pavement applications.

The mixture should have a stress less than 60 percent of the splitting tensile strength when tested in the dual ring (T 363) with a temperature of 23 ± 1°C (73.4 ± 1.8°F), and RH of 50 ± 2 percent RH for 7 days 6.4.2.1.2

The specimen configuration in this test method is similar to that of AASHTO T 334 except that there is both an inner and outer steel ring. The temperature of the sample can also be reduced after a given time to obtain a result in a reasonable timeframe.

Performance in Freeze-Thaw Environments

Freezing and Thawing

In many climates, concrete pavements are subjected to multiple annual cycles of freezing and thawing. Below a certain level of saturation (critical saturation), HCP will not be damaged by these freeze-thaw cycles because the larger pores in the HCP remain empty and provide adequate space to accommodate the hydraulic, osmotic, and crystallization pressures that develop due to ice formation. However, if this same concrete undergoes freezing and thawing while in a critically saturated state (above approximately 85 percent saturation), damage will occur within a few cycles, irrespective of air-void volume (Jones et al. 2013).

As concrete freezes, ice first begins to form within the larger pores in the HCP. The temperature at which this occurs depends both on the sizes of these pores and on the chemistry of the pore solution within them. The formation of ice is expansive (an approximate expansion of 9 percent occurs when liquid water transitions to ice) and results in changes in the pore solution chemistry that, in turn, result in the generation of stress within the concrete (Powers 1945, Powers 1954, Powers 1955, Powers and Helmuth 1953, Marchand et al. 1995, Penttala 1998, and Scherer and Valenza 2005).

It is known that the freeze-thaw durability of HCP in concrete is influenced by the sizes and volume of the air bubbles entrained in the concrete and the concrete's permeability (ACI 2016, Kosmatka and Wilson 2016). Combined, these factors strongly influence the rate at which and the degree to which concrete becomes saturated when exposed to a fluid such as water. The role of entrained air voids in protecting concrete against freeze-thaw damage is to provide additional void space that remains largely unsaturated even as the concrete's gel and capillary pores become saturated. The presence of these unsaturated voids reduces the total degree of saturation below critical levels (Todak et al. 2015).

Additionally, the w/cm ratio directly impacts the volume and sizes of the capillary pores in the concrete, as well as the transport properties that influence the rate at which fluid is absorbed. Concrete made with a high w/cm ratio has a higher initial degree of saturation than concrete made with a low w/cm ratio (Todak et al. 2015). Furthermore, concrete made with a high w/cm ratio and thus a greater amount of connected pores has a high rate of absorption and becomes saturated more quickly than concrete with a dense, disconnected pore structure. Therefore, concrete with a high volume of entrained air and a low w/cm ratio can take a very long time to reach critical saturation, whereas concrete with a low volume of entrained air and a high w/cm ratio may quickly reach critical saturation.

A w/cm of 0.45 is the maximum recommended for use for concrete that will be exposed to moisture in a freeze-thaw environment (ACI 2016). General recommendations are to use a w/cm between 0.40 and 0.45.....6.5.1.1

Air content between 5 and 8 percent using T 152, T 196, or TP 118. .....6.5.1.2

As discussed above, the amount and spacing of the air entrained in concrete are key factors in protecting the HCP from freeze-thaw damage. The required air content to protect the HCP is dependent on both the freeze-thaw exposure conditions and the paste content (or mortar fraction) in the concrete. While it is possible to specify air content based on the paste volume in the concrete, specifications often simply recommend a total air content of 5 to 8 percent. AASHTO R 101 uses the maximum air content of 8 percent, but mixtures can contain a higher air content than this if adequate strength is obtained. Higher air contents may be necessary at times because some modern mixture combinations and construction practices produce air-void systems with coarse air voids that do not contribute significantly to freeze-thaw durability. In some cases, air contents as high as 9 percent have been deemed necessary to provide adequate freeze-thaw durability (Felice et al. 2014, Ley et al. 2016).

Common test methods used to assess the air content in fresh concrete have focused almost exclusively on measuring the total volume of air in the mixture and not on the size and distribution of the air voids. Standard tests (or variations thereof) used by state highway agencies include the following:

- AASHTO T 152, Standard Method of Test for Air Content of Freshly Mixed Concrete by the Pressure Method. A shortcoming of this method is that the air void volume within the aggregate influences the total air void volume measured by the test because water is forced into the pores in the aggregate. The results can be corrected in many cases by using an aggregate correction factor, but this test is not typically suitable for concrete that uses lightweight or highly porous aggregate.

- AASHTO T 196, Standard Method of Test for Air Content of Freshly Mixed Concrete by the Volumetric Method. This test method does not require the use of a correction factor to account for the voids within the aggregate and therefore is commonly used to examine mixtures with lightweight aggregate.
- AASHTO T 121, Standard Method of Test for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete. The accuracy of this test is impacted by the accuracy of the batching process and the water content of the mixture. Air volume variations of up to 3 percent can occur under allowable batch weight tolerances. This test is commonly used during laboratory mixture proportioning and not for field acceptance.

Note that many factors can affect air content and the quality of the air-void system in the field, including mixing duration, retempering, temperature, admixtures, SCMs, and so on. These factors are discussed in Kosmatka and Wilson (2016) and Taylor et al. (2019).

Air content greater than 4 percent and SAM number less than 0.20 using TP 118 [now formalized as T 395]..... 6.5.1.3

As discussed above, the freeze-thaw durability of concrete is closely linked to the sizes, distribution, and volume of the air bubbles entrained in the concrete (ACI 201 2016, Kosmatka and Wilson 2016). Air-entraining admixtures (AEA) (AASHTO M 154) are added during concrete mixing to create stable air bubbles in the fresh concrete that remain once the concrete has hardened. Most entrained air bubbles range in size from 10 to 500 microns in diameter (Kosmatka and Wilson 2016) and, ideally, are uniformly dispersed throughout the HCP. Because direct measurement of the size and distribution of the air voids in concrete by ASTM C457, Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete, is difficult and time-consuming, it is common to specify the required total volume of air that will generally provide the needed bubble sizes and spacing. This is the strategy espoused in Section 6.5.1.2 of AASHTO R 101.

It is recognized that simply specifying the total air content is not necessarily adequate, and therefore an alternative test that is able to indicate the quality of the air-void system in fresh concrete is desirable. The most common method to evaluate the air-void system in concrete is to view a polished concrete slab under a microscope in accordance with ASTM C457. In addition to calculating the total volume of air in the concrete, ASTM C457 estimates the average spacing of the air bubbles within the concrete. This parameter is called the spacing factor (Powers 1954). The ability of the concrete to resist freeze-thaw damage increases as the spacing factor decreases (i.e., as the air voids become more closely spaced). ACI Committee 201 (2016) suggests a desired maximum spacing factor near 0.200 mm (0.008 in.) to ensure adequate freeze-thaw resistance for concrete subjected to moderate exposure conditions, stipulating that this value can be higher for mild exposure and may need to be lower for concrete exposed to deicing chemicals.

Because ASTM C457 can only be conducted on hardened concrete by a trained technologist, alternative methods are desirable. A recommended approach to overcome this shortcoming is to correlate the observations made on hardened concrete with the results from more common tests, such as AASHTO T 152 or AASHTO T 196, conducted on plastic concrete for the specific mixture under consideration. However, the limitation of this approach is that conditions may change during construction, rendering the correlation established in the laboratory no longer valid. This can be problematic because certain combinations of concrete-making materials can result in mixtures that have air contents that meet specifications but have air-void distributions that may not protect the concrete in severe freeze-thaw environments (Freeman 2009, Felice et al. 2014, Ram et al. 2012, Ley et al. 2017).

An alternative method has been developed to address this shortcoming that uses a device called the Super Air Meter (SAM). The SAM method has been standardized under AASHTO T 395 (formerly AASHTO TP 118), Standard Method of Test for Characterization of the Air-Void System of Freshly Mixed Concrete by the Sequential Pressure Method. The SAM is a modified version of the AASHTO T 152 pressure meter (Ley et al. 2017), but instead of using a single testing pressure, as is used in AASHTO T 152, the SAM uses sequential pressures to not only determine the volume of total air but also infer the quality of the air-void system using a term called the SAM number.

The SAM number has been correlated to the air-void spacing factor obtained through ASTM C457 and the durability factor of concrete as assessed through AASHTO T 161. Ley et al. (2017) found that the SAM number correlates better with the AASHTO T 161 durability factor than with the ASTM C457 spacing factor. Furthermore, round robin testing has shown that the variation in the SAM results is similar to the variation in the results from ASTM C457 and ASTM C666 (Ley et al. 2017).

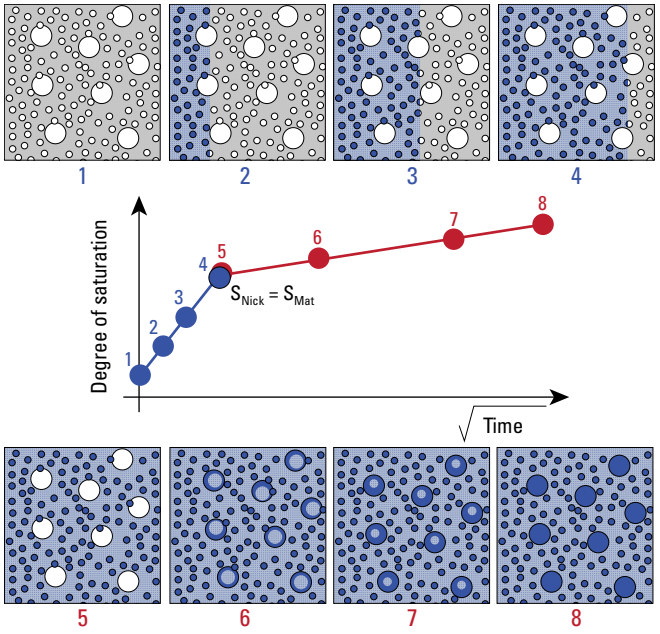
In addition to a SAM number of less than 0.20, AASHTO R 101 recommends that an air content of at least 4 percent be provided in the concrete. This value was chosen to ensure that a minimum air content is included in the concrete mixture. While air contents as low as 3 percent have been shown to provide adequate freeze-thaw durability in ASTM C666 (Felice et al. 2014, Ley et al. 2017), 4 percent was chosen for AASHTO R 101 to provide a factor of safety.

A calculated time to saturation of greater than 30 years 6.5.2.1

Based on the information discussed above regarding air content, the recommended mixture proportions should aim to achieve a calculated time to saturation of greater than 30 years, as described in Section 6.5.2.1 of AASHTO R 101.

As mentioned above, the freeze-thaw durability of HCP in concrete is influenced by the sizes and volume of the air bubbles entrained in the concrete and the concrete's permeability (ACI 2016, Kosmatka and Wilson 2016). This influence is illustrated in Figure 2, which shows eight stages of saturation (Weiss 2014).

When an unsaturated concrete is exposed to a fluid on one side, as would occur in a standard sorptivity test such as ASTM C1585, the fluid is first absorbed into the smallest pores, which include the gel pores and capillary pores. The wetting front progresses through the sample, saturating these small pores and bypassing the larger entrained and entrapped air voids, as shown in Stages 2 through 5 in Figure 2. At Stage 4, nearly all of the gel and capillary pores have become saturated, whereas the entrained and entrapped air voids remain empty. The line drawn through points 1 to 5 in Figure 2 defines the initial rate of absorption, which normally takes 10 to 18 hours under standard test conditions when the fluid is water (Todak et al. 2015).

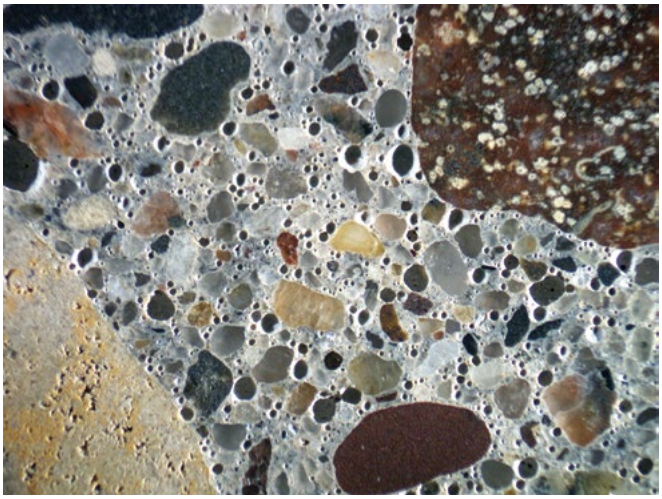


Recreated from Weiss 2014

Figure 2. Water absorption into concrete during sorptivity test

As fluid continues to be adsorbed, the larger entrained and entrapped air voids become progressively saturated, as illustrated in Stages 6 to 7 in Figure 2, until complete saturation occurs at Stage 8. The line drawn through points 5 to 8 in Figure 2 illustrates the secondary rate of absorption, which can take months to years before Stage 8 is reached (Todak et al. 2015).

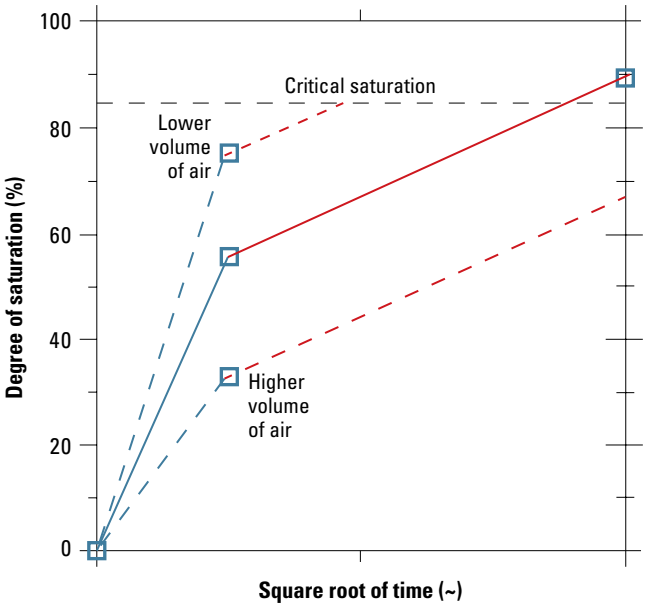
A network of uniformly dispersed entrained air bubbles (such as that shown in Figure 3) can provide the needed empty space to relieve the pressures generated as the concrete freezes. The entrained and entrapped air bubbles are the last spaces to become saturated in concrete, remaining largely unsaturated under normal service conditions. It is widely recognized that concrete that is saturated below a critical level will not be damaged by freeze-thaw cycles because enough of the larger pores in the concrete are empty and provide adequate space to accommodate the hydraulic and osmotic pressures that develop as ice forms. However, if this same concrete undergoes freezing and thawing in a critically saturated state (a saturation value somewhere between 75 to 91 percent, with the most recent research pointing to a value of around 86 percent), damage will occur even in a single freeze-thaw cycle, irrespective of air-void volume (Fagerlund 1977, Yang et al. 2007, Li et al. 2012).



Length of field scale: left to right = 0.49 in.; top to bottom = 0.37 in.

Ron Sturm, Braun Intertec, used with permission

Figure 3. Stereo micrograph of entrained air voids (spherical bubbles) in hardened concrete, with larger, irregular entrapped air voids



Adapted from Todak et al. 2015

Figure 4. Effect of the volume of entrained air and w/cm ratio on the degree of saturation during a sorptivity test

As shown in Figure 4, increasing the volume of entrained air decreases the overall degree of saturation at the inflection point that occurs when the gel and capillary pores become saturated (Stage 5 in Figure 2). This increase in entrained air reduces the degree of saturation below critical levels and enhances the freeze-thaw durability of HCP (Todak et al. 2015).

The w/cm ratio directly impacts the volume and sizes of the capillary pores in the concrete and impacts the rate at which fluid is absorbed. Concrete made with a high w/cm ratio has a higher initial degree of saturation than concrete made with a low w/cm ratio (Todak et al. 2015). Furthermore, concrete made with a high w/cm ratio and having well-connected pores has a high rate of absorption and becomes saturated more quickly than concrete with a dense, disconnected microstructure. Therefore, concrete with a high volume of entrained air and a low w/cm ratio can take a very long time to reach critical saturation, whereas concrete with a low volume of entrained air, a high w/cm ratio, and well-connected pores may quickly reach critical saturation.

The basis for the performance specification in AASHTO R 101—a time to saturation of greater than 30 years—is that concrete will fail when it is critically saturated and freezes. A model is used to predict the time required for the concrete to reach a critical degree of saturation. The relationship between the time to critical saturation in years and the volume and quality of the entrained air and the w/cm ratio is demonstrated in the equation below, which computes the time to reach critical saturation for a given concrete mixture based on the results of ASTM C1585 (bucket test):

$$S(t) = S_{Nick} + \phi S_2 \sqrt{t} \leq DOS_{Critical} \quad \textbf{(1)}$$

where

- S_{Nick} = degree of saturation for a concrete whose gel and capillary pores are filled. This is commonly referred to in ASTM C1585 as the nick point. While this can be determined by ASTM C1585, it can also be computed directly from the mixture proportions.
- ϕ = a factor associated with aggressive moisture exposure. This should be taken as 1.0 to represent continuous contact with water but may be increased to 1.3 if the concrete is in an area where deicing salt is used.
- S_2 = slope of the line formed by the degree of saturation (DOS) versus $t^{0.5}$ after the nick point saturation, determined in accordance with ASTM C1585.
- $DOS_{Critical}$ = critical degree of saturation for a concrete at which point damage will begin to occur. This can initially be assumed to be 0.85 for a concrete with sufficient air-void spacing. This value varies slightly with SAM number; however, a SAM number of 0.20 corresponds to a $DOS_{Critical}$ value of 0.85, with greater SAM numbers corresponding to lower values for the critical degree of saturation required to cause failure.

Adding to the harshness of the freeze-thaw environment is the application of chemical deicers. Applied to improve the safety of the roadway, many deicers can negatively impact concrete durability. Chemical deicers can physically affect concrete by increasing the degree of saturation to levels that approach or exceed damaging critical saturation (Spragg et al. 2012), amplifying osmotic pressure through changes in the HCP pore solution chemistry, generating thermal shock due to the thermodynamics of melting ice, and contributing to the development of salt crystallization pressures, among other factors (Mindess et al. 2003, Kosmatka and Wilson 2016). These factors can contribute to the development of deicer-related scaling of the pavement surface.

Chemical Attack by Deicing Salts

It is recognized that the use of some chemical deicers may initiate chemical deterioration mechanisms in concrete. These mechanisms are gaining attention due to the increased frequency and severity of joint deterioration linked to the increased use of aqueous solutions of calcium chloride ($CaCl_2$) and magnesium chloride ($MgCl_2$) for pavement deicing (Sutter et al. 2008, Weiss and Farnam 2015).

The resulting pavement distress often first appears as darkening of the concrete adjacent to the pavement joints (referred to as "shadowing"), which is attributed to water being trapped in microcracking near the joints. Over time, this microcracking may progress into disintegration of the concrete and loss of material. The stages of deterioration are shown in Figure 5. This manifestation of chemical deterioration is commonly called joint deterioration (Taylor et al. 2012, Weiss and Farnam 2015).

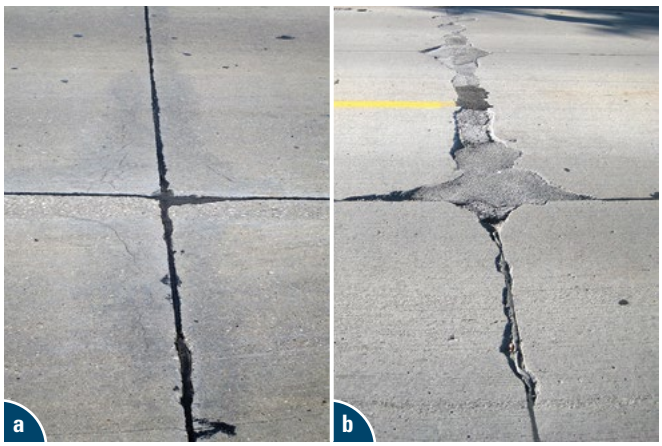


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Figure 5. Joint deterioration: (a) "shadowing" at pavement joints and (b) eventual disintegration of concrete at the joints

The primary mechanism thought to be responsible for chemical deicer attack is a phase change that occurs in the presence of water, in which the calcium hydroxide present in typical HCP reacts with $CaCl_2$ to form calcium oxychloride (CAOXY) (Sutter et al. 2006, Sutter et al. 2008, Weiss and Farnam 2015, Monical et al. 2015). The $CaCl_2$ is a component of some deicers or a product of chemical reactions that occur between the $MgCl_2$ found in certain other deicers and the calcium compounds in HCP (Sutter et al. 2008).

The phase change to calcium oxychloride is highly expansive, with the resulting damage to the HCP likely due to crystallization pressures. Sutter et al. (2008) investigated the effect of this expansive reaction on cylinders of mortar soaked in solutions of 17 percent $CaCl_2$ and 15 percent $MgCl_2$ at 4.4°C (40°F) (i.e., above freezing) for 150 days. Scanning electron microscopy showed that the distress that occurs in specimens soaked in $CaCl_2$ and $MgCl_2$ solutions is characterized by radial cracking from the outside of the cylinders inward, as if the surface were peeling away like layers of an onion. Furthermore, the reaction alters the affected areas of the HCP, rendering them more porous and nearly devoid of calcium hydroxide (Sutter et al. 2008).

The phase change to calcium oxychloride occurs above the freezing point of pure water at temperatures between 0°C and 50°C (32°F and 122°F), with the salt concentration in the solution determining the specific temperature at which the phase change occurs (Weiss and Farnam 2015). At low salt concentrations, the chemical reaction may not occur at all, and any distress that occurs is largely due to physical freeze-thaw deterioration. As the salt concentration increases, the chemical attack mechanism may become an important, if not the dominant, factor contributing to distress.

The importance of salt concentration in solutions of $CaCl_2$ and $MgCl_2$ cannot be overstated. Deicers are applied at concentrations in excess of what is needed for the calcium oxychloride phase change to occur. When $CaCl_2$ and $MgCl_2$ solutions are used in anti-icing operations (that is, when solutions are applied prior to but in anticipation of a precipitation event), they remain at full concentration until becoming diluted as the precipitation event occurs, if it occurs at all. Furthermore, anti-icing solutions are often applied when the pavement surface is dry prior to the start of the event, and therefore the fully concentrated salt solutions can be drawn into the unsaturated concrete through sorption. Penetration is especially high if the solution concentrates in the joints.

When deicers are applied during or after a precipitation event, the melting of ice dilutes the solution and decreases the potential for an occurrence of a deleterious chemical reaction. However, salts may concentrate over time if the solution is retained in the joints and the deicing cycle is repeated, resulting in the potential for a damaging chemical reaction.

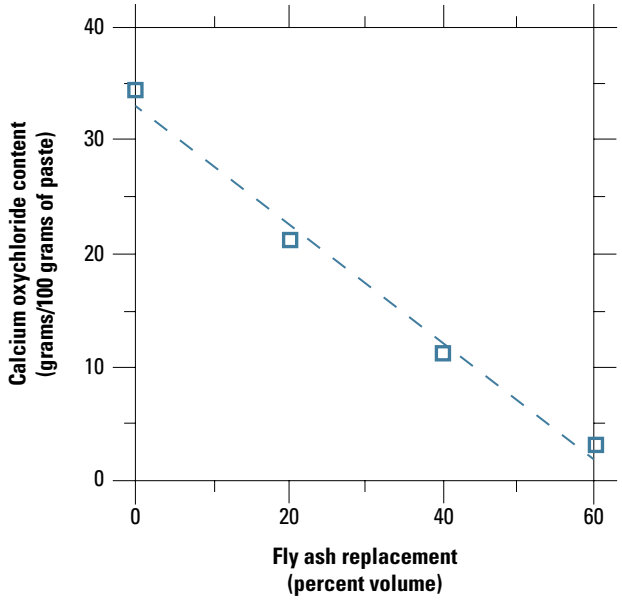
Guidance to address conventional physical freeze-thaw deterioration in the presence of deicers is well documented in *ACI PRC-201.2R-16: Guide to Durable Concrete* (ACI Committee 201 2016), and some measures described above include the use of a relatively low w/cm ratio (0.45 or lower) and the entrainment of an air-void system capable of relieving the stresses generated as ice forms. To avoid physical freeze-thaw damage at joint locations, it is essential that the concrete not become critically saturated. Critical saturation can be minimized through design, construction, and maintenance strategies that (1) reduce the ingress of water-salt solutions into the joints and (2) rapidly drain these solutions from the joints through the base course. These strategies will help avoid critical saturation of the concrete that can lead to freeze-thaw damage in even a few cycles.

Recommendations for addressing chemical deicer distress are provided in the following sections.

SCM should be used to replace the cement with a volume of at least 30 percent 6.5.3.1

Recommendations for addressing chemical deicer distress are still under development, but research has focused on the need to (1) reduce the amount of calcium hydroxide present in the HCP and (2) decrease the ingress of salt solution into the concrete. The use of SCMs both reduces the amount of calcium hydroxide in the HCP and decreases the transport properties of the concrete, thereby reducing salt solution ingress. The ability of SCMs to reduce the amount of calcium hydroxide is partially due to dilution, in which the portland cement is replaced by SCMs, and partially due to the consumption of calcium hydroxide through the pozzolanic reaction that occurs with certain SCMs (e.g., fly ash, slag cement, and silica fume) (Suraneni et al. 2016). Reducing the amount of calcium hydroxide in the HCP results directly in a reduction in the formation of calcium oxychloride, as illustrated in Figure 6.

A topical treatment (sealer) should be used consistent with M 224 .....6.5.3.2



After Suraneni et al. 2016

Figure 6. Average calcium oxychloride contents for a wide range of binders as a function of fly ash (SCM) replacement level

Another method for reducing the ingress of salt solution into concrete is the use of some type of barrier, either applied as a topical treatment (sealer) or created through carbonation. Research has suggested that the application of certain penetrating sealants (e.g., silanes, siloxanes, soy methyl ester polystyrene, and others) directly to concrete joints may provide a barrier to the ingress of chemical deicers and decrease the rate of ingress of outside chemicals (Sutter et al. 2008, Weiss and Farnam 2015).

The calcium oxychloride should be determined to be less than 15 g CAOXY/100 g cementitious paste as determined in accordance with T 365 .....6.5.4.1

A test method based on the use of a low-temperature differential scanning calorimeter (LT-DSC) can evaluate the potential reactivity between HCP and a salt solution by measuring the heat associated with calcium oxychloride formation (Weiss and Farnam 2015, Monical et al. 2015). This method provides a means for optimizing the composition of the cementitious materials in terms of reducing the potential for the formation of calcium oxychloride, as shown in Figure 6. Specific portland cement and SCM systems can be tested, making this method a practical screening tool for selecting materials for use under anticipated deicing conditions.

Transport Properties

The transport of fluids and gases into concrete is a key aspect of concrete durability because all durability-related distresses involve the transport of moisture—and at times other substances, such as chloride ions responsible for corrosion or sulfate ions responsible for external sulfate attack—into concrete. The transport properties of concrete and the test methods employed to assess them are described in several documents, including Stannish et al. (2000), Hearn et al. (2006), and Weiss (2014). The following test methods have been employed to assess transport properties in paving concrete in the United States:

- ASTM C1585, Standard Test Method for Measurement of Rate of Absorption of Water by Hydraulic Cement Concrete
- AASHTO T 277, Standard Method of Test for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration
- AASHTO T 358, Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration
- AASHTO TP 119, Standard Method of Test for Electrical Resistivity of a Concrete Cylinder Tested in a Uniaxial Resistance Test

ASTM C1585 is a relatively simple absorption test in which the degree and rate of water absorption (I) into a conditioned thin concrete sample (50 mm thick by 100 mm diameter [2 in. thick by 4 in. diameter]) is measured at specified intervals for a minimum of 8 days.

The rate of water absorption (mm/$s^{½}$) is defined as the slope of a best fit line of I plotted against the square root of time ($s^{½}$). It is typically observed that the slope makes a definitive change at some point, and therefore two absorption values are defined: the initial absorption and the secondary absorption. The absorption can be converted to degree of saturation (S), defined as the ratio of the absolute volume of absorbed water to the total volume of water-accessible pores. The degree of saturation at the intersection between the initial and secondary absorption is related to the point at which the capillary pore system becomes saturated (Weiss 2014).

Since around 2010, AASHTO T 277 has gained widespread acceptance by many highway agencies, especially for testing bridge deck concrete. AASHTO T 277 involves measuring the total charge passed by an applied voltage of 60 VDC in 6 hours across a 50 mm thick by 100 mm diameter (2 in. thick by 4 in. diameter) concrete specimen that has been placed between sodium hydroxide (NaOH) and sodium chloride (NaCl) solutions. The test measures the conductivity of the saturated concrete, including the effects of all dissolved ions (Hearn et al. 2006). The results of the test in coulombs are used to make a general assessment of the chloride ion penetrability of the concrete that is comparable to the assessment made by ASTM C1543 (bulk ponding test). Table 1 shows the typical specified values for AASHTO T 277. Note that the assessment is not specific, but rather the chloride ion penetrability is assigned a qualitative rating.

Although this test has been embraced by many state highway agencies (SHAs) due to its ease of use, it suffers several limitations, including the following (Stannish et al. 2000):

- The current passed is related to all ions in the pore solution, not just chloride ions.
- The measurements are made before a steady-state migration is achieved.
- The temperature of the specimen increases due to the applied voltage.

Several highway agencies have recognized the limitations inherent in AASHTO T 277 and are investigating the adoption of the surface resistivity method, described in AASHTO T 358, as an alternative (Rupnow and Icenogle 2011, Tanesi and Ardani 2012, Kevern et al. 2015). The surface resistivity test evaluates the electrical resistivity of water-saturated concrete, providing a rapid means to assess the concrete's ability to resist moisture penetration. In the studies cited above, a correlation was found between surface resistivity and the results of other electrical indicator tests, including AASHTO T 277. AASHTO T 358 takes approximately 5 minutes to conduct, does not involve the use of chemicals, and is generally easier to conduct than AASHTO T 277. The surface resistivity test is also nondestructive; thus, test specimens can subsequently be used for other testing. For these reasons, AASHTO T 358 is gaining in popularity.

Table 1. Chloride ion penetrability based on charge passed

Charge Passed (coulombs)	Chloride Ion Permeability
>4,000	High
2,000–4,000	Moderate
1,000–2,000	Low
100–1,000	Very Low
<100	Negligible

In AASHTO T 358, the resistivity of saturated concrete cylindrical specimens (100 mm diameter by 200 mm long, or 150 mm diameter by 300 mm long [4 in. diameter by 8 in. long, or 6 in. diameter by 12 in. long]) is measured using a four-probe Wenner array. An AC potential difference is applied in the outer probes of the Wenner array, generating current flow in the concrete. The two inner probes measure the potential difference generated by this current, from which the resistivity of the concrete is calculated. The resistivity, in Ω-cm, has been related to the resistance of the specimen to chloride ion penetration.

Although AASHTO T 358 is conducted more easily and cheaply than AASHTO T 277, it suffers many of the same limitations, in that the results are dependent on the sample geometry, the testing temperature, the degree of saturation, and the storage protocol used for the sample (Spragg et al. 2012, Weiss 2014).

AASHTO TP 119 operates according to the same principles as AASHTO T 358, except the resistivity is measured uniaxially along the length of the cylinder, end to end. Although AASHTO TP 119 can be conducted more easily and cheaply than AASHTO T 277, it suffers many of the same limitations. As is true for AASHTO T 358, work is underway to address the limitations of AASHTO TP 119 by normalizing the results of all electrical tests through the development of standards based on the saturated formation factor (F factor), which is directly related to the volume and connectivity of the concrete pores as well as the conductivity of the pore solution (Weiss 2014).

The w/cm ratio shall be: Less than 0.50 if concrete not subjected to freezing and thawing or deicer application or less than 0.45 if concrete subjected to freezing and thawing or deicer application.....6.6.1.1

The relationship between w/cm ratio and concrete transport properties is well established (ACI 2016, Kosmatka and Wilson 2016). In general, as discussed above, using a w/cm ratio below 0.45 improves the properties of concrete, especially if the concrete will be in a freeze-thaw environment (ACI 2016). The general recommendation is to use a w/cm ratio between 0.40 and 0.45 under such conditions. A slightly higher w/cm ratio is acceptable under non-freeze-thaw conditions if all other requirements are met.

The saturated F factor value (as determined using T 277 or electrical resistivity based on T 358 or TP 119) should be identified as meeting the requirements below for the following exposure conditions: Concrete not subjected to freezing and thawing or deicer application: saturated F factor greater than 500 or Concrete subjected to freezing and thawing and deicer application: saturated F factor greater than or equal to 1,000..... 6.6.1.2

Roughly equivalent saturated F factor, rapid chloride permeability test (RCPT), and resistivity values are provided in Table 2. These values should be achieved within 91 days of curing at 73°F (23°C) or after curing at 73°F (23°C) for 3 days followed by 25 days of curing at 122°F (50°C).

Table 2. Saturated F factor, RCPT, and resistivity values

Chloride Ion Penetrability	Highest Saturated Formation Factor [a]	Lowest Saturated Formation Factor [a]	Minimum Charge Passed at 6 Hours [b]	Maximum Charge Passed at 6 Hours [b]	Highest Resistivity [c]	Lowest Resistivity [c]
	~	~	Coulombs	Coulombs	Ω M	Ω M
High	500	~	4,000	~	50	~
Moderate	1,000	500	2,000	4,000	100	50
Low	2,000	1,000	1,000	2,000	200	100
Very Low	20,000	2,000	100	1,000	2,000	200
Negligible	~	20,000	0	100	~	2,000

Notes:

[a] A conversion between saturated formation factor, RCPT, and resistivity is provided assuming a pore solution resistivity of 0.10 W m. More accurate methods for determining the pore solution could alternatively be used. Specifications should designate the acceptable methods permitted for determining the pore solution.

[b] As determined from ASTM 1202-12.

[c] As determined based on first principles (Spragg et al. 2012) using AASHTO T 358 or AASHTO TP 119 (assuming a saturated sample).

Agencies need to select a chloride ion penetrability criterion that is based on local climatic conditions and locally available materials. For most applications, a chloride ion penetrability of at least "moderate" should be sought.

A saturated F factor (as determined from electrical resistivity based on T 358 or TP 119) is specified that is related to the desired depth of ionic penetration for a given ion exposure during a specified service life..... 6.6.2.1

The specified saturated F factor value is selected based on the desired service life and exposure conditions of the concrete pavement in accordance with Table 3.

Saturated F factor values should be achieved within 91 days of curing at 73°F (23°C) or after curing at 73°F (23°C) for 3 days followed by 25 days of curing at 122°F (50°C).

Table 3. Saturated F factor limits

Exposure Conditions	Saturated F Factor Limits	
	Desired Service Life (Years)	
	25–35	>35
Non-Freeze-Thaw and No Deicers	>500	>1,000
Freeze-Thaw and Deicer Exposure	>1,000	>2,000

Aggregate Stability

Aggregates typically occupy 60 to 75 percent of the concrete volume (Kosmatka and Wilson 2016), though the volume of aggregates in paving-grade concrete has been trending upward to minimize cement paste content (Taylor and Fick 2015). Ideally, the aggregates used in concrete are clean, hard, strong, and durable and will contribute positively to the economy, constructability, and long-term performance of the concrete pavement.

Unfortunately, several mechanisms can affect the stability of certain aggregates in concrete, particularly when the concrete is subjected to a harsh environment that includes freezing and thawing under saturated conditions and exposure to chemical deicers (Taylor and Wang 2015). For this reason, careful screening of potential aggregates for use in paving-grade concrete is required to achieve resistance to both damage from cyclic freezing and thawing as well as damaging reactions that may occur between the aggregate and the highly alkaline concrete pore solution.

The following sections discuss aggregate-related issues that may arise in concrete pavements.

D-Cracking

Evaluate aggregates for the potential susceptibility to damage due to freezing and thawing (D-Cracking) using T 161 (ASTM C666), or accepted local SHA practice.....6.7.1

Certain coarse aggregates, when subjected to repeated cyclic freezing and thawing in a critically saturated state, fracture and/or dilate, resulting in the deterioration of concrete. This phenomenon is commonly referred to as D-cracking in pavements. D-cracking is initially visible as a series of fine cracks generally running parallel to joints, cracks, or the free edges of the slab. As the number of freeze-thaw cycles accumulates, spalling and deterioration of these cracks occur at the surface. A dark staining due to microcracking and calcium hydroxide or calcium carbonate residue leaching to the surface generally precedes and accompanies the cracking. D-cracking often appears in the shape of an hourglass on the pavement surface at affected joints and cracks (Van Dam et al. 2002). Figure 7 shows a jointed concrete pavement suffering severe aggregate freeze-thaw deterioration.

Jim Grove, ATI Inc./FHWA, used with permission

Figure 7. Jointed concrete pavement suffering severe aggregate freeze-thaw deterioration

Aggregates that are typically identified as being susceptible to D-cracking tend to fracture and/or dilate as they freeze, resulting in cracking of the surrounding mortar. It has also been hypothesized in some cases that the expulsion of water during freezing contributes to the dissolution of soluble paste components, such as calcium hydroxide, in the interfacial zone (Van Dam et al. 2002). Most susceptible aggregates are of sedimentary origin and are commonly composed of limestone, dolomite, or chert (Stark 1976), with limestone aggregates found in a band running from Kansas to Michigan being particularly susceptible (Taylor and Wang 2015). In addition to composition, aggregate properties related to susceptibility include the nature of the pore structure, adsorption characteristics, permeability, and size of the aggregate particle (Schwartz 1987, Kosmatka and Wilson 2016).

Generally, susceptible aggregates have a high total porosity and medium-sized pores (0.1 to 5 microns), which allows for saturation of a significant volume of water in the freezable pore space (Kosmatka and Wilson 2016). The critical size at which an aggregate will fail is dependent upon many factors, but in general the aggregate's susceptibility to freeze-thaw deterioration decreases as the particle size is reduced.

The most effective means of preventing aggregate freeze-thaw deterioration is to prohibit the use of susceptible aggregates by specifying that coarse aggregates pass certain freeze-thaw requirements. Highway agencies experiencing aggregate freeze-thaw damage have developed screening protocols that are almost exclusively based on variations of AASHTO T 161, Standard Method of Test for Resistance of Concrete to Rapid Freezing and Thawing. The concrete specimens tested in AASHTO T 161 must be rigorously prepared and cured using a procedure such as ASTM C1646/C1646M, Standard Practice for Making and Curing Test Specimens for Evaluating Resistance of Coarse Aggregate to Freezing and Thawing in Air-Entrained Concrete; otherwise, considerable variability may be introduced in the test results.

In the aggregate screening protocol used by many highway agencies, ASTM C1646/C1646M provides the standard requirements for preparing concrete specimens whose aggregates will be evaluated for susceptibility to damage resulting from cyclic freezing and thawing. Specimens prepared in accordance with ASTM C1646/C1646M are then tested according to AASHTO T 161, and the potential for D-cracking is assessed based on changes in dynamic modulus, linear expansion, or weight loss (Kosmatka and Wilson 2016). In this testing methodology, concrete beams are prepared with the aggregate under evaluation and subjected to rapid freezing and thawing cycles. In Procedure A, the specimens are frozen and thawed in water, whereas in Procedure B the specimens are frozen in air but thawed in water. Procedure A is the preferred method for aggregate screening (TRB 2013).

This combination of two testing standards, one for specimen preparation and one for the testing sequence, is the most widely used test method for evaluating the freeze-thaw resistance of aggregates. This method has been widely adopted as an aggregate screening tool, often with slight modification, by several Midwestern state highway agencies to address their specific needs and observations.

The main criticism of the test method is that it is not representative of actual field conditions. The concrete is subjected to rapid freezing and thawing in a saturated state, which is not likely to occur in the field. Therefore, although the test is able to rank aggregate from excellent to poor, it is not clear that the test can be used reliably to predict the field performance of marginal aggregate. Because the test is more severe than actual field conditions, aggregates that pass this test will generally perform well in the field, but aggregate sources that have demonstrated good field performance may be rejected.

The potential for freeze-thaw deterioration is most often assessed through the linear expansion of the specimen or through the reduction in the dynamic modulus of elasticity of the concrete as follows:

- A maximum expansion failure criterion of 0.035 percent dilation at 350 freeze-thaw cycles is often used as an indicator of aggregate susceptibility to freeze-thaw deterioration (Kosmatka and Wilson 2016).
- A durability factor criterion (often 60 or 80 percent) based on changes in the dynamic modulus of the specimen as determined through the resonant frequency method (ASTM C215) can also indicate susceptibility to freeze-thaw deterioration. Typically, the test is run between 300 and 350 cycles. Each agency sets its own acceptance criteria based on local aggregate sources and climatic conditions.

As an alternative to rejecting an aggregate source outright, some states have found that reducing the maximum size of the susceptible coarse aggregate has been effective in reducing the freeze-thaw deterioration of aggregate in many instances. Depending on the aggregate source, the nominal maximum aggregate size may need to be reduced to 19 mm (¾ in.) or even 12.5 mm (½ in.). In such cases, it will likely be necessary to blend in larger sized non-susceptible coarse aggregates to create an aggregate blend that allows for a relatively low paste volume.

Alkali Reactivity

Evaluate aggregates for the potential susceptibility to alkali-aggregate reactivity (AAR) using R 80 .....6.7.2

The pore solution present in HCP is highly alkaline, typically having pH values in excess of 12. At higher alkalinity values, certain aggregate minerals can become unstable, resulting in instability within the aggregates themselves and the formation of reaction products that may swell and damage the concrete. There are two widely recognized deleterious alkali-aggregate reactions: alkali-carbonate reactivity (ACR) and alkali-silica reactivity (ASR). Both are summarized below and discussed in detail by Thomas et al. (2013).

Alkali-Carbonate Reactivity (ACR)

ACR is not as common as ASR but is extremely damaging, resulting in significant expansion and rapid failure of concrete structures. ACR is a result of a chemical reaction between the hydroxyl ions of alkalis in the pore solution and certain carbonate rocks (notably calcitic dolostone and dolomitic limestones) (Thomas et al. 2013). There is no known strategy that can be employed to prevent ACR other than identifying ACR-susceptible aggregates and avoiding their use in concrete. Tests that can be used to screen aggregate sources for ACR susceptibility include the following:

- ASTM C295, Standard Guide for Petrographic Examination of Aggregates for Concrete. Aggregates are microscopically evaluated by a trained petrographer who examines them for the presence of reactive constituents. The reliability of this test is highly dependent upon the experience and skill of the petrographer, who looks for specific diagnostic features within the aggregates, especially calcitic dolostones and dolomitic limestones, that are indicative of ACR susceptibility.
- CSA A23.2-26A, Determination of Potential Alkali-Carbonate Reactivity of Quarried Carbonate Rocks by Chemical Composition. The CaO, MgO, and Al_2O_3 contents of carbonate rock are measured, and the Al_2O_3 content in percent is plotted on the horizontal axis against the log of the CaO/MgO ratio on the vertical axis. Aggregates that are considered potentially expansive have a composition that will plot as follows:
  - Above a line drawn from an Al_2O_3 content of 0 and a CaO/MgO ratio of approximately 3.3 to an Al_2O_3 content of approximately 6.5 and a CaO/MgO ratio of approximately 1.75
  - Below a line drawn from an Al_2O_3 content of 0 and a CaO/MgO ratio of approximately 12.5 to an Al_2O_3 content of approximately 5.5 and a CaO/MgO ratio of approximately 63
- ASTM C1105, Standard Test Method for Length Change of Concrete Due to Alkali-Carbonate Rock Reaction. This test is conducted on concrete prisms containing cement with a lower alkali content than the cement used in ASTM C1293 (discussed next). ASR will not occur due to the lower alkali loading, and therefore any expansion observed is associated with ACR. The expansion limit is set at 0.025 percent at 6 months or 0.030 percent at 1 year. Aggregates exceeding these limits are considered susceptible to ACR.
- ASTM C1293, Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reaction. While this test is designed to evaluate aggregates for ASR, it will also trigger expansion in aggregates due to ACR. Therefore, concrete that suffers unacceptable expansion in this test should be evaluated petrographically using ASTM C856, Standard Practice for Petrographic Examination of Hardened Concrete, to determine the cause of expansion. This test is discussed in more detail with regard to ASR in the following section.

See Thomas et al. (2013) and AASHTO R 80 for a more in-depth discussion of these test methods.

Alkali-Silica Reactivity (ASR)

In contrast to ACR, ASR has been reported by most highway agencies in the United States. ASR is a result of a chemical reaction between the hydroxyl ions of alkalis in the hydrated cement pore solution and certain siliceous rocks and minerals (including opal, chert, microcrystalline quartz, and acidic volcanic glass) that are present in some aggregates. The reaction results in the formation of an alkali-silica gel that, under certain circumstances, can imbibe water, expand, and fracture the affected aggregate particles and surrounding paste.

Extensive information is available regarding the mechanisms responsible for ASR and the strategies to mitigate it (Thomas et al. 2013). For highway applications, AASHTO R 80, Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction, provides the most comprehensive recommendations on mitigating ASR. The protocols in that standard are quite detailed and should be consulted if ASR is a concern.

ASTM C295, described in the previous section as a method for identifying ACR-susceptible aggregates, can similarly be used as part of an ASR testing regime to identify many, but not all, of the potentially reactive constituents within an aggregate source. Due to the uncertainties involved, however, it is recommended that the results be used in conjunction with the results from other laboratory tests. To this end, the following two test methods are the centerpiece of AASHTO R 80 when this standard is used for ASR mitigation:

- AASHTO T 303 (ASTM C1260), Standard Method of Test for Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction
- ASTM C1293, Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reactivity

AASHTO T 303 is often referred to as the accelerated mortar bar test (AMBT) because it provides results in a relatively short time of 16 days. This makes it useful not only as a screening test but also as a test that can be conducted during the mixture design process or even during construction if one of the relevant concrete constituents changes. The test involves casting mortar beams containing the aggregate of interest (either fine aggregate or crushed coarse aggregate) and, after 2 days, soaking the beams in an 80°C (176°F) 1N NaOH solution for 14 days. Length change measurements are made periodically, and the total expansion after 14 days of soaking is typically used as the criterion to classify the aggregate as potentially reactive.

Different agencies have different criteria for the expansion limit. In AASHTO R 80, the limit is an expansion of 0.10 percent after 14 days of soaking in the NaOH solution (a total of 16 days since casting). Aggregates with an expansion of less than 0.10 percent after 14 days of soaking are considered non-reactive. An important caveat in AASHTO R 80 is that the results from AASHTO T 303 are not considered to be as accurate as those from ASTM C1293 for evaluating the reactivity of aggregates, and therefore there is some risk that a reactive aggregate source may be accepted. Because AASHTO T 303 is known to be very severe, there is also a risk that an acceptable aggregate source may be rejected. It is therefore recommended that both AASHTO T 303 and ASTM C1293 be conducted for aggregate screening.

ASTM C1293 is commonly referred to as the concrete prism test because the test is performed on concrete prisms made with the aggregates under evaluation. The concrete prism test is considered to be the best available test for assessing the potential field performance of aggregates (Thomas et al. 2013). In the concrete prism test, a standard concrete mixture with an alkali loading of 1.25 percent by mass of cement (equivalent to an alkali loading of 5.25 kg/m^3 [8.85 lb/yd^3]) is made and cast into prisms. After an initial 24-hour curing period, the concrete prisms are stored over water at 38°C (100°F), typically for 1 year when screening aggregate for use in concrete containing only pure portland cement. The expansion limit, which is included in the appendix to ASTM C1293 and cited in AASHTO R 80, is 0.04 percent.

The major limitation of ASTM C1293 is the duration of testing (a minimum of 1 year), which is feasible for aggregate source screening but highly impractical for project-specific evaluation. Another problem is that alkalis are known to leach from the concrete during testing. While the initial alkali loading can be increased beyond what would normally occur to compensate for the loss in alkalis over time, this approach only partially addresses the issue because alkali leaching can have a profound effect in practice. Therefore, it is not recommended that ASTM C1293 be used to establish the alkali threshold for an aggregate source or aggregate-binder combination (Thomas et al. 2013). Regardless of the limitations inherent in ASTM C1293, it is currently recognized as the most accurate and effective test method for screening aggregates for ASR as described in AASHTO R 80, although new test methods are emerging.

As discussed under Section 6.7.2 of AASHTO R 101, an aggregate source found to be potentially susceptible to ACR must be rejected because there is no reliable method to mitigate this type of materials-related distress. The situation is different for ASR because various mitigation strategies are available depending on the reactivity of the aggregate, the importance of the structure, and the environmental conditions, as described in the AASHTO R 80 protocol. If an aggregate is identified as being potentially ASR reactive, AASHTO R 80 requires that it be either rejected or used with one or more of these preventive measures.

There are basically four strategies available to mitigate ASR (Thomas et al. 2013) that can be used alone or in combination:

- Avoid the use of reactive aggregate. Aggregate reactivity must be assessed through testing, including a combination of petrographic analysis (ASTM C295), expansion testing of mortar (AASHTO T 303) or concrete (ASTM C1293), and an evaluation of field performance. It is emphasized that this strategy may not always be the best option, because non-reactive aggregates may not be locally available, reactive aggregates with otherwise suitable properties for use in concrete may be available locally at a low cost, or the tests may produce uncertain results.
- Minimize the total alkalis in the concrete mixture, considering both the alkalinity and amounts of the cementitious materials contained in the mixture. The required limit may vary depending on several factors, but a maximum alkali limit of 1.8 kg/m³ (3.0 lb/yd³) sodium oxide equivalent (Na_2Oe) has been adopted in the AASHTO R 80 protocol for a high level of prevention against ASR.
- Use SCMs. The use of SCMs is a very efficient strategy to control ASR, and AASHTO T 303 can be used to assess the effectiveness of SCMs at different dosage rates. Table 4 provides general recommendations regarding typical SCM levels.
- Use lithium-based admixtures. Lithium compounds, particularly lithium nitrate, have demonstrated effectiveness in mitigating ASR. Their effectiveness is highly dependent upon aggregate reactivity, however, and testing must be done to determine the required dosage of lithium-based admixtures.

The specific measures for controlling ASR are determined using either a prescriptive or performance-based approach. The prescriptive approach in AASHTO R 80 relies on the AMBT (AASHTO T 303) to classify the reactivity of the aggregate from non-reactive to very highly reactive. From these results, a level of ASR risk is defined considering the exposure conditions. The level of prevention required is then determined based on this level of ASR risk and the classification of the structure based on the consequences should an ASR problem occur. Pavements are commonly classified as S3, meaning that while minor ASR damage can cause significant safety, economic, or environmental consequences, minor risk of ASR is acceptable. The final step within the prescriptive approach to ASR mitigation in AASHTO R 80 is to identify preventive measures based on the level of prevention required. Options including limiting the alkali content of the cementitious materials, using SCMs, or a combination of the two, as discussed above.

The performance-based approach in AASHTO R 80 recommends that the concrete prism test (ASTM C1293) be used to evaluate the efficacy of ASR mitigation strategies, such as the use of SCMs or lithium compounds. The effectiveness of mitigation strategies during mixture design is assessed using slightly modified versions of two of the same test methods used for aggregate screening. The first is the AMBT (AASHTO T 303), which is conducted with the same specimen geometry, aggregate gradation, storage conditions, and acceptance criteria but with SCMs replacing some of the portland cement. It is stipulated in the AASHTO R 80 protocol that this test can only be used to assess the effectiveness of a mitigation strategy if a reasonable correlation between AASHTO T 303 and ASTM C1293 was first developed during the aggregate screening process (Thomas et al. 2013). If the correlation is not established, ASTM C1293 should be used to assess effectiveness of a mitigation strategy that involves the use of SCMs. The test method and acceptance criteria remain identical except that the test duration is extended to 2 years and SCMs may be used to replace portland cement on a mass basis. If lithium nitrate is being considered as a mitigation strategy, the recommendations presented in AASHTO R 80 must be followed.

Table 4. Required levels of SCMs to control ASR

Type of SCM	Level Required (%)
Low-Calcium Fly Ash (<8% CaO; Typically Class F Fly Ash)	20–30
Moderate-Calcium Fly Ash (8%–20% CaO; Can Be Class F or Class C Fly Ash)	25–35
High-Calcium Fly Ash (>20% CaO; Typically Class C Fly Ash)	40–60
Silica Fume	8–15
Slag Cement	35–65
Metakaolin (Calcined Kaolin Clay)	10–20

Source: Thomas et al. 2013

Workability

The American Concrete Institute (ACI) defines workability as "that property of freshly mixed concrete or mortar that determines the ease with which it can be mixed, placed, consolidated, and finished to a homogeneous condition" (ACI 2013). As stated in ASTM STP169D, the workability of concrete is a somewhat subjective property, although several test methods are available to assess workability for various applications (Daniel 2006, Cook et al. 2013, Taylor et al. 2015). For paving concrete, the most common test method used to assess workability for quality control purposes is AASHTO T 119, Test Method for Slump of Hydraulic Cement Concrete, or simply the slump test. Though considered to provide an indication of the consistency of an individual batch of concrete (Daniel 2006), the slump test is widely recognized for its limited ability to assess workability for modern concrete paving mixtures (Cook et al. 2013, Taylor et al. 2015).

To better understand the ease with which concrete can be consolidated and finished as it passes through a paving machine, its behavior under and immediately after vibration must be assessed. The vibration must adequately fluidize the stiff concrete to promote good consolidation, yet the concrete must have sufficient stiffness after vibration for the slipformed paving edges to resist edge slumping. Taylor et al. (2015) provides an excellent summary of concrete workability and the test methods developed to assess it. The Box Test and the VKelly Test are two test methods that consider both a mixture's response to vibration and its stiffness after vibration.

Box Test (AASHTO T 396)

Edge slump less than 6 mm (0.25 in.) and less than 30 percent surface voids (Ranking of 2 or less) 6.8.1

The Box Test (Cook et al. 2013, Cook et al. 2014) is designed to assess the ability of a concrete paving mixture to consolidate under vibration yet remain stiff enough to maintain a straight edge after vibration. The test conditions model the action under the slipform paving process.

The Box Test consists of a platform, two right-angled side forms, and two clamps that form a "box" having a roughly 0.3 m^3 (1 ft^3) volume. Concrete is uniformly placed into the box to a depth of 240 mm (9.5 in.), and an internal vibrator is inserted and removed from the center of the box. Immediately after vibration, the clamps are detached and the side forms removed to expose the vertical edges of the concrete. The concrete is then qualitatively evaluated for the degree of consolidation and edge slumping. The four steps involved in conducting the Box Test are shown in Figure 8.

As reported in Cook et al. (2013) and Cook et al. (2014), the test method was validated in the laboratory for a range of concrete mixtures with multiple evaluators. Limited field testing was also conducted comparing the Box Test results to the level of consolidation achieved by two different slipform pavers, and very good repeatability was achieved.

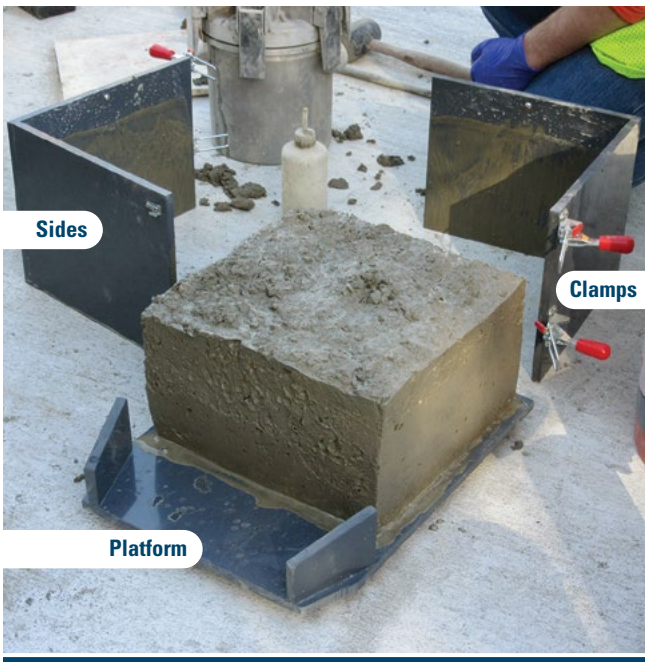
VKelly Test (AASHTO T 403)

VKelly Test: Results between 15 and 30 mm (0.6 and 1.2 in.) per root second.....6.8.2

The primary documentation for the VKelly Test is a report by Taylor et al. (2015). As summarized in the report, the VKelly Test was derived from the Kelly ball test, which was discontinued as a standard in 1999 due to lack of interest/use. Some advantages of the Kelly ball test over the slump test included better accuracy, the ability to be performed on concrete in forms (Daniel 2006), and the ability to indicate concrete yield stress (Ferraris 1999). However, the conventional Kelly ball test had limited usefulness when testing paving-grade concrete, which is typically low slump or highly thixotropic, meaning that energy is required to overcome the initially high yield stress at rest.

To overcome some of these limitations, Taylor et al. (2015) modified a conventional Kelly ball through the addition of a vibrator, which allows the test to be run both statically and with vibration. To simulate field conditions, the vibration frequency is set to 8,000 vpm. The unit is stabilized using an adjustable steel frame, and a graduated stem facilitates measurement during testing.

Taylor et al. (2015) undertook a three-phase validation study of the VKelly Test, including analyses of both laboratory and field mixtures with multiple operators and replication. The results of this work showed good repeatability over a range of mixture types. The test method proved to be sensitive to changes in mixture design parameters and correlated well with the Box Test. Furthermore, the test provided a good "static" measurement of consistency that correlated well with the slump test (AASHTO T 119) while providing additional insight into the behavior of mixtures subjected to vibration.



Step 1
Gather the different components of the Box Test.

Step 2
Construct box and place clamps tightly around box. Hand scoop mixture into box until the concrete height is 9.5 in.

Step 3
Insert vibrator downward for 3 seconds and upward for 3 seconds. Remove vibrator.

Step 4
After removing the clamps and the forms, inspect the sides for surface voids and edge slumping.

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Figure 8. Four steps involved in conducting the Box Test

Mixture Qualification

Report the following additional data in [Table 5] for mixture qualification purposes.....6.9.1

Use [Table 6] to select the property or properties desired from each section that corresponds with the narrative above to develop project specific requirements... 6.10.2

Table 5. Properties to report for mixture qualification

Property	Test Method	Purpose
Rate of flexural strength development to 90 days	T 97	Correlation with compression
Rate of strength development to 90 days	T 22M/T 22	Correlation with flexural
Coefficient of thermal expansion	T 336	Design check
Unit weight	T 121	Basis for QC monitoring
Apparent formation factor (F_{APP})	TP 119, T 358	—
Matrix saturation	TP 135, TP 136	—
Calcium oxychloride	T 365	—
Restrained shrinkage (cracking age)	T 334	—
Restrained shrinkage (stress/strength) dual ring	T 363	—

Source: AASHTO R 101

Table 6. Specification worksheet for mixture proportioning

Section	Property	Specified Test	Specified Value		Mixture Qualification	Acceptance	Selection Details	Special Notes
6.3 Concrete Strength								
6.3.1	Flexural Strength	T 97	4.1 MPa	600 psi	Yes	Yes	Choose either or both	—
6.3.2	Compressive Strength	T 22M/T 22	27.5 MPa	4000 psi	Yes	Yes		—
6.4 Reducing Unwanted Slab Warping and Cracking Due to Shrinkage (if cracking is a concern)								
6.4.1.1	Volume of Paste	—	≤25%	—	Yes	No	Choose only one	—
6.4.1.2	Unrestrained Volume Change	T 160	420 με	At 28 days	Yes	No		—
6.4.2.2	Unrestrained Volume Change	T 160	360, 420, 480 με	At 91 days	Yes	No		—
6.4.2.1.1	Restrained Volume Change	T 334	No cracking	At 180 days	Yes	No		—
6.4.2.1.2	Restrained Volume Change	T 363	<60% f'r	At 7 days	Yes	No		—

Table 6 continued on following page

Table 6 continued from previous page

Section	Property	Specified Test	Specified Value		Mixture Qualification	Acceptance	Selection Details	Special Notes
6.5 Durability of Hydrated Cement Paste for Freeze-Thaw Durability								
6.5.1.1	Water to Cementitious Ratio	—	0.45	—	Yes	Yes	[a]	—
6.5.1.2	Fresh Air Content	T 152, T 196, TP 118	5% to 8%	—	Yes	Yes	Choose only one	—
6.5.1.3	Fresh Air Content/SAM	T 152, T 196, TP 118	≥4%; ≤0.20	—	Yes	Yes		—
6.5.2.1	Time of Critical Saturation	ASTM C1585	30	yr	Yes	No	[a, b]	Variation controlled with mixture proportion observation or *F* factor and porosity measures
6.5.3.1	Deicing Salt Damage	—	30%	SCM	Yes	Yes	Choose only one if concrete will be exposed to deicing salts	Are calcium or magnesium chloride used
6.5.3.2	Deicing Salt Damage	M 224	—	Topical treatment	Yes	Yes		Are calcium or magnesium chloride used; use specified sealers
6.5.4.1	Calcium Oxychloride Limit	T 365	<0.15 g CAOXY/100 g paste		Yes	No		Are calcium or magnesium chloride used
6.6 Transport Properties								
6.6.1.1	Water to Cementitious Ratio	—	≤0.45 or ≤0.50	91 days[c]	Yes	Yes	Choose only one	The required maximum water to cementitious ratio is selected based on freeze-thaw conditions
6.6.1.2	Formation Factor	Table 1	≥500 or ≥1000	91 days[c]	Yes	Yes		Based on freeze-thaw conditions; other criteria could be selected
6.6.2.1	Ionic Penetration, *F* Factor	—	25 mm at 30 yr	91 days[c]	Yes, F	Through ρ		—
6.7 Aggregate Stability								
6.7.1	D-Cracking	ASTM C1646, T 161	—	—	Yes	No	—	Procedure A
6.7.2	Alkali Aggregate Reactivity	R 80	—	—	Yes	No	—	—
6.8 Workability								
6.8.1	Box Test	TP 137	<6.25 mm, <30% surface void		—	No	—	—
6.8.2	VKelly Test	T 403	15–30 mm/root s		—	No	—	—

Source: AASHTO R 101

Notes:

[a] *Choose either Section 6.5.1.1 or 6.5.2.1.*

[b] *Choose either Section 6.5.1.2, 6.5.1.3, or 6.5.2.1.*

[c] *Other ages can be used if desired. However, for SCMs sufficient time should be allowed for the pozzolanic reaction.*

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