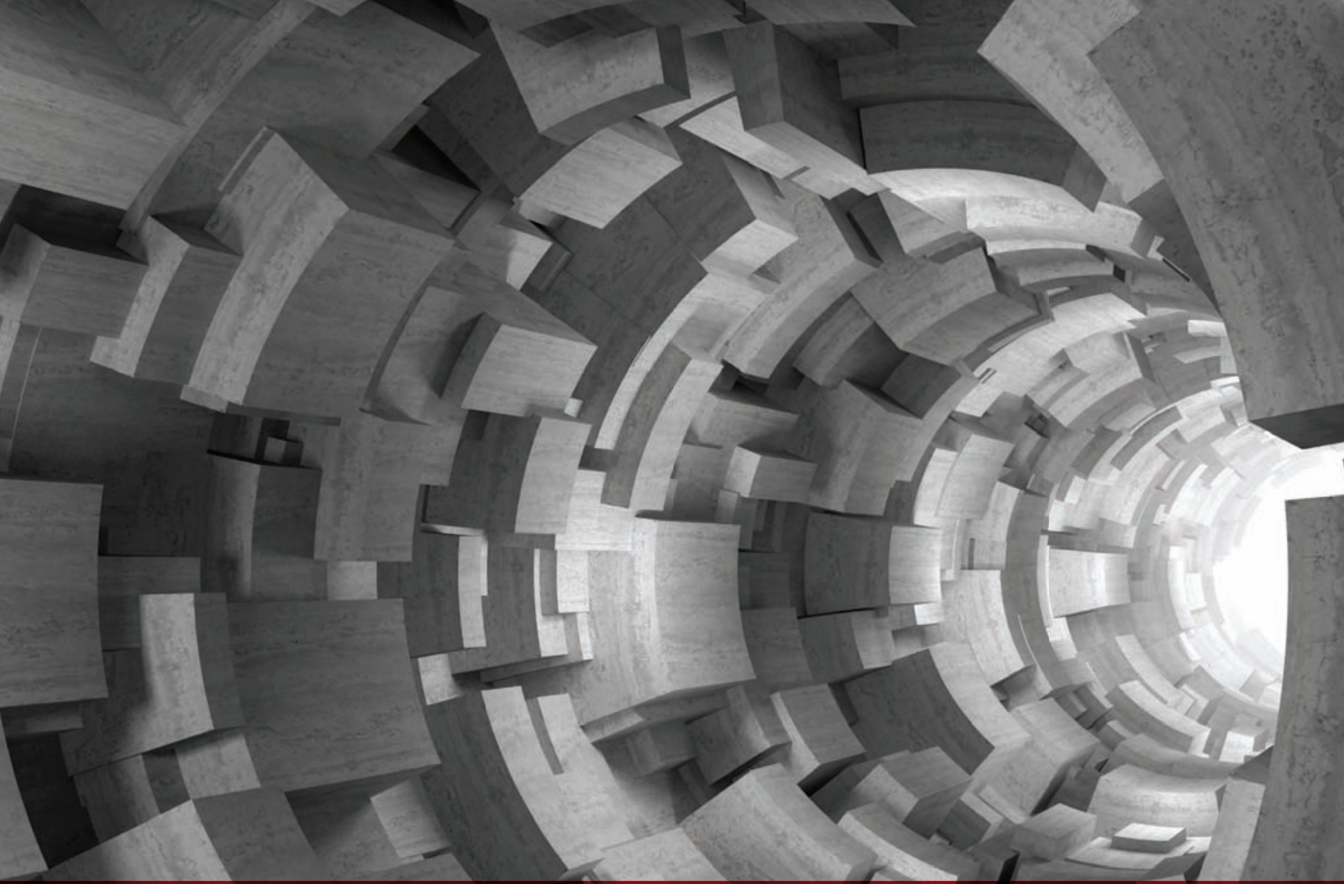

CONCRETE FOR UNDERGROUND STRUCTURES

GUIDELINES FOR DESIGN AND CONSTRUCTION

EDITED BY ROBERT J.F. GOODFELLOW

SME

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Contents

Preface

Concrete is an essential tool of the underground industry and one that has a significant impact on both the durability and capital cost of a constructed project. Concrete is used in almost every project as tunnel support, backfill, or internal structure. As such, all of the parties to a project—owners, designers, and contractors—have an interest in ensuring that concrete is designed and placed in the most cost-efficient way that ensures an appropriate level of quality in the end product.

Because of the importance of concrete to the underground industry, the Underground Construction Association of the Society for Mining, Metallurgy, and Exploration (the UCA of SME) prepared this book to summarize the current best practices of the industry with regard to the design and construction of concrete underground. The intent is to describe key design and construction issues and practices, emerging technologies and solutions, and areas in which improvements are being actively sought by the industry. The book is written as both a source of reference material for industry veterans as well as an educational tool for those new to the industry.

This document was written collaboratively by a committee of authors, and representatives of the tunnel industry provided review and comments. Industry review allowed us as authors to gather input from a wide circle within the industry in order to convey in these pages the industry consensus on design and construction practices as accurately as possible. The industry consensus is reflected in all aspects of this book.

In an accessible, but not overly simplistic manner, the main body of text in this book summarizes the basics of concrete and goes on to describe how the use of concrete underground differs from its use aboveground. Three applications of concrete are considered here: (1) cast-in-place concrete, (2) precast concrete segments, and (3) shotcrete. Each chapter addresses specific applications of these materials, including shafts, tunnels, and caverns. This book specifically does not consider cementitious materials in certain applications, including: grout of any kind, backfill behind pipelines or composite material applications (where concrete is placed behind a structural steel shell), and underwater or tremie concrete placement.

Chapter 5 discusses in detail the different types of concrete admixtures and their impacts on a concrete mixture and Appendices A, B, C, and D contain sample specifications for each type of concrete. While the specifications contain guidelines for language that may be used, it is acknowledged by the authors and by SME that each project has unique aspects, and the specifications should be examined carefully for compatibility with the project's needs and consistency with other contract documents before using this language. Commentary in each guideline specification is provided to help in writing a better concrete specification for each project application.

Acknowledgments

It is inevitable in a guideline document such as this that, although one name appears on the cover, the book is the product of many people's hard work. Among those who had an active and significant role in putting this guideline together, first and foremost is George Yoggy, who was the driving force in initial discussions and a continual inspiration to defining the need and purpose of this book.

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CHAPTER 1

Introduction

Robert Goodfellow

To thoroughly understand the information presented in this book, it is important for the reader to have a basic knowledge of concrete components, how concrete is placed underground, and how the underground environment affects concrete mixture design overall. Many readers will already possess this knowledge, but some may not. This chapter aims to provide a brief and reasonably basic foundation in these topics for readers who are not completely versed in concrete material science. For a more detailed description of concrete basics and concrete mixture design, many textbooks are available, such as *Design of Reinforced Concrete* (McCormac and Brown 2009).

OVERVIEW OF CONCRETE TYPES AND PLACEMENT

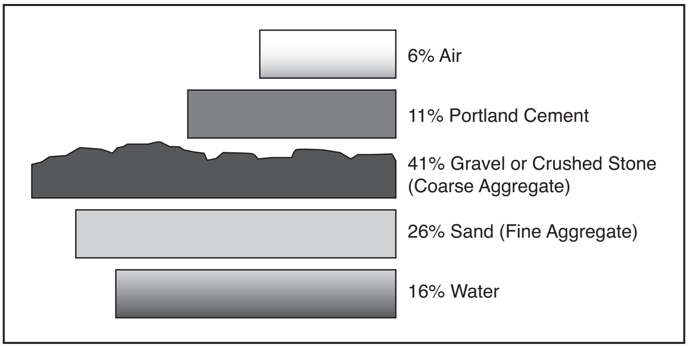
Underground projects such as tunnels, shafts, and caves almost always incorporate concrete elements. The most significant use of concrete underground is as a lining that provides initial and/or final ground support and, if needed, protection from corrosive environments. Initial and final linings may be cast-in-place (CIP) concrete, precast concrete segments, shotcrete, or combinations thereof. CIP concrete uses forms into which the concrete is placed and allowed to set until it attains a specified strength and the forms can be removed. Precast concrete segments are manufactured at a segment manufacturing plant and installed in the tunnel behind tunnel boring machines (TBMs). Shotcrete is transported, similar to CIP concrete, to the point of application before being sprayed directly onto the tunnel surface using a spray nozzle without the need for formwork.

All three applications of concrete raise construction issues underground that differ from considerations aboveground. The biggest differences arise from the confined nature of underground construction, distance from point of delivery to point of placement, and the atmosphere or environment underground. These issues will recur again and again as we discuss in later chapters the construction and specification considerations related to each of the concrete applications.

Different methods of construction require different applications of concrete, whether excavating in rock or soft ground and whether using drill-and-blast or mechanical methods of excavation. Combinations of CIP concrete, shotcrete, and precast concrete segments are applied in almost all underground excavations in either primary or secondary linings or in one-pass lining systems.

UNDERGROUND CONCRETE MIXTURES

The following information about concrete mixture design will not be new to anyone with design or construction experience. Rather, it is intended to provide context for briefly introducing the aspects of concrete mixture design that are specific to the use of concrete underground, which will



Courtesy of Portland Cement Association.

Figure 1.1 Typical volumetric proportions of concrete mixture basic ingredients

be discussed in greater detail in later chapters of this book. It is important to note that in a well-designed concrete mixture all the ingredients are properly proportioned for the specific purpose of the concrete (typical volumetric proportions are shown in Figure 1.1). CIP mixtures are not the same as precast segments or shotcrete mixtures. A simple example of the interrelations of concrete mixtures is the gradation of aggregates. Reducing the maximum size of coarse aggregate—for example, in a shotcrete mixture—requires an increase in the proportion of fine aggregate, consequently requiring a higher proportion of cement to adequately cover the increased surface area of aggregate with paste. This increased surface area of aggregate and more cement means that more energy is required for proper mixing prior to application. There are further consequences of increasing cement content with regard to water content, water-to-cementitious-materials (W/C) ratio, and many other factors. Suffice to say, all aspects of a concrete mixture are related and can rarely be taken in isolation if the most efficient application of concrete is desired. Most frequently, there is more than one correct answer when so many variables exist.

Cement

Portland cement is the most common cement used in concrete. Production of portland cement follows the requirements of the American Society for Testing and Materials (ASTM) standards C150/C150M. There are eight types of portland cement:

1. Type I: normal
2. Type IA: normal, air entraining
3. Type II: moderate sulfate resisting
4. Type IIA: moderate sulfate resisting, air entraining
5. Type III: high early strength
6. Type IIIA: high early strength, air entraining
7. Type IV: low heat of hydration
8. Type V: high sulfate resisting

The various types are more or less available around the United States, depending on where the cement is mined. For example, Type V cement may be difficult to obtain and therefore may be more expensive in some areas. The use of and need for a particular type of cement is dictated by the structure being built. For example, sewer applications typically require at least Type II cement for moderate sulfate resistance, and many specifications demand the use of Type V cement for this application, whereas sulfate resistance may not be necessary for a highway tunnel.

Coarse Aggregate (Stone and Gravel)

Coarse aggregate often makes up more than 40% of the volume of a concrete mixture. As such, selection of appropriate coarse aggregate is critical. The ideal aggregate is chemically inert to the mixture and to the conditions to which it will be exposed, and has adequate strength to resist the load conditions anticipated throughout the life of the structure. In addition to chemical properties and strength, the selection of coarse aggregate is highly influenced by the intended placement method. Different placement methods often call for aggregates with different gradation, shape, and roughness/roundness.

Gradation has a significant impact on pumpability and workability of a concrete mixture, because the gradation of the aggregate determines the amount of cement needed to adequately coat each particle. By affecting pumpability and workability, gradation also impacts the type of equipment needed to pump the mixture.

Shape, roughness, and roundness of coarse aggregate affects the strength of the mixture and, like gradation, affects placement. Elongated aggregate is difficult, if not impossible, to pump long distances because it blocks pump lines. Larger aggregate is not used in shotcrete applications for similar reasons.

One of the most serious issues with aggregate, discussed later in this chapter under "Durability and Degradation," is an alkali–silica reaction (ASR).

Fine Aggregate (Sand)

Fine aggregate is used to fill in holes between the coarse aggregate and provide a more robust mix. A basic concrete mixture may be 25% to 30% fine aggregate by volume. However, some concrete—such as pervious concrete—does not contain any fine aggregate. Pervious concrete is often used to provide drainage pathways.

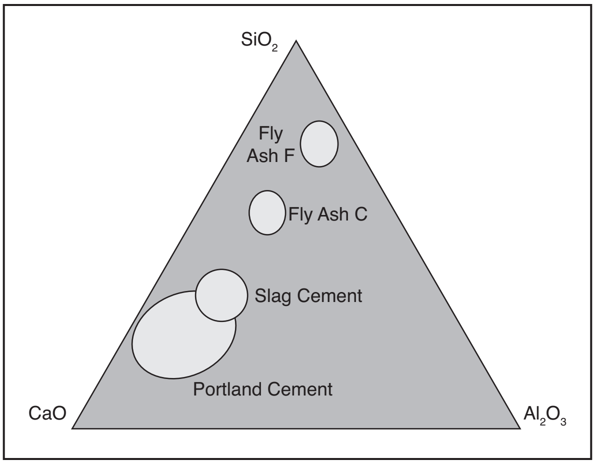
Aggregate for use in concrete should meet the requirements of ASTM C33/C33M. The combined coarse and fine aggregate grading must be smooth without any gaps between particle sizes. ASTM C1436 gives requirements for combined grading for shotcrete mixtures.

Water

The conventional wisdom about the water used in concrete is that if you can drink it, you can make concrete out of it. Excessive chemical pollution of water can interfere in the hydration process, effectively introducing additional unknown "admixtures" that can influence concrete strength gain and durability. Potable water has sufficiently low levels of pollutants to remove this concern. Water for use in concrete should meet the requirements of ASTM C1602/C1602M.

Supplementary Cementitious Materials

The hydration of cement is an exothermic reaction, producing heat. Fly ash and other pozzolans as well as ground granulated blast furnace slag (slag cement) can be added to the mixture to help reduce the heat of hydration and consequently can reduce the incidence of thermal cracking. The chemical composition of these replacement materials is shown in Figure 1.2. Other benefits of these materials include increased resistance to ASR as well as increased fines leading to better pumpability of the mixture.



Courtesy of Slag Cement Association.

Figure 1.2 Ternary diagram for cementitious materials. Note the chemical similarity allowing a higher level of cement replacement using slag cement as compared with fly ash.

Microsilica, or silica fume, is a particle many times smaller than cement, and, as a consequence, microsilica fills the gaps in the matrix between the cement particles on hydration. Adding microsilica to a mixture increases density and reduces permeability. Using microsilica has great benefits in applications where increased durability, strength, and reducing water infiltration are important, such as water and wastewater conveyance projects.

Supplementary cementitious materials added directly to concrete mixtures are governed by ASTM C618 (fly ash and natural pozzolans), ASTM C989 (slag), or ASTM C1240 (silica fume).

Admixtures and Entrained Air

Air is naturally entrained in the concrete mixture during mixing. ACI 318 states that air content for concrete exposed to freeze–thaw cycles may vary between 3.5% and 7.5%, depending on exposure and size of aggregate with allowable variation of ±1.5%.

Air-entraining admixtures are chemical admixtures that are added to concrete to introduce microscopic air bubbles into the concrete during mixing to improve its fresh and hardened properties. Entrained air can also enhance the workability and pumpability of concrete. Air-entraining admixture should meet the requirements of ASTM C260. See Chapter 5 for details.

Chemical admixtures are chemical compounds that create variations in the properties and behavior of fresh and hardened concrete. These variations may be desirable for many reasons. Admixtures are also discussed in detail in Chapter 5.

QUALITY OF UNDERGROUND CONCRETE MIXTURES

The quality of concrete in underground construction is determined by several factors:

- W/C ratio
- Workability
- Mixing

- Placement
- Hydration and curing

W/C Ratio

The W/C ratio is the most important single measure of the quality of a mixture and is defined as the ratio of water used (by mass) to cementitious materials used (by mass). If supplementary cementitious materials are used, such as fly ash and silica fume, these are added to the cement content to provide the total cementitious materials by mass. With all the other many variables being equal, a lower W/C ratio generally results in a stronger and more durable concrete.

Workability

As will be discussed in some detail in later chapters, workability is defined as the ability of a mixture to be conveyed, placed, and consolidated without segregation. Workability of the mixture is closely related to the W/C ratio. Adding more water without adding more cement can improve workability but reduces strength. As such, the degree of workability required for adequate placement is weighed against the required structural strength and durability to select an appropriate W/C ratio.

Mixing

Mixing concrete ingredients requires that energy is imparted to the mixture such that all ingredients become evenly distributed. Traditionally in North America, CIP concrete is mixed inside a drum equipped either with fins or paddles, which mix the cement. When a normal to large aggregate mixture (0.75 to 1.5 in. or 20 to 35 mm) is prepared, the shear energy required to homogenize the components is provided by the coarse aggregate tumbling through the fine particles of the mixture. Precast concrete plants use horizontal paddle mixers that directly apply energy to the mixture through the action of those paddles. In general, the smaller the aggregate, the more energy required to mix the cement.

A high-shear modern mixing system such as a ring-pan or twin shaft system is appropriate for underground construction applications as high-performance mixtures are generally used with demanding specifications. Without proper mixing, the concrete mixture will not perform up to its maximum level of performance.

Management plans for concrete projects should address mixing procedures and mixture monitoring. This is especially critical if shotcrete or other small aggregate mixtures are being used. Batching and mixing procedures should result in a mixture that at least meets the requirements of ASTM C94/C94M for ready-mixed concrete, ASTM C1116/C1116M for fiber-reinforced concrete, and ASTM C685/C685M for volumetric batched and mixed concrete.

Placement

The success of an underground project is often determined by the efficiency of the concrete placement cycle. For example, where CIP concrete is being used as a lining, the production and construction cycle of place concrete, strip forms, reset forms, and place again is the lifeblood of the contractor. It can literally make or break a project. Where precast concrete segments are being used as lining, a similar cycle is equally important, both at the segment manufacturing plant and when installing the ring behind the TBM. Though it doesn't involve forms, shotcrete also relies on efficient delivery of a constant volume of concrete for placement, because an interruption in supply can be catastrophic when immediate ground support is required within an excavation cycle.

Put simply, the underground industry has taken concrete, a basic and ancient construction material, and applied it in difficult-to-access underground spaces in relatively small volumes that require very efficient placement to be commercially viable. Accordingly, major considerations in concrete mixture design are those that influence the ability of concrete to be

- Pumped long distances,
- Dropped from the surface to the tunnel elevation, and/or
- Cured in a moist and hot environment.

These factors are discussed in greater detail in the chapters that follow.

Hydration and Curing

Curing concrete involves limiting loss of moisture from the concrete (hydration) and keeping the mixture above approximately 40°F (4°C) throughout the hydration process. Curing is generally easier underground than curing aboveground due to the confined, hot, and moist atmosphere underground. This type of atmosphere is the exact condition that often must be artificially created aboveground for better curing.

The speed of the curing process relates directly to concrete strength gain, which is, of course, a central concern in any underground project. Both the designer and contractor have a stake in determining how quickly the mixture will achieve an initial set and then how quickly it will gain additional strength. The ultimate specified strength for the mixture is typically a 28-d strength measurement. However, strength tests are carried out on the mixture at several stages during construction at time intervals that depend on the method of placement. Shotcrete may be tested several times in the first 24 h to determine initial set and ground support characteristics before it is tested to see if it has achieved its 28-d specified strength. CIP concrete is generally tested for early stripping strength and then at 28 d for quality control purposes.

DURABILITY AND DEGRADATION OF CONCRETE

Reinforced concrete is generally a highly durable material in most environments but is inherently vulnerable to some forms of deterioration. These can be either chemical or physical attack of the concrete itself or corrosion of embedded reinforcement. Special precautions may be needed to ensure the required performance.

External Degradation

Chemical. Underground concrete may encounter aggressive chemicals through contact with groundwater, both natural and contaminated, or in materials that are being contained or transported. Mobility of water can have a large influence through its ability to continually refresh the aggressive ions. Static groundwater will generally be much less aggressive.

Sulfates. Sulfates are commonly present in groundwater and can react with the concrete matrix to form ettringite, gypsum, and/or thaumasite, depending on prevailing conditions such as temperature. This can result in progressive damaging expansion or softening of the concrete surfaces. Water containing very high concentrations of sulfates should be prevented from contact with concrete through the use of a suitable barrier. Lower concentrations can usually be adequately resisted through the use of well-compacted concrete with a low free water/cement ratio and dense surface, and through selection of cement type (secondary cementitious materials or portland cement with low tricalcium aluminate, C_3A, content).

Acids. Concrete can be attacked by acids with pH less than 6.5, although generally the attack is not severe until less than 5.5, and potentially very severe below pH 4.5. The type of acid (mineral or organic) is also influential. Attack occurs through gradual dissolution of the concrete matrix and any soluble aggregates, leading to weakening and loss of section. Portland cement–based concrete (including that containing secondary cementitious materials) is inherently vulnerable to strong acid, and direct contact should be avoided. Weaker acids can be resisted through the use of well-compacted concrete with a low free water/cement ratio and dense surface. The choice of cement type is less important, although the use of silica fume may be beneficial.

Seawater. Seawater contains many different species of ion, but its effect on concrete is mainly through the action of magnesium sulfate, although the sulfate action is modified by the presence of chlorides, and damaging expansion does not generally occur. Indeed, the magnesium can cause the formation of a layer of brucite, which, if not eroded by abrasive actions, can be protective. The choice of cement type to resist seawater attack is less important than the use of a low free water/cement ratio and the achievement of a dense surface. Nevertheless, portland cement with tricalcium aluminate content greater than about 8% should be avoided because of the risk of sulfate attack.

Leaching. Flowing soft water, low in calcium, in contact with concrete will gradually dissolve calcium compounds from the concrete matrix until eventually all that remains is an incoherent silica gel. Nevertheless, this action is generally very slow for concrete with a low free water/cement ratio and a dense surface. No-fines concrete used for drainage may be especially vulnerable to this type of attack.

Sewage. Sewage is not, in itself, aggressive to concrete but if sludge is allowed to build up—wherein hydrogen sulfide is produced by anaerobic bacteria and subsequently oxidized by aerobic bacteria to sulfuric acid—severe attack can occur rapidly. Because it is not possible to produce portland cement–based concrete to resist such conditions, they must be avoided by design or a barrier provided to prevent contact. The implications for wastewater tunnels are self-evident.

Other Chemicals. Other chemicals particularly aggressive to concrete include aluminum chloride and most ammonium salts. Such chemicals are likely to be encountered only in solutions associated with industrial processes. Ammonium nitrate may be present in groundwater through its use as fertilizer, but a concentration sufficiently high to be dangerous to concrete would not normally be expected.

Influential Factors. Chemical attack of concrete will be affected by many factors, including

- Concentration of aggressive ions,
- Temperature (most chemical reactions are accelerated by heat),
- Presence and mobility of water (to replenish the aggressive ions),
- Composition of concrete (cement type, free W/C ratio),
- Quality of concrete surface, and
- Hydrostatic head.

Physical Degradation

Freezing and Thawing. Concrete surfaces subjected to freezing while saturated experience large stresses within the pores as the water expands to form ice. This can cause cracking and loss of the concrete surface. Freeze–thaw resistant concrete can be produced through the inclusion of a suitable entrained air-void system or, in moderate freeze–thaw conditions, by use of concrete with sufficient strength to resist the expansion forces. Partially saturated surfaces do not generally suffer freeze–thaw damage.

Salt Weathering. Concrete in contact with water containing salts on one face and subject to strong drying conditions on the other can suffer disintegration of the internal face. Recrystallization of salts in the surface pores, as permeating pore water evaporates at the free surface, can destroy the pore structure. This can be prevented by the use of an external barrier or an integral pore-blocking admixture to prevent absorption.

Abrasion. Flowing water containing hard solids can abrade concrete surfaces. Improved abrasion resistance can be achieved through increased strength, good finishing and curing, and by the use of silica fume as a secondary cementitious material. Cavitation can occur where flowing water undergoes a change of pressure such that bubbles form and subsequently collapse, causing a shock wave that can be very aggressive to concrete surfaces. This should be avoided by design.

Internal Degradation

Alkali-Aggregate Reaction. Certain types of aggregates contain materials that are potentially reactive with alkalis in concrete under wet conditions; a subset of this type of degradation is ASR where silica in aggregate is reactive. The reaction can be expansive and may result in damaging deformation and cracking of concrete elements (Figure 1.3). ASR is caused by a reaction between ions in the alkaline cement and reactive forms of silica in the aggregate (e.g., chert, quartzite, opal, strained quartz crystals). In areas where ASR is possible with the proposed aggregate source, this must be checked by chemical testing before approval of the aggregate. Potentially reactive aggregates are best avoided, but, where this is not possible, risk can be minimized through specification measures including limiting alkali content and inclusion of certain secondary cementitious materials.

Delayed Ettringite Formation. In wet conditions, delayed ettringite formation (DEF) is a potential risk if excessive temperature is developed during hydration because ettringite cannot form normally during hydration. Ettringite is a large, needle-like crystal and cannot be readily accommodated if it forms later within the structure of the hardened concrete matrix, and damaging expansion, similar to ASR, may result. The risk of DEF increases with peak hydration temperature above about 158°F (70°C) or higher for certain secondary cementitious materials. It can be avoided altogether by keeping the hydration temperature below this critical level at all stages of the curing process.

Reinforcement Corrosion

Reinforcement in concrete can corrode if the alkaline protection is reduced by carbonation or if the chloride level at the reinforcement depth builds up to beyond the corrosion threshold level, provided there is adequate moisture and oxygen available.

Carbonation-induced Corrosion. Carbon dioxide in the atmosphere will react with the concrete matrix, reducing its alkalinity to near neutral pH. If the carbonated zone reaches the reinforcement, the protective oxide layer will be broken down and corrosion will be initiated. This process is normally slow in good-quality concrete, and required design lives can normally be achieved by provision of adequate depth of cover. Heavily trafficked road tunnels and industrial processes can result in greatly elevated atmospheric carbon dioxide (CO_2) levels and proportional increases in carbonation rates. Resistance to carbonation-induced corrosion can be enhanced by increased concrete quality, increased cover depth, and anticarbonation surface treatments. In

Courtesy of Portland Cement Association.

Figure 1.3 Examples of alkali–silica reaction between cement paste and reactive aggregates

relatively dry environments, corrosion rates may be very slow, and it may be several decades after initiation before cracking or spalling damage occurs.

Chloride-induced Corrosion. Where concrete is in contact with waterborne salt from brackish groundwater, seawater, industrial processes, or de-icing chemicals (including spray from vehicles), chloride ions will ingress toward the reinforcement by diffusion through water-filled pores. Initially dry surfaces may suffer more rapid ingress through capillary absorption. Conditions may be particularly onerous where one face is in contact with chloride-bearing water and the other is exposed to air, such as an immersed tunnel. If the chloride ion concentration at the reinforcement or other corrodible embedded metal reaches the so-called threshold or critical level, corrosion will be initiated. Rates for chloride-initiated corrosion are generally much greater than for carbonation-induced corrosion, and the period between initiation and manifestation of damage may be only a few years.

Influential Factors. Both carbonation and chloride-induced corrosion processes are influenced by many factors, including

- Temperature: Both ingress and corrosion rates will be increased by heat but may be very low in very cold conditions.
- Moisture: Optimum internal relative humidity ranges for the initiation processes may differ from those for corrosion.
- Oxygen availability: Completely and deeply buried concrete may have an insufficient supply of oxygen to support significant corrosion rates.
- Depth of cover.
- Composition of concrete (cement type, free W/C ratio).
- Concentration of chlorides or CO_2 at the surface of the concrete.

Corrosion-resistant Reinforcement. For long design lives in particularly aggressive conditions or where cover depth is constrained by other considerations, the use of corrosion-resistant reinforcement may be the most effective solution. It should be noted that corrosion-resistant reinforcement is not considered appropriate for conventional application underground. Where it is considered necessary, stainless steel is the most suitable. Fusion-bonded epoxy and galvanized reinforcement are generally less reliable in chloride-bearing environments where corrosion-resistant reinforcement is most likely to be considered.

CONCLUSION

The information presented in this chapter is important because the rest of the book is based on this knowledge. The following chapters build on this knowledge with in-depth discussion of the three applications of concrete underground: CIP concrete, precast concrete segmental linings, and shotcrete.

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CHAPTER 2

Cast-in-Place Concrete

Shane Yanagisawa and Leon Jacobs

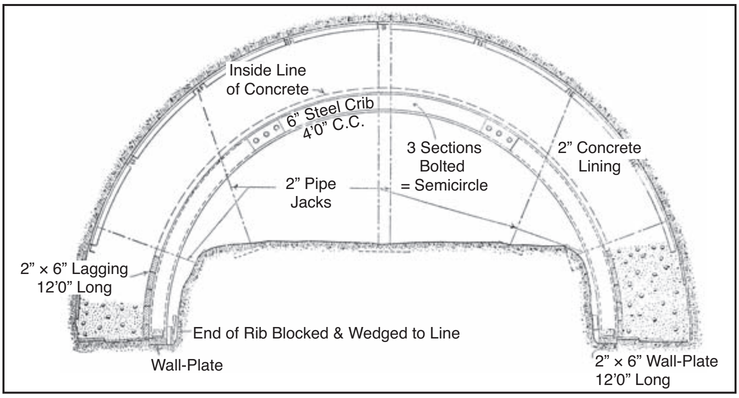
Cast-in-place (CIP) concrete may be placed directly against rock, shotcrete, or a waterproofing membrane or slip sheet. As in construction aboveground, forms are used to place the concrete and are removed when the concrete has gained sufficient strength. However, there are many differences between aboveground and belowground applications of CIP. The concrete mixture, the factors involved in placement, the forms, and a variety of other design and construction factors all differ in underground construction.

INTRODUCTION

During the late 1800s, most tunnels requiring linings for long-term stability used brick or cast-iron segmental plates. CIP tunnel linings started being placed in the early 20th century. The Colorado River Siphon Tunnel near Yuma, Arizona, was driven and lined from 1909 to 1912. The siphon had cast concrete caisson sunk shafts 90 ft (27 m) deep and a 900-ft (274-m) long tunnel with a 14-ft (4-m) finish diameter. The circular section tunnel was driven under compressed air in a top heading and bench manner. The ground was temporarily supported by thin steel segmental plates that bolted together to make a ring 1 ft (0.3 m) wide. After the top heading was advanced 14 ft (4 m), a form with steel ribs and wood lagging was set up, and a 2-ft (0.6-m) thick concrete lining was placed (Figure 2.1). The concrete was delivered by small railcars through the pressure locks and was hand shoveled behind the forms. The bench was then excavated and concreted using the same formwork rotated 180 degrees. The concrete had the proportions by weight of 1 part cement, 2½ parts sand, and 5 parts crushed rock.

By the 1930s concrete had become an essential component of tunnel linings. During the construction of Hoover Dam, which borders Arizona and Nevada, four 56-ft (17-m) diameter tunnels were driven through the rock to divert the Colorado River. These tunnels, totaling 16,000 ft (4.8 km) in length, were lined with a 3-ft (0.9-m) thickness of concrete to a finished diameter of 50 ft (15 m). The concrete was placed in three lifts (invert, sides, and top arch) using steel forms for the sides and arch. The top arch concrete was placed using a pneumatic concrete placer. In 1932, the tunnel linings were completed. At the conclusion of dam construction, two of the tunnels were permanently plugged with 400 ft (122 m) of concrete.

The pneumatic placer operated by dumping a fixed amount of concrete (somewhere between ½ yd^3 (0.38 m^3) and 1 yd^3 (0.76 m^3) into a sealable pot. The pot was closed and pressurized with air forcing the concrete through a 6-in. (152-mm) steel line into the annular space between the form and the ground. The steel pipe, called a slickline, was situated at the 12 o'clock position and was retracted as the space filled with concrete. The force of the concrete did a better job of filling



Source: Schobinger 1914, reproduced with permission from ASCE.

Figure 2.1 1914 drawing of concrete placement and formwork scheme for Colorado River Siphon

the top arch of the form with concrete than could be done by hand shoveling. Placement rates for pneumatic placers were in the range of 30 yd^3/h (23 m^3/h) compared to the 1-to-1½ yd^3/h (0.76-to-1 m^3/h) possible with hand shoveling. Although concrete pumps began placing concrete in the late 1930s, they did not have much power. Concrete could be placed at rates of 25 to 40 yd^3/h (19 to 31 m^3/h) but required the assistance of an air slugger to pack the concrete into the forms.

Separate invert, sidewall, and arch forms gave way to combined wall and arch forms filled by a pneumatic concrete placer. By the 1940s, small tunnels were still formed with wood but steel-skinned forms were being used for large tunnels. Lack of suitable retarders made delivery of concrete to the tunnel form within 45 min to 1 h essential. For shallow tunnels, the favored method was to drop the concrete through shafts spaced 1,000 to 1,500 ft (305 to 456 m) apart. Batching of concrete inside large tunnels was also done.

The first hydraulic-powered twin cylinder concrete pump was introduced in 1957. Since the 1970s, modern concrete pumps sped up the placement of tunnel concrete, and placement rates of 100 yd^3/h (76 m^3/h) are now commonplace. The advent of modern concrete admixtures in the 1990s, most notably superplasticizers (e.g., high-range water-reducing admixtures such as a polycarboxylate) and long-term hydration stabilizers, radically improved the ability of the tunnel builder to place high-quality concrete on a high-production basis. Example specifications for CIP concrete linings are provided in Appendix A.

DIFFERENCES BETWEEN ABOVEGROUND AND UNDERGROUND APPLICATIONS OF CAST-IN-PLACE CONCRETE

The differences fall under five categories: concrete mixture, placement, formwork, schedules, and testing. These differences highlight the logistical challenge of concrete material delivery to one location at the surface and placement of the material into a confined area in a reasonably small quantity and a potentially long distance from the point of surface delivery.

Concrete Mixture

The differences between aboveground and belowground application of CIP concrete begin with the concrete itself. The concrete mixture must have a well-balanced combination of short- and long-term characteristics. In the short term, tunnel concrete is usually placed on a 24-h cycle because the lining is on the project schedule's critical path, and most projects carry substantial liquidated damages for late completion. The concrete must gain strength quickly enough to allow for timely form stripping and resetting. The concrete should have enough cementitious material to achieve the required early strength but no more than what is necessary to reliably meet the long-term design requirements. Excessive cementitious material added to gain early strength will increase heat generation and thermal expansion, causing cracks.

From a long-term perspective, tunnel-lining concrete must typically provide a service life of up to 100 years while requiring minimal maintenance. Tunnel portals have to resist freeze and thaw cycles. Concrete must resist attack by aggressive groundwater or road salts. Sewer tunnels must resist corrosion from gases and impact loads at the base of drop shafts from inflows into the tunnel. Water tunnels must resist abrasion from rocks and grit in the invert. Hydropower tunnels must resist high velocities and cavitation. Transportation tunnel linings must resist intense fire. Depending on the application, the lining must resist internal or external water pressure. The lining must withstand ground-induced loads of all kinds, including ground pressures and earthquake loads.

Placement

Tunnel construction is notable for restricted access. The portals and shafts are the only means of supplying labor, materials, and supplies—all of which must be brought to the heading in a carefully coordinated fashion. Access to many tunnels is by small-gauge rail systems, often with only single-track access.

The access limitations affect concrete placement as well. The final point of concrete placement may be far from the point of conventional delivery. Concrete may be pumped thousands of feet and may encounter substantial vertical drops or increases in height via a shaft or incline. It may also be transferred multiple times from one method of conveyance to another. For example, concrete might be sent by truck from a batch plant to the job site, transferred to railcars, and then pumped into the forms. The long supply chain required to deliver a time-critical material (fresh concrete) to the point of placement requires that the entire chain work in a well-coordinated fashion. Otherwise the risk of plugged concrete lines, disrupted schedules, and rejected concrete is high.

Tunnel liner concrete is typically placed via a slickline (an assembly of steel pipe that is continuously smooth on the outside to allow easy withdrawal of the pipe from the space between the form and the excavated tunnel wall) over the top of the concrete form or by injecting the concrete through the form using a special injector car and ports built into the form. Concrete may also be placed through close-fitting hatches fitted to the side of the form.

Formwork

Typical modular panel formwork systems used aboveground don't work in a tunnel, where the arch is a predominant feature. Specialized forms are required. The formwork is typically one-sided and has no form ties to resist loads from freshly placed concrete. Because the cavity to be filled with concrete is narrow and often occupied with reinforcement, access to the space for introducing internal vibrators is limited. External vibrators are usually bolted to the steel formwork to consolidate the concrete by vibrating the form skin (as shown in Figure 2.2). The vibrators and the concrete liquefied by the vibrators place high loads on the form. When the pump forces concrete into the cavity between the form and the excavated tunnel, the form must resist the gravity load of

Courtesy of Shane Yanagisawa.

Figure 2.2 Typical steel form interior with window, with form vibrator at right and positioning spuds at top

the concrete and the pumping pressures. The form also must resist lateral loads caused by differential heights of concrete on opposite sides of the form. The forms are also exposed to dynamic loads caused by the activation of vibrators bolted to the form framework. Full-round forms (circular forms that allow concrete to be placed around the entire 360-degree envelope at once) must resist strong uplift forces. In tunnels where the arch is cast separately from the invert, the form must resist uplift if the portion below springline has inclined sides, as is typical with New Austrian Tunneling Method (NATM) style tunnels. Tunnel form design loads are typically in the range of 1,500 psf (70 kPa) for the invert and sides and up to 3,000 psf (140 kPa) for the top quarter arch.

Tunnel forms must resist cyclic loading, deformation, and wear caused by multiple reuses, sometimes for hundreds of cycles. The form must provide a high-quality finish even after these repeated cycles of use. Concrete forms for long tunnels are generally constructed with a steel skin and integrally welded steel bracing. They come with carriers designed to help with stripping and moving the form. If the form is a full-round form, the rails will be built into the form invert section. Arch-only forms normally have sections of track that must be advanced with the form. The form may range from hundreds of feet long in the case of a tunnel bored by a tunnel boring machine (TBM) to just 30 ft (9 m) long in the case of a drill-and-blast excavated tunnel or a soft ground tunnel. Steel tunnel forms are generally designed to be collapsed and moved in sections. Telescopic forms have the additional feature that the collapsed form section can be advanced through a long series of erected forms and be erected at the other end (see Figures 2.3 and 2.4).

Wood forms are also used for casting tunnel linings but only for unusual geometries or one-time uses where a steel form would not be economical (e.g., the intersection of two arched tunnels at a subway station; see Figure 2.5). The wood forms have to resist the same forces as steel forms and involve the added complication that they cannot be externally vibrated; special provisions have to be made for the introduction of internal vibrators, or a self-consolidating concrete mixture must be used. Wood forms with complicated geometries or large dimensions are typically shop-built in modules for shipping purposes and for easier handling underground. Restricted

Courtesy of Shane Yanagisawa.

Figure 2.3 Telescopic formwork—collapsed arch and invert form on carrier inside expanded full round form

Courtesy of Shane Yanagisawa.

Figure 2.4 Telescopic formwork—expanded full-round tunnel form with concrete injector in place

access to the tunnel normally prevents the final assembly of steel or wood forms with an overhead crane. The forms must be designed with the final assembly location in mind.

It is becoming more common for waterproof membranes to be placed between the initial ground support and the final lining. In such cases, the centering pins that are usually used to brace the form against uplift and shifting cannot be used. Instead, the forms must be internally braced against movement (as shown in Figure 2.6).

Schedules

Tunnel projects normally have tight schedules and high daily operating costs. Tunnel lining installation typically takes place 24 h a day, using multiple shifts of workers. When long (100 to

Courtesy of Shane Yanagisawa.

Figure 2.5 Modular wood forms shop-built for subway station

Courtesy of Shane Yanagisawa.

Figure 2.6 Waterproofing membrane in place with sealed dowels to support template bars

300 ft, or 30 to 91 m) reinforced sections of tunnel are placed using multiple form sections, work takes place on a three-shifts-a-day basis. The first shift places the concrete; the second shift cleans up the next placement area and places reinforcement; and the third shift strips and resets the form for the next placement.

Testing

The objectives of testing underground concrete, including CIP concrete, are similar to the objectives of testing aboveground concrete. They are

- To ensure that the mixture and application perform according to specifications under real conditions and on-site with the actual equipment that will be used;

- To verify that these tests meet the requirements of the design and function of the facility; and
- To ensure that the adequacy and consistency of both the material and its application are maintained by continuing to test throughout construction.

However, in underground placement of concrete, producing representative concrete samples for testing is often more challenging than for aboveground concrete. The location of concrete placement changes every day, and there is often no place underground to store test cylinders, even for a day. A compromise often has to be reached between the engineer and the contractor about the best place to take the samples and the best way to store and transport them for testing.

MATERIAL CHARACTERISTICS

The characteristics of CIP concrete are the typical underground concrete characteristics described in Chapter 1. Unique characteristics and mixture considerations are discussed where appropriate in the following sections related to design and construction.

KEY DESIGN ISSUES

The two key issues in the design and specification of tunnel concrete linings are (1) how to minimize concrete cracking and (2) when to allow stripping of the tunnel formwork. For serviceability reasons, the designer wants to minimize cracking by limiting thermal expansion, early shrinkage cracks, and long-term shrinkage. For schedule and cost reasons, the contractor wants to have enough early strength to strip the formwork as soon as is prudent. Both parties want to guarantee achievement of the specified minimum compressive strength. These issues are more closely related than they would appear at first glance.

North American building codes and the American Concrete Institute (ACI) code and guidelines give little guidance on the subject of tunnel-lining concrete. Fortunately the Germans, Austrians, and Swiss have developed codes, guidelines, and model specifications that provide ample advice on the subject. European codes, reviewed in this chapter, provide good guidance on what strength the concrete should have before the tunnel form can be stripped and how to avoid cracks in the fresh concrete but don't describe well how to predict temperature differentials and strength gain or how to verify these properties in the field. This section also discusses a method to predict temperature differentials and strength gain using a semi-adiabatic calorimeter with the appropriate software.

In drill-and-blast and soft ground tunnels, sections of concrete can be thicker than 3 ft (914 mm). Drill-and-blast tunnels can have substantial overexcavation (overbreak) from the blasting process. Soft ground tunnels can have inverts 4 to 6 ft (1 to 2 m) thick. Concrete thicker than 3 ft (914 mm) is often called out in North American specifications as "mass concrete," and special requirements are invoked to minimize cracking due to high thermal differentials within the concrete. But the method to analyze the thermal stresses is left to the contractor. The methods described in this chapter can also be used to model temperature differentials and strength gain in thick concrete sections commonly found in tunnels.

Other issues tend to lie in wait until touched upon at the worst time. These items are brought up for discussion later in the "Other Common Problems" section of this chapter.

Early Form Stripping and Crack Formation Prevention

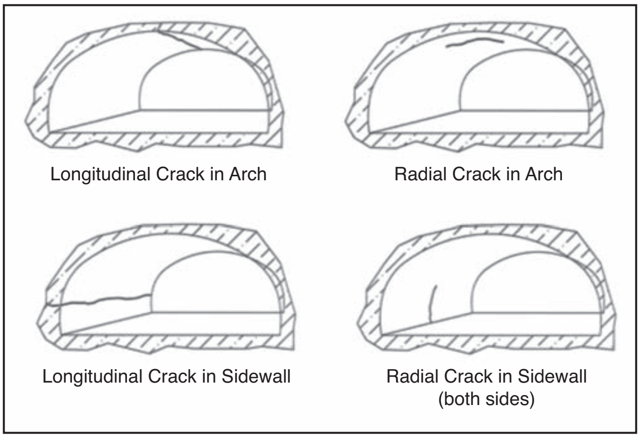
In the classic book *Practical Tunnel Driving* (Richardson and Mayo 1941), the authors note, in the "Time of Stripping" section, that specifications are "often based on open cut practice, where the arch must have gained sufficient strength to resist the distortion before the support is removed," but that in a rock tunnel the concrete cannot distort. Further on, they note that specifications for the 28½-ft (8.5-m) wide Pennsylvania Turnpike tunnels allow the arch forms to be stripped when the concrete attains a compressive strength of 600 psi (4 N/mm^2). Arguments about when to strip tunnel forms are still common on job sites today.

Designers' concrete mixture proportioning specifications often specify the maximum amount of cement permitted. This is an attempt to limit the heat of hydration and thermal differential expansion—and thereby cracking. Occasionally, designers specify slow-setting cements in an attempt to minimize thermal gradients. Sometimes the designer specifies the minimum amount of cement as a means of ensuring that the concrete will achieve the required strength. However, specifications often require concrete strengths from 1,000 to 2,500 psi (7 to 17 N/mm^2) or a percent of ultimate design strength (such as 50%) before a tunnel form can be stripped. With normally proportioned concrete, these requirements are not practically achievable within 12 to 14 h.

Often, such stripping strength requirements are borrowed from specifications for above-ground elements such as elevated slabs and beams, and the borrowers do not consider that they may be inappropriate for underground construction. Stripping strength requirements should take into account the shape of the cast concrete, the boundary conditions of the lining, and the common knowledge of the tunnel industry. CIP tunnel linings are installed after the tunnel excavation is complete and the ground support has already been installed. The time elapsed between ground support and tunnel lining installation is normally long enough to allow the ground to redistribute stresses and cease short-term movement. The circular, parabolic, and horseshoe cross sections common in tunnel linings show very low internal bending stresses when analyzed for self support. Shotcrete sprayed arches support themselves immediately, despite the low initial strength of the accelerated shotcrete. Contractors' experience is that they can set, place, and strip tunnel forms on a daily basis in tunnels with diameters 30 ft (9 m) or larger, but their ability to prove this on an analytical basis is lacking.

From the designer's perspective, as previously noted, the key issue is to minimize cracking by limiting thermal expansion and long-term shrinkage.

The ACI 318 Building Code does not provide specific direction on minimum stripping times for tunnel forms. The only relevant guidance in the ACI literature is publication 347R *Guide to Formwork for Concrete*, Section 6.6.4, which recommends that "at the start of a tunnel arch concreting operation, the minimum stripping time be 12 h for exposed surfaces and 8 h for construction joints." If the specifications provide for a reduced minimum stripping time based on site experience, such reductions should be in time increments of 30 min or less and should be established by laboratory tests, visual inspection, and surface scratching of sample areas exposed by opening the form access covers. In Austria, Germany, and Switzerland, designers have produced extensive codes, guidelines, and model specifications that emphasize crack control in CIP linings without requiring high early concrete strengths for form stripping. A common theme of these specifications is that certain types of cracks form at very early stages of concrete curing and can be minimized (Figure 2.7). Cracks are limited by removing the tunnel form about 10 to 12 h after concrete placement when the concrete has only 218 to 435 psi (1.5 to 3 N/mm^2) compressive strength. The form must be stripped before the concrete reaches 725 psi (5 N/mm^2). German



Source: Wilhelm and Grube 2000.

Figure 2.7 Typical cracks in tunnel linings

specifications require that forms be stripped before the concrete compressive strength exceeds 435 psi (3 N/mm^2) above the minimum for stripping strength. Early form removal prevents heat from accumulating in the early stages of cement hydration and allows the green concrete to expand and contract with less restraint. The Austrian guidelines recommend a slip sheet behind the final lining if additional crack reduction is desired. A waterproof membrane between the initial and final lining can serve as this slip sheet. The smoothing shotcrete and compressible fleece behind the membrane further reduce the interlock between the irregular substrate and the concrete.

Under these circumstances, the concrete is very green, so the specifications require curing at a constant temperature in a high-humidity environment, with protection from high air velocities and excessive cooling during the initial cure. For tunnels with large cross sections and short forms, a framework assembled to fit within the tunnel profile that is draped with insulated curing blankets is sometimes suggested to protect the green concrete. Curing using water mist or curing compounds is also normally required, although Austrian specifications do not require curing compounds or mists if the humidity can be maintained at higher than 90% and air velocities can be kept below 200 ft/min (1 m/s).

These model specifications recommend that concrete batch temperatures not exceed 64°F (18°C) with a maximum temperature of 77°F (25°C). Cement should also be the minimum required to limit long-term shrinkage and is suggested in the range of 474 to 540 lb/yd^3 (280 to 320 kg/m^3). Fly ash in the range of 85 to 135 lb/yd^3 (50 to 80 kg/m^3) is recommended to reduce the heat of hydration while providing long-term strength. Aggregates should not exceed 1.25 in. (32 mm). Water-to-cementitious-materials (W/C) ratios are suggested to be in the 0.50–0.55 range, much higher than typical American practice. The concrete mixture ultimate compressive strength should not be significantly higher than the strength required for loads and serviceability.

Finally, Austrian, German, and Swiss model tunnel specifications require that a method be used to determine the in-place concrete strength prior to form removal. One suggestion is to strip the bulkhead as soon as possible and use a Schmidt hammer to determine the compressive strength. The Austrian specifications suggest that the bulkhead be stripped after 4 to 5 h for strength checks. Another approach is to cast test cylinders with integral caps and individually cure the cylinders in insulated compartments. The cylinders are to be broken prior to stripping

Courtesy of Christian Neumann.

Figure 2.8 European-style form for NATM tunnel

the form. These specifications state that tunnel linings with concrete radii up to 19 ft (6 m) can be stripped when concrete strengths reach 290 psi (2 N/mm^2). Special analysis is required in cases of concrete liners that have unusual cross sections, large radii, or nonuniform thicknesses. It is recommended that niche formwork and reveals be detachable from the main form to allow early stripping without breaking off sharp edges.

By using the early form stripping criteria combined with a modern form, European contractors have been able to strip, set, and pour the CIP tunnel linings for a highway tunnel in one 12-h shift with minor cleanup work being performed on the second shift. The single section combined wall and arch form had a span of 43 ft (13 m), a length of 33 ft (10 m), and required 324 yd^3 (248 m^3) of concrete to fill the form (a typical example of European style formwork is shown in Figure 2.8).

In summary, the overall approach of the Austrian, German, and Swiss concrete tunnel lining codes, guidelines, and specifications is to strip the concrete at very low compressive strengths to allow expansion and contraction of the concrete in a temperature- and humidity-controlled environment. The heat buildup of the concrete is minimized because there is less need to add more cement to meet the form stripping strengths commonly called for in the United States. The ultimate concrete strength is closer to the design strength because the compressive strength target for stripping the formwork is lower. This approach is recommended.

One of the author's experience is that the amount of cement required to meet the stripping strength requirement of 600 psi (4 N/mm^2) in 12 h has always resulted in a concrete mixture that exceeded the 4,000-psi (27-N/mm^2), 28-d strength requirement. This observation is limited to conventional mixtures with fly ash or slag cement at a 20% replacement rate and no accelerator usage.

Identifying and Addressing Potential Thermal Problems

For large or extensive tunnels, structural analysis should be done to establish the minimum concrete strength required for form removal. The contractor should show that the selected concrete mixture can meet the time and strength criteria. The rate of strength gain for the proposed concrete mixture can be established by using a combination of modeling software and test cylinders of the proposed concrete. The concrete test cylinders are instrumented and placed in a drum calorimeter to detect the heat generation and loss. Test cylinders broken as part of the program establish a heat/strength curve. The modeling software can take into account the mixture design and the thermal characteristics of the surrounding environment—including the form, tunnel air temperature, and ground temperature—and provide heat generation and strength curves for various ground conditions. This modeling can establish the performance of the concrete before any fieldwork begins. Examples of this software would be the Quadrel system by Digital Site Systems (Pittsburgh, PA) and the 4C-Temp&Stress system by Germann Instruments (Evanston, IL). The contractor should select the mixture design and model the strength development. The software can also model the temperature differentials in thick concrete sections and check for the possibility of thermal cracking.

The model will show that ground temperature has a substantial effect on concrete strength gain when the concrete is placed directly against shotcrete or rock substrate. The substrate usually has a temperature in the 50°–60°F (10°–15°C) range and acts as a large heat sink. Concrete placed directly against rock will drop in temperature before rising again as the cement begins to hydrate. A temperature profile across the concrete section is highest in the middle and drops toward the rock and the form skin. Because waterproofing membranes with fleece have an insulating effect on the concrete, the concrete will not cool down much before the hydration process kicks in and raises the concrete temperature. The temperature profile is more uniform across the concrete section with the lowest temperatures at the form skin. Consequently, strength gain is faster for the same tunnel concrete mixture placed against waterproof lining membranes with fleece than against bare rock or shotcrete. Shotcrete smoothness criteria for waterproofing membranes tend to reduce thickness differences occurring within short distances and therefore reduce sharp temperature differentials in curing concrete. If a sudden change in lining thickness is unavoidable, a construction joint at the thickness change will be better than a continuous placement to avoid cracking.

The very early stages of concrete strength development are reduced by air entrainment in the normal ranges of 3% to 5%. Non-air-entrained concrete shows earlier strength gain in the 0–1,000 psi (0–7 N/mm^2) range because the mixture is more compact. The impact of air entrainment can be countered by reduction in W/C or adding other admixtures.

The maturity meter is a recognized method (ASTM C1074) to determine the strength of in-place concrete from thermocouples placed in the concrete. This method does not predict concrete performance but provides a real-time estimate of in-situ concrete strength and is further discussed in the “Quality Control” section later in this chapter.

Other Common Problems

Concrete mixture design should follow normal good practices but should also consider the limitations of tunnel forms. Access to the freshly placed concrete is limited. The tunnel concrete is usually placed by injection through the form or by placement via a slickline over the form. The concrete needs to flow as a plastic mass toward its final location without segregation. The form is usually externally vibrated, with about 30 ft^2 (2.8 m^2) of area serviced by each vibrator. Internal vibrators are limited to application below the springline because of the form window locations. Tightly spaced reinforcement blocking the form windows may also limit the use of internal

Courtesy of Shane Yanagisawa.

Figure 2.9 Tunnel concrete must be able to flow around reinforcement with limited use of immersion vibrators

vibrators. Tightly spaced reinforcement blocking the form windows may also limit the use of internal vibrators (Figure 2.9).

Concrete specifications should permit slumps up to 8 in. (203 mm) as long as the mixture is plastic and does not segregate. With modern superplasticizers, high slumps with reasonable water-to-cement ratios are easily achieved without retardation of set time or early strength gain.

Coarse aggregates for underground concrete, where the concrete is injected through a lining to fill a space and encapsulate reinforcing steel, are usually smaller than what is normal for aboveground practice where access is good for internal vibration. Highly congested zones of reinforcement will require smaller coarse aggregate. Generally speaking, aggregates in the 1 in. (25 m) and smaller range have been found to allow good flow through the form and around the reinforcement.

Viscosity-modifying admixtures can be used for special applications requiring higher slumps or for self-consolidating concrete (SCC). The workability of SCC is determined by slump flow (ASTM C1611/C1611M), which is typically 24 to 30 in. (61 to 76 cm) for underground applications. Pumped concrete should be checked during trial batches for excessive bleeding. Research has shown that a bleeding mixture can plug a pump line when pumping resumes after a brief stop.

Polypropylene microsynthetic fibers are becoming popular for adding fire resistance to concrete linings. In normal concrete, high temperatures that can occur in a tunnel fire cause the free water in concrete to flash into steam and spall the concrete, thus exposing more concrete to rapid heating and spalling. The microfibers in the concrete matrix melt at a lower temperature and provide a pathway for steam to escape, reducing the spalling. The benefits are more pronounced in dense concretes (strengths in excess of 5,000 psi, or 35 N/mm^2). Trial batches of microfiber concrete are necessary to establish superplasticizer dosages and check air-entrainment levels. The high surface area of the fibers requires more superplasticizer. Coatings on the fibers can cause unexpected levels of air entrainment. Section 6.1 of the European Standard EN 1992 Eurocode 2 refers to the use of 3.4 lb/yd^3 (2 kg/m^3) of monofilament polypropylene microfibers to control explosive spalling in high-strength concrete (BS EN 1992). This is often used as a de facto standard as it has been proven to be a conservative value, and its use eliminates the need for expensive spalling resistance testing.

When concrete with fibers is to be placed around steel reinforcement, the interaction between the two must be checked to ensure that no cavities are created around the reinforcement. A mock-up test of a concrete placement may be necessary to be sure that the concrete mixture completely envelops the reinforcement using the planned form vibration method.

Designers should avoid congestion of reinforcement at lap splices and intersections where the concrete is likely to receive only external form vibration. A good example would be an arch intersection at a subway station where the reinforcement is tightly spaced #11 bars with long lap lengths and small stirrups. If possible, the reinforcement should be spread out using larger stirrups or the use of mechanical rebar connectors can be considered in lieu of splices at congested areas.

KEY CONSTRUCTION ISSUES

As noted earlier, one of the key construction issues associated with CIP concrete is access to the tunnel, which also has a significant impact on how easily inspection and testing can be completed. Before beginning a discussion of how CIP concrete is delivered to underground projects and inspected after placement, it is helpful to understand steps the contractor can take before beginning construction to help ensure successful concrete placement.

To help mitigate as many potential issues as possible before they become actual problems during construction, the contractor should always prepare trial batches of the intended concrete mixture, unless the contractor has a long acquaintance with the particular mixture. The contractor should not accept a mixture design from a ready-mix supplier without stipulating that the contractor must witness trial batches and perform a field test. This helps the contractor understand local concrete materials, mixtures, and practices. The laboratory setting allows the contractor to see the mixture, understand its behavior, and make adjustments without the pressure of crews standing by with a form waiting to be filled. Knowing what the mixture should look like and how it should behave will give the contractor more confidence in the field when accepting, adjusting, or rejecting a batch.

A good batch plant will accurately weigh all ingredients, and—if the ingredients are of consistent quality—will be able to provide a uniform product simply by adjusting the water added to the batch. The total water in the mixture should closely match the mixture design. Batch plants should check sand moisture content frequently and manage aggregate stockpiles to avoid large variations in moisture content. Both of these activities are matters of good batch plant management and should be enforced by the contractor, as the ability to deliver a consistent mixture is very important—particularly when concrete is being pumped. The aggregate stockpiles should be protected against large temperature swings that might cause concrete to be batched too cold or too hot. The stockpiles may need to be shaded and sprinkled in hot weather or covered and heated in cold weather. The plant may require hot water heaters or facilities to add ice. The batch plant must be in good operating order and should be certified by the National Ready Mixed Concrete Association.

The contractor should take into account the time between batching and placement, including delivery times, batch temperatures, and external temperatures and their effect on slump loss. If necessary, more superplasticizer can be used to stay within the water-to-cement ratio at the point of delivery if remixing is possible and the delays of remixing are acceptable. A later-generation

Courtesy of Shane Yanagisawa.

Figure 2.10 Moran cars waiting to be filled with concrete at the surface

retarder (hydration stabilizer type) can be used in small quantities for longer delivery times. With this type of retarder, slump loss is minimal.

Delivery and Placement

Concrete delivery from the batch plant into the tunnel may require only one method of conveyance, or it may require a delivery chain involving several conveyances. If the batch plant is offsite or not conveniently located near the portal, a mixer truck will generally deliver concrete to the portal. Within the tunnel, delivery is typically by railcar in smaller tunnels and by mixer truck in larger tunnels. The railcars may be open-top agitator dump cars or “Moran” cars, which look like milk bottles lying on their sides and have internal flights like a concrete mixer (as shown in Figure 2.10). Moran cars can be coupled together at the point of delivery so that, beginning with the last car, each car discharges into the tail end of the other, forming a delivery chain that pushes concrete up through the first car and into the concrete car hopper. Moran cars and agitator dump cars usually have the ability to agitate concrete but are not capable of thoroughly mixing in additives or mixing together the initial ingredients.

Whatever the delivery method, it is recommended that contractors use a minimum 5-in.-diameter pipe or hose to pump tunnel liner concrete into the form, in order to avoid plugging and maximize flow. A power-activated shutoff valve should be placed at the bottom of long vertical drops if the concrete pump is at the surface, and the shutoff valve should be closed immediately when pumping stops to prevent air lock in the line. Good coordination is required between the pump operator and placing crew to ensure that the drop line remains full at all times so that no concrete falls in the line.

Some contractors prefer to drop the concrete down a pipe to a concrete pump or railcar at the bottom of the shaft. In the case of long vertical drops, a static remixer is usually installed at the bottom of the pipe primarily to absorb the impact of falling concrete and produce a manageable delivery stream to the pump hopper. A well-proportioned concrete mixture can survive a long drop without remixing; however, the static remixer is a good way to absorb energy.

Some projects lend themselves to the possibility of concrete deliveries via a series of drop shafts from the surface spaced along the length of the alignment. The drop holes are located in truck-accessible locations and drilled down to the crown of the tunnel. This method is not

Courtesy of Shane Yanagisawa.

Figure 2.11 Standpipes to pump concrete through the formwork from a single location

possible where there is no legal access at the surface or where the depth, location, and geology of the tunnel make drilling a drop hole uneconomical.

Pumped concrete should rely on a minimum of superplasticizer or other admixtures to maintain slump if the pump line is longer than 300 ft (91 m). It should be noted that the effects of the superplasticizer decay with time, and a stoppage that exceeds the superplasticizer's life will result in a plugged pump line that cannot be restarted. Stiff concrete placed through the form may not be adequately consolidated if only external form vibration is available. Reasonable form stripping strengths can allow higher water-to-cement ratios and reduced use of superplasticizers.

For tunnels where the invert has been placed first, concrete placement behind the form begins with the placement of concrete along the invert/arch joint. On larger forms where there is room for windows, the concrete is placed through the windows initially, and the concrete at the joint is consolidated with an internal vibrator. Once the concrete builds up to the windows, the concrete is placed by injection through ports in the arch or by a slickline at the 12 o'clock position. Injection ports do a better job of directing the concrete to get an even load of fresh concrete on the form. For concrete placed in the upper part of the tunnel, standard practice in Europe is to pump concrete through standpipes fixed to the form (as shown in Figure 2.11). The standpipes have guillotine valves and are cleaned while the concrete is still fresh.

Small-diameter tunnels may not have room for a concrete injector car and may require use of a slickline over the top of the form. Hung from chains or rollers, the slickline reaches all the way back to the end of the form. As the tunnel form is filled with concrete, the slickline is withdrawn. Now that the injector car method has been refined, slicklines are no longer as popular.

The concrete level is monitored by looking through form windows that are still open and by looking over the top of the form where a few bulkhead panels will have been left out. On forms with vertical bulkhead ends, the last stroke of the concrete pump is the most critical. Not placing enough concrete will leave a void at the top arch, while placing even a little bit too much concrete will blow the vertical bulkhead out.

Special care must be taken when injecting concrete into a tunnel form that is set between two previously placed concrete arches (such as in a closure placement). If the cavity outside the form is overpressurized with concrete, there is no bulkhead on either end of the form that can give way. The concrete is basically an incompressible substance, and the concrete pumping pressure is powerful enough to buckle the form skin if the cavity is overfilled. One option to avoid this problem is to install retractable pipes in the arch for filling observation. The cavity left by the pipe can also serve as a packer hole for later injection of contact or void-filling grout.

Arguments about the permissible drop of concrete should be avoided by ensuring that the form and associated drops are reviewed during early project submittals. Specifications that are blindly applied from aboveground construction often limit the free fall of concrete to 5 ft (1.5 m). The concrete placed in an arch form typically will not fall but will flow down the arch as a mass. Once the toe is built up, the concrete is typically advanced as a sloping mass toward the bulkhead with no segregation. Use of elephant trunks (thin wall, flexible, concrete placement hoses) to place concrete behind the form is not a workable solution since they often break free, are difficult if not impossible to retrieve, and therefore can become cast into the concrete. If there is a dispute about free fall and segregation, the contractor should be allowed to demonstrate through a field trial that the placement method is satisfactory.

Strength Testing

The tunnel form should not be stripped until the strength of the concrete is known. A good indication of the concrete strength can be gained by removing the bulkhead wall, starting with the concrete that was placed first and moving up to the freshest (usually from the bottom to the top), and examining the exposed concrete, then stopping if the concrete appears soft. This method reveals a cross section of the placement and provides a good indication of what is going on inside the concrete.

A more quantitative method is to check the strength gain using a maturity meter connected to thermocouples embedded in the concrete. This method is described by ASTM C1074. Maturity testing requires developing a maturity curve for a specific concrete mixture that correlates the strength development to the curing time and concrete temperature history. To develop a maturity curve, a laboratory batch of concrete is placed in a large number of standard concrete test cylinders. One test cylinder is instrumented with thermocouples connected to a maturity meter. The maturity meter is capable of integrating time and temperature into a single number based on a mathematical function (either Arrhenius or Nurse–Saul). Modern maturity meters are downloadable to a portable computer. The instrumented cylinder and the other cylinders are all cured under the same conditions, and the cylinders are broken in pairs or triplets during the strength/time interval of interest to the project. The maturity number is noted at the time of cylinder breaks. Using the test information, a strength maturity curve is established.

Special handling is required when testing concrete cylinders at very early ages of 8 to 24 h. The cylinders have to be slit open instead of using a cylinder mold splitting tool. Test cylinders must be broken using sulfur-capping compounds instead of neoprene caps to get consistent reliable breaks.

Note that little literature exists on the early (4 to 18 h) strength testing of normal weight concrete. Experience shows that, up to compressive strengths of about 600 psi (4 N/mm^2), normal test cylinders tend to fail in a plastic manner without a clear, sharp break. The peak compressive strength is very dependent on the rate of loading. Around 500 to 600 psi (3.5 to 4 N/mm^2), the concrete shows the quick drop in strength associated with a normal cylinder break. While calculations will show that a lower compressive strength in the 200–400 psi (1.5–3 N/mm^2) range is adequate for form stripping, the 600-psi mark has been found to be a stripping strength mutually acceptable to the designer and contractor for tunnels approximately 20 ft (6 m) in diameter and smaller. Larger-diameter tunnels should be analyzed for the minimum practical stripping strength. Once the curve is developed, the field concrete placement is instrumented with thermocouples and connected to the maturity meter. The meter usually has several channels so different parts of the placement can be checked for temperature rise and maturity. The maturity number calculated by the meter can be correlated back to the strength maturity curve to get the estimated strength. This method has been shown to work well in numerous applications as long as the concrete mixture placed is the same as the mixture used to establish the strength maturity curve.

If a long series of tunnel sections needs to be placed, the initial sections should be instrumented with thermocouples and the strength development curves tracked. Data from the maturity meter can be downloaded to a computer, and the maturity curves can be plotted. Once the strength gain characteristics of the concrete mixture are established and shown to be consistent for a given set of field placement conditions (e.g., batch temperature, air temperature, and rock temperature), one can be comfortable with stripping the form after the number of hours determined, unless temperature or other environmental or batch mixture conditions change. The maturity meter can be used as an occasional quality control check of the mixture by checking the time/temperature curve against previous placements. However, it should be noted that concrete with different water-to-cement ratios, cementitious materials, or admixtures will produce a different maturity curve. The use of the maturity meter is a complement to, not a supplement for, the modeling of concrete strength gain and temperature gradients done during the initial concrete trial mixture stage.

Other Common Problems

On the surface, reinforcement is usually placed against a wall or slab form that provides a location reference for the reinforcement. Underground, the reinforcement is placed against a rough arch or wall with no clear reference. Reinforcement to form clearance requirements are normally 2 in. (50 mm), so there is not much room for error. The tied reinforcement cannot be pushed into place by the arch form. The best practice is to survey in the location of longitudinal bars that provide a template for the installation of the contract-required reinforcement.

Special care should be taken when moving the tunnel form ahead and oiling the surface so that the oil does not splash on the reinforcement. It may be necessary to mop on the form oil instead of spraying it. Various steel form coatings other than oil have been tried with limited success.

Bulkheads are a crucial but often neglected part of setting up a tunnel form. In drill-and-blast tunnels, bulkheads always have an irregular shape that varies with each placement. Bulkheads must be able to handle 2 to 3 ft (0.6 to 0.9 m) of overbreak plus the gravity-induced pressure. In very blocky ground, overbreak may be even greater than 3 ft (0.9 m) (Figure 2.12).

In TBM tunnels, the tunnel form end may not be centered in the TBM bore but may be offset to meet a specified line and grade. Bulkheads in bored tunnels must be able to handle these variations.

Bulkheads must allow for penetrations and waterstop (if required). For ease of handling and to allow incremental placement during concrete placements, bulkheads are typically placed in small sections. Bulkhead forms in tunnels with waterproof linings must be designed to work without using backstops pinned to the tunnel wall. This requires a cantilever style of bulkhead with a strong form to resist the bending moments imposed by the bulkhead.

Courtesy of Shane Yanagisawa.

Figure 2.12 Wooden bulkhead against waterproofing in drill-and-blast tunnel

Full vertical bulkheads are generally required for steel-reinforced concrete; however, in many cases, the tunnel lining is unreinforced. Unreinforced tunnels can use a sloping joint bulkhead instead. Sloping joints occur where the last part of the concrete placement is left at its natural angle of repose and a portion of the last form is left uncovered by concrete. For example, a sloping joint may be considered where the tunnel form is in multiple sections (e.g., 9 sections × 25 ft [8 m]/section). The last form section with the sloping joint is left in place to provide a form for the start of the next placement, and the remaining form sections will be erected on the other side of the sloping joint. The next concrete placement will fill the top part of the sloping joint.

Sloping joints have the advantage that an exact quantity of concrete is not required to fill the form and all the concrete in the delivery system can be discharged into the forms. With a full vertical bulkhead, an exact quantity is required to fill the arch. Because estimating the quantity to finish is difficult, overordering concrete is the normal practice, and the leftover concrete in the pump line and delivery system must be thrown away. Sloping joints also eliminate the hazard of blowing out the bulkhead.

Many engineers do not like the sloping joint because the joint's toe is a thin sliver of concrete. This objection has been largely overcome by use of a modified sloping toe joint where a partial vertical bulkhead is used. The concrete at the joint is vertical to the top of the bulkhead and then slopes back from there. Sloping joint bulkheads work in situations where the overall tunnel form is long in relation to the length of the sloping joint.

A few precautions are in order when using sloping joints to ensure that the concrete in the next placement completely fills the top arch of the tunnel at the joint. Otherwise, a patch of thin concrete over the form arch and a void above are left. If the concrete is injected through the form crown, the last port covered with concrete should be probe drilled once the concrete has achieved final set. If the concrete is solid all the way to the ground, the next injection port toward the end of the joint should be used for the subsequent concrete placement. If a void is encountered over the form, then the next port toward the previous concrete placement should be probe-drilled until a continuous thickness of concrete is encountered. The idea is to place the concrete through the injection port nearest to the sloping joint and not be misled by a skim coat of concrete over the port. If the concrete is placed using a slickline over the top of the form, the slickline should be positioned so that the end of the pipe is as close to the sloping joint as possible.

Unanticipated cold joints can happen when equipment breaks down. Having a second concrete source at the surface can protect against a batch plant breakdown. Keeping a spare concrete pump on hand is also prudent if the pump location allows a rapid change-out of the broken pump. Otherwise, the contractor must be prepared to clear the pumpline with air or high-pressure water. When, despite all efforts, the cold joint occurs, the next placement should proceed as a continuation of the sloping joint.

Water control is just as important during concrete-lining operations as during tunneling. Water flowing in through the excavated lining must be controlled to prevent washout of the fresh concrete. Underground structures with waterproof membrane linings solve the problem by placing an impermeable barrier between the final lining and the water. The water is drained away to prevent a hydrostatic pressure buildup.

For tunnel linings placed against bare rock, the basic methods are to either cut off the water by grouting or direct the water away from the tunnel lining by use of panning and drain pipes. Prior to lining the tunnel with concrete, the excavated tunnel should be examined to determine the extent and location of water inflows so that the size and extent of the water handling or grouting requirements may be determined. When the tunnel lining occurs at the end of the excavation process, sufficient time may have passed to lower the water head around the tunnel so that the water flows into the tunnel through the invert only. In some areas water may be seeping through a fissure above the invert.

Grouting is effective only if the water flow comes from a few well-defined areas that can be intercepted by drilling and grouting the water-bearing zone. High-volume flows that are more than just seeps have to be temporarily shut off or diverted for grouting to take effect. Often grouting off a water leak leads to a pressure buildup and a leak from a new location. Entire books have been written on the subject of grouting, so the main point is that grouting to cut off water is an option but only in limited circumstances.

The other strategy is to place a panning material against the seepage zone and direct the water to a slotted pipe that freely drains the water. Previously, panning was corrugated steel pinned to the rock face but now is usually a flexible plastic dimple drainboard. The side facing the ground has a fleece lining that allows only water to flow through. The panning and drainpipe are placed immediately in front of the planned concrete placement for that day. The concrete is placed against the drainpipe and the panning so the water is forced into the drainpipe. In areas with extensive water inflow and full circular steel sets, open drain rock may be placed on the tunnel invert and covered with a geotextile. The system must allow free flow of the water because the fresh concrete can only exert pressure resulting from its own weight. The drainpipe may be placed continuously along the invert or may be designed to turn out of the tunnel form if only a short section of tunnel is producing water. Once the tunnel lining has reached its design strength, the drainpipe can be pressure grouted with cementitious grout.

Curing without additional measures such as a misting or curing compound should be permitted provided the relative humidity is kept above 90% and air velocities are controlled. Temperatures should also be kept in a consistent range to avoid shocking the fresh concrete and inducing cracks by thermal differential stresses.

QUALITY CONTROL

Quality control is difficult in tunnels where access is limited, and producing representative concrete test cylinders for testing is a challenge. The concrete injection point into the form is not a safe or convenient place to collect a sample. The daily process of cleaning the invert, placing reinforcement, and advancing the form does not leave much room for safe test-cylinder storage to initially cure the test cylinders.

The transport of green test cylinders out of the tunnel is susceptible to mishandling and damage. Unless special arrangements are made, concrete test cylinders cast inside a tunnel will be moved, loaded, and transported out of the tunnel by the craft labor with no special regard for the delicate nature of the fresh concrete. This will result in inconsistent test results and unnecessary retesting. Test-cylinder storage locations inside the tunnel must be protected from vibrations induced by equipment operating nearby.

At the beginning of the tunnel-lining process, the air content and unit weight can be checked with an air meter as close to the heading as possible in order to establish air loss so compensation can be done if air entrainment is required. Then the test cylinders can be taken at the nearest convenient delivery point where the last adjustments may be made to the concrete slump. This may be where the concrete truck makes deliveries to the pump, or it may be at the portal or shaft.

Defects observed in the concrete after form stripping are generally honeycombing (rock pockets), failure to fully encapsulate the reinforcing steel, cold joints, air inclusions (bug holes), and sand streaks.

Honeycombs can be caused by

- Loose-fitting form joints or windows that allow mortar to seep through,
- Washout from groundwater,
- Inadequate vibration of concrete, or
- Improper mixture design (low workability, large aggregate, premature set).

Honeycombs should be chipped back to sound rock and replaced with a dry packed grout.

Failure to fully encapsulate the steel reinforcement can be caused by

- Improper mixture design (low workability, large aggregate, premature set),
- Reinforcement placed too close to the form,
- Congested reinforcement in one place (e.g., multiple lap splices back to back),
- Inadequate vibration of concrete, or
- Use of fibers in an improperly proportioned concrete mixture.

The concrete around exposed reinforcement should be chipped back to a clearance of about 2 in. (5 cm) and the cavity dry packed with grout. Repairs are done using a low water/cement-ratio mortar to reduce shrinkage cracking around the edges of the repairs. A bonding compound or grout slurry should be applied to the base surface. The source of the problem should be eliminated before the next concrete placement. Larger repair areas may require hydro-demolition of the affected area and use of repair materials containing coarse aggregates.

Cold joints are caused by a placement delayed long enough to allow the concrete already in the form to reach initial set, and subsequently the concrete is not integrated into the mass. Because unreinforced tunnel linings normally are designed to work in compression, a cold joint is not a fatal flaw. Reinforced linings may have a tension or shear region that the reinforcement is designed to resist. Although normally more of a cosmetic flaw, a cold joint is more likely to seep water and provides a path for water to reach the reinforcement. Normally the best thing to do with a cold joint is leave it alone. If the joint seeps excessively, polyurethane grout injection may be performed. It is important to eliminate the source of delay in the concrete delivery chain. Cold joints should not be confused with lift lines that occur when there is a momentary delay between deliveries of concrete to the form.

Air inclusions (bugholes) are the result of air being entrapped against the form and usually occur on the sloping sides of the form below springline. Bugholes are caused by

- Form not completely coated with release agent or evenly coated at the proper rate,
- Release agent not compatible with the concrete or form material,
- Form that is not sufficiently clean, or
- Inadequate vibration or too much vibration.

Bugholes are usually considered a cosmetic defect. Unless the holes need to be filled for reasons of appearance, they should be left alone and efforts concentrated on reducing them in future placements.

Sand streaks are caused by excessive concrete bleeding along the form skin. Harsh concrete mixtures without air entrainment are susceptible to sand streaks, while well-proportioned mixtures with air entrainment rarely have this problem. The best course is to leave the concrete as is and change the concrete mixture for subsequent placements.

In general, repairs of concrete defects are never as good as when the concrete has been properly proportioned and placed to begin with. It is better to spend the time and energy upfront to ensure that the concrete placing system is a complete and smoothly operating package.

CONCLUSION

The means of delivery, placement, type of formwork, and needs of the concrete mixture design for underground tunnel liner concrete are different than in aboveground work. This chapter has aimed to provide some guidance on resolving the key issues of crack control and timely form removal, and to demonstrate that the two issues do not have to be incompatible. The European tunneling community has led the way by coming up with comprehensive codes and guidelines for tunnel concrete. The North American tunneling community should contribute to the further development of the tunneling design and construction. The method described in this chapter for predicting strength gain and thermal differentials within a concrete mass using semi-adiabatic calorimetry is not currently described by an ASTM standard but should be created for this method of early strength development much as use of the maturity meter is described by ASTM C1074. The strength development and the structural and thermal properties of concrete at very early ages (8 to 24 h) would be useful information to publicize and develop. The question of what portion of the concrete mixture design should be performed by the contractor should be further debated and clarified by industry participants.

The issues that often come up in tunnel work, such as when is water curing necessary, what are allowable concrete drop heights, and where to sample concrete would be fertile research topics for graduate students. Published results of the research would lead to recommendations that can become industry policy.

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CHAPTER 3

Precast Concrete Segmental Linings

Jon Hurt and John Hart

Precast concrete segmental linings are commonly used in tunnels constructed by tunnel boring machines (TBMs). They are installed concurrently with excavation by the TBM, which erects the segments as the excavation is advanced. The linings provide the immediate initial ground support required in soft ground or broken rock and also serve as high-quality, watertight final support.

When a precast lining provides initial and final support, it is referred to as a one-pass lining system. Sometimes, additional support is provided after the segments are erected, in the form of a welded steel pipeline or internal concrete lining. These are referred to as two-pass lining systems.

This chapter describes the unique requirements for precast concrete segments, the required material characteristics, design and manufacturing requirements, and waterproofing and construction considerations. Special requirements for wastewater tunnels and temporary linings are also included.

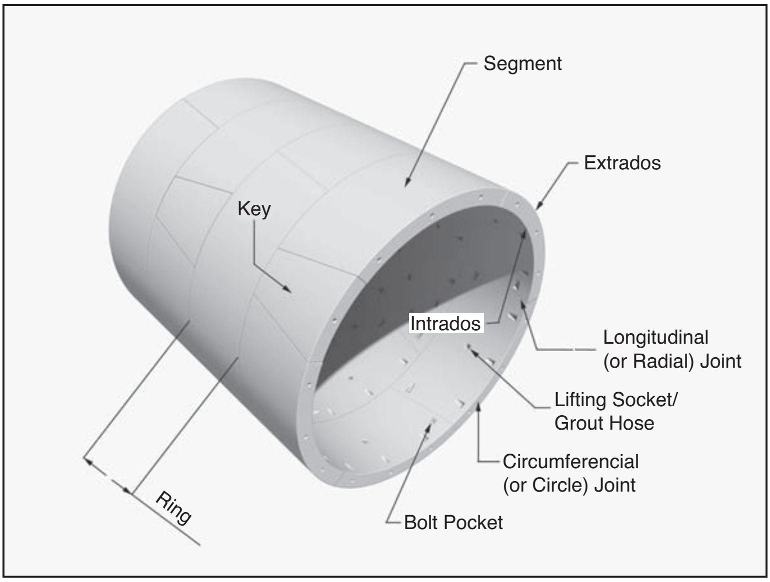
INTRODUCTION

Segments have been used in tunneling since 1869. Early segments were manufactured with cast iron, but in the 1960s precast concrete began to be used for segmental linings (Skelhorn and McNally 2009). Initially, these segments were sized so they could be lifted and bolted into place by hand, but over the years sophisticated segmental lining systems have been developed that can be mechanically installed by TBMs and provide an efficient, durable, and watertight tunnel lining.

Precast concrete segments are manufactured at a segment manufacturing plant, and the segments are erected by the TBM to form a completed ring during excavation. The various components of segmental linings are illustrated in Figure 3.1.

Precast concrete segmental linings provide several advantages. The manufacturing of the segments at a precast yard allows casting to be carried out in a controlled environment, with high quality control. The high degree of mechanization offered by modern TBMs allows rapid and accurate installation of the segments. And the need for an initial temporary support system is avoided, as are the challenges of providing a waterproof membrane and placing concrete in the tunnel environment.

Along with these advantages, however, the use of segments also presents challenges, which need to be addressed in the project specifications. The fabrication process needs to be able to repetitively deliver segments to high tolerances cast with high-quality, durable concrete. Segments need to be demolded, stored, transported, and erected without suffering damage. The segments and the various fittings need to be designed and specified to work as a complete system that allows the required tunnel alignment and lining tolerances to be achieved.



Courtesy of Jon Hurt.

Figure 3.1 Precast concrete segmental lining

Example specifications for precast concrete linings and for waterproofing gaskets are provided in Appendices B and C.

DIFFERENCES BETWEEN ABOVEGROUND AND UNDERGROUND APPLICATIONS OF PRECAST CONCRETE

The use of precast concrete is not unique to the underground environment. Precast concrete is used to form a variety of structures aboveground, sometimes similar to how precast concrete segments are formed into tunnel linings belowground. However, there are also a number of important differences in the use of precast concrete in the underground environment:

- The segments and inserts need to be designed and specified to work as a system to resist the applied loads, provide watertightness, and be durable over the life of the tunnel.
- The segments need to be cast with tight tolerances because of high construction and permanent loads at segment interfaces.
- In hydraulic applications the segments need to meet required roughness criteria.
- Segments are cast at high production rates with extensive reuse of molds.
- Segments are subject to a wide range of loading conditions during casting and transportation erection and from propulsion of the TBM—as well as the loads from earth pressure once in place in the tunnel.
- Segments are erected in the confined environment of the tunnel using specialized equipment.

MATERIAL CHARACTERISTICS

Each component of precast concrete is important to the overall performance of the structure. These components are considered individually in this section.

Concrete Mixture

Concrete for segmental linings is normally similar to that for standard cast-in-place (CIP) concrete, with some additional requirements to accommodate a high-quality precasting process. As such, many projects use CIP concrete specification (often prepared for other elements of an underground project) as the basis for segmental lining specification, simply cross-referencing the CIP specification as needed in the segment specification. However, a standalone segment specification has the advantage of being tailored specifically to the production of high-quality precast concrete and can be provided to the precast concrete manufacturer as a single document.

Concrete for precast segmental linings should be prepared and placed in accordance with ACI 318. For tunnels that carry liquids, such as water and wastewater, the requirements of ACI 350 should also be considered. Precast concrete generally has high strength (28-d compressive strength greater than 6,000 psi, or about 40 MPa), which is primarily required to allow early strength gain for handling in the precasting facility but also to withstand high TBM thrust forces. Low-permeability concrete is an important part of providing high durability. Hydraulic conductivity of concrete can be measured, and values less than 2.8×10^{-10} ft/d (1×10^{-13} cm/s) should be targeted.

Reinforcement Steel

Traditionally, precast concrete segments have been manufactured using reinforced concrete. Although steel fiber-reinforced segments are now a viable alternative, the use of reinforcement in segments is still common.

The reinforcement is used to provide sufficient strength to resist loads from manufacturing, handling, and installation, and the long-term loads from ground and groundwater. In addition to flexural and distribution reinforcement in each face, reinforcement at the joints is normally required to resist bursting loads.

Specifications for precast concrete segmental linings often incorporate requirements from the project-wide specification for steel reinforcement. A few changes may be required to make this more applicable for use in segments:

- Steel wire: Since reinforcement cages are normally mass produced, fabricators may use rolled steel wires rather than discrete straight bars. The steel wire may also be stronger than standard rebar, with a strength of 75,000 or 80,000 psi (about 515 to 550 MPa).
- Welding cages: Reinforcement cages are commonly prefabricated from a weldable grade of steel and welded together, either as a single cage or a series of elements (intrados and extrados mesh sheets as well as edge “ladders”).

The appropriate ASTM steel material specification (either ASTM A496/A496M, ASTM A497/A497M, ASTM A706/A706M, or ASTM A615/A615M) should be selected based on these factors.

Providing adequate cover to the steel is an important part of producing high-durability concrete segments. Although some design codes (e.g., ACI 318) allow a lower cover to reinforcing steel for precast concrete, a minimum cover of between 1 to 1.5 in. (25 to 38 mm) to the main reinforcement is typically specified to reduce the risk of corrosion to the rebar. This may be reduced as low as 0.75 in. (19 mm) in local areas of congested reinforcement. For tunnel linings designed to AC 350, the minimum cover is 1.5 in. (38 mm). International practices related to cover requirements vary, ranging from the British Tunnelling Society and the French Tunneling and Underground Space Association recommendations of 20 to 30 mm (0.8 to 1.2 in.) to recommendations of a minimum of 40 mm (1.6 in.) in some locations in Asia.

Cages must be placed accurately in the molds to ensure adequate cover on all sides. Cages are conventionally fabricated to a uniform width, without making adjustments for ring taper (i.e., where the width of the segment varies around the ring to permit the lining to follow a curved alignment). The minimum cover at the narrowest segment in the ring must be selected to control the minimum cover while recognizing that the cover at the widest part of the ring will be much greater, leading to the possibility that unreinforced corners could become susceptible to damage.

Precast concrete segments for TBM tunnel linings have traditionally been reinforced with steel bar reinforcement. The reinforcement is primarily required for handling loads during construction, because once installed in the ground, the tunnel lining is generally in compression, and the reinforcement is not required except to deal with bursting stresses at the joints. There are disadvantages to steel bar reinforcement, however. Any cracking, spalling, or edge damage that occurs during installation may lead to future problems with durability if the reinforcement is exposed to groundwater. The steel bar reinforcement is also unable to strengthen corners and edges where handling damage typically occurs.

Steel Fiber Reinforcement

Steel fiber-reinforced concrete (SFRC) has been used on many projects for TBM tunnel linings and can be used to create a lining that is more durable and less susceptible to handling damage than a steel bar-reinforced lining. It may also provide cost savings in production.

Steel fibers act in a slightly different way than steel-reinforcing bars by having the area of steel distributed throughout the concrete mix. This produces more ductile behavior, extending the composite material tensile strength beyond the point of crack development.

Segmental tunnel linings predominantly act in compression in their final condition in the ground and do not rely on flexural strength capacity. However, flexural tensile forces occur during the manufacturing, handling, and construction stages. These forces can be minimized using appropriate handling techniques to suit the strength of the segments. However, there will always be permanent tensile stresses that occur close to the radial joints and that require some form of reinforcement (bars or fibers) to be present in the permanent situation.

The North American building codes and the American Concrete Institute (ACI) code and guidelines give little guidance on the subject of tunnel lining concrete. Designing a code-compliant SFRC lining may require a plain concrete design approach for permanent loads or a variance based on the extensive range of literature and case histories available. Several guides are available that provide recommendation for designing structures with SFRC, including the German *DBV Guide to Good Practice* (GSCCT 2007) and RILEM TC 162-TDF (RILEM Publications 2002).

When designing steel fiber-reinforced segments, an interaction diagram is typically used to evaluate concurrent compressive and tensile loads that are computed by conventional methods (Figure 3.2). In addition, the design thickness is typically reduced by 1 in. (25 mm) to account for potential irregularities in fiber distribution at the inner and outer surfaces of the segments during casting and potential carbonation of near surface fibers. Also some designers choose to increase the number of segments per ring when using SFRC, which has the effect of decreasing ring stiffness and bending moments within the segments. Alternatively, some projects use a

Courtesy of Jon Hurt.

Figure 3.2 Completed SFRC segmental lining

combination of steel bar reinforcement together with steel fibers to provide the required combination of strength and durability.

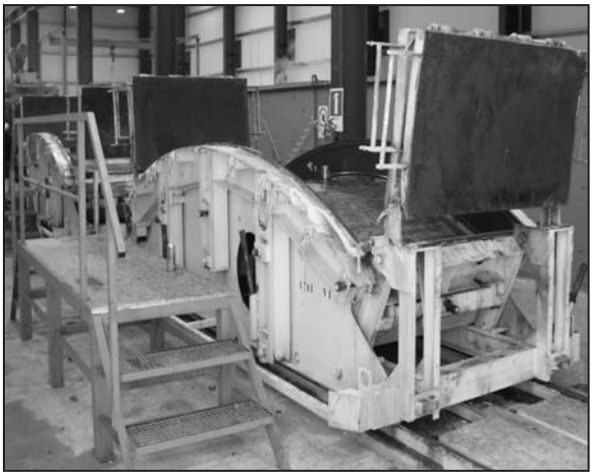
When designing with SFRC, it is important to specify a mixture that provides stable post-crack behavior. This means using fibers at least 2 in. (50 mm) long with a high aspect ratio (the ratio of fiber length to diameter). Steel fibers should meet the requirements of ASTM A820/A820M, and fiber-reinforced concrete should meet the requirements of ASTM C1116/C1116M.

From a corrosion propagation point of view, SFRC provides a material at least as durable as normal reinforced concrete. Carbonation does not pose a significant threat to SFRC, and although fibers at the immediate surface layer may corrode to a depth of surface carbonation, the fibers in the segment body do not corrode, largely because of the electrical path's discontinuity.

While flexural strength of normal reinforced concrete is relatively independent of tension cracking, the flexural strength of SFRC is closely related to tension cracking. Accordingly, potential cracking of SFRC is an important design aspect. The deterioration of SFRC depends on crack width, crack depth, type and diameter of steel fiber, and the severity of the environment. The limitation of cracks to that associated with conventional reinforced concrete (0.008 in. or 0.2 mm for water retaining structures) appears to limit chloride diffusion, with the cracks showing a tendency to completely heal with the corrosion products. Corrosion of the steel fibers at the surface of tunnels segments may lead to discoloration of individual fibers, but the concrete surface finish is not affected and does not lead to rust staining, as appears with corrosion of conventional reinforced concrete. More information on the durability of SFRC can be found in ACI 544.5R-10.

Synthetic (Polypropylene) Fibers

The addition of synthetic fibers, most commonly polypropylene fibers, to concrete provides segments with additional robustness in the event of a fire in the tunnel (Shuttleworth 2001). When exposed to high temperatures, segments without synthetic fibers undergo spalling, as the moisture in the concrete evaporates and creates vapor pressure that fractures the concrete. The explosive nature of the spalling adds to the hazards encountered by firefighters, and there is risk that spalling can reduce the segment thickness to such an extent that collapse or water inflow



Courtesy of Jon Hurt.

Figure 3.3 Typical segment mold

occurs. Synthetic fibers, which melt when the temperature rises and create paths for vapor to escape, have been shown to eliminate spalling in most fire scenarios.

Monofilament polypropylene fibers should conform to ASTM C1116/C1116M. A typical dosing rate is between 1.7 and 3.4 lb/yd^3 (1 and 2 kg/m^3). The exact dosage rate depends on the individual characteristics of the fibers and the fire loading for which the lining is designed. The addition of fibers to the concrete mix will decrease the concrete mixture's workability. Entrapping air during vibration may also impact the finish quality because of an increase in bugholes within the segment, and this should be considered as the mix design is developed.

Segment Molds

Molds should be of machined steel construction made to the tolerances required; typical molds are shown in Figure 3.3. They need to be designed to not warp or distort with repeated use and after being subjected to vibration during each placement. Molds must continue to provide segments that meet required tolerances. It is particularly important that molds allow joint surfaces to be formed to meet the design requirements for smoothness and plane in order to provide good bearing surfaces and that the edges of the mold form watertight joints to avoid poor-quality concrete on segment corners. Molds should be measured for conformity prior to commencing production and on a regular basis during production.

KEY DESIGN ISSUES

There are several key design issues that should be considered for precast linings. The most important is the number and type of load cases under consideration. These and other issues are described in this section.

Design Criteria

Prior to designing a segmental lining, a set of design criteria needs to be agreed upon between the owner/operator and the designer. This includes the required design life, geometry, any permanent facilities or fixings, watertightness, and fire resistance.

Segment Geometry

Segmental tunnel linings can be configured in a number of different ways to suit the ground conditions, tunnel size, and type of TBM. Segments are normally formed in either a rectangular, trapezoidal, or rhomboidal arrangement. The last segment to be inserted in a ring, known as the key, generally has tapered sides to simplify sliding it into place.

The width of a segmental ring is primarily dependent on the TBM's design, which defines the available space for handling the segment and the weight constraints of the available lifting equipment.

To allow a tunnel to follow the designed alignment, including curves, without having to insert packing materials in the circumferential joint to make the lining form a curve, tapered segments are used. This means that the two sides of a ring are not parallel, instead being slightly angled to each other. This results in a segment width that varies slightly around the ring. On curves, the widest part of the ring is positioned on the outside of the curve. On straight sections, adjacent rings can be placed so that the widest section alternates between opposite sides. During construction, the TBM navigation system is typically preprogrammed with the tunnel alignment and will determine the required orientation of each ring.

Design

The required segment thickness, concrete strength, and reinforcement requirements can be determined using one or more from a range of design methods, including closed-form solutions, bedded beam-spring models, and numerical analyses. In the permanent case, the applied loading is primarily from the ground and groundwater, although surcharges and internal loads should also be considered as appropriate. Careful consideration should be given to future changes in loading, which may be caused by other tunnels or deep excavations nearby. Loads resulting from the various lifting and erection operations during construction should also be considered in the design.

Although intended from building structures, concrete design codes such as ACI-318 are generally applicable to underground structures. One exception to this in some circumstances is the load factor applied to groundwater loads. The load factors used are based on ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures*, which specifies a load factor of 1.4 to be applied to fluid loads, which would require significant capacity for underground structures at significant depth below the water table. An alternative reduced load factor of 1.2 may be appropriate in some cases, because the commentary to ASCE/SEI 7-10 regarding fluid load explains that the load factor was selected because "emptying and filling causes fluctuating forces in the structure, the maximum load may be exceeded by overfilling; and densities of stored products in a specific tank may vary." Compared with this commentary, the groundwater loading on deep underground structures is much less variable because the change rate of fluid load is very slow, so no fluctuating forces are exerted on the structures, the maximum load cannot be exceeded by overfilling, the density of the lake water does not vary, and the number of load cycles is low compared to a storage tank.

A more appropriate design approach is contained in ACI 357, *Guide for the Design and Construction of Fixed Offshore Concrete Structures*, which states in Section 3.1 that dead loads include "external hydrostatic pressure" and in Section 4.4.1.1 provides a load factor of 1.2 for these dead loads. This is in line with the approach taken by international standards, which more appropriately address the situation of chiefly nonfluctuating and clearly defined fluids.

Detailing

Defining the geometry of the segment edges is an important part of segment design. Contact between edges of adjacent segments must be avoided, as otherwise the corners will break off after the load is applied. Recesses, or stress relief grooves, should be provided on all four side faces of a segment. At the extrados of the side faces, a deeper groove should be provided for the EPDM (ethylene propylene diene monomer) gasket. This groove should be dimensioned to match the exact requirements of the gasket manufacturer in order to provide optimum watertightness.

Hydraulics

Work done by the Construction Industry Research and Information Association provides guidance on the appropriate friction factors and roughness to use for hydraulic design in segmental concrete lined tunnels, taking into account the average step distance (misalignment) between adjacent segments along the longitudinal axis of the tunnel (Pitt and Ackers 1982). Guidance on whether bolt pockets should be filled is also provided.

Manufacturing

The manufacture of tunnel lining segments needs to take place in a covered, sheltered environment free from the harmful effects of sun, rain, snow, ice, and wind. Manufacture can be by carousel or by static process, such method being evaluated for cost effectiveness based on the quantity to be manufactured, the dimensions of the segments, and the reliability of equipment in guaranteeing a constant output to meet tunneling progress.

Different curing methods are available depending on whether the carousel or static process is adopted. If a carousel is used, then the facility generally includes a low-pressure steam curing area, where accelerated curing provides the productivity and output required. Segments are cured in controlled heat conditions with sufficient humidity to prevent moisture evaporation. This can be done by low-pressure steam, hot water, or electrical heating coils that provide humidity controls. If a static system is used, then moist curing, curing compounds, steam curing, or a combination of these systems can be used. For steam curing, static molds can be covered with a steam hood; for moist curing, hessian or similar fabric is placed over exposed surfaces and constantly kept wet.

Gaskets and packers can be installed at any time after demolding. Depending on the curing and protection required, the process needs to accommodate undercover storage for segment ring stacks after the demolding process. If the segments receive a curing compound on demolding, they can be taken to outside storage at the end of each shift. If the ring stacks are 100% humidity cured in chambers, they can be transferred to the outside storage area after the curing period.

Concrete for segments should be batched and mixed at the manufacturing facility and delivered by skip or truck to the molds to minimize the delay between batching and placing. When possible, the concrete should be mixed in batch sizes that match the segment volume to ensure consistency and conformity. Vibration can be either external or internal, but external vibration is more conventional. Whichever method is selected for vibration, it should not cause the molds to be damaged or distorted.

Calculations are necessary to determine the strength at which segments are demolded at a maturity age when they are capable of being safely handled and stacked without stress. Segments must be lifted and debonded from the mold and afterward rotated to be stacked with the intrados face up. In many specifications, this strength is given as a minimum of 2,000 psi (14 MPa) but the exact specification should be determined based on loads imparted by the type of lifting and handling equipment, the method of stacking segments in storage, and the location on the segment where these loads will be applied (e.g., vacuum or mechanical equipment will impart different loads on the segment). Induced loads from fork truck transportation and misplaced dunnage are often overlooked and can cause unnecessary damage.

Casting Tolerances

To minimize damage to the segments as they are erected in the tunnel (and particularly as the TBM uses the shove rams to push forward, applying large, concentrated forces to the segments), it is important that the segments are manufactured to tight tolerances. Tolerances that are normally specified, and the reasons for such specifications, include the following:

- Circumferential length: Building a ring out of segments not conforming to the required length will prevent the formation of a true circle, and the longitudinal joints between segments will be only in partial contact (birdsmouthing), resulting in increased contact stresses under load.
- Width (length along the tunnel): If segments within a ring are of different lengths, then the leading edge of the ring will contain a series of steps. The following ring will have incomplete support around the circumference, which may allow damage when the ram loads are applied.
- Smoothness/waviness of the extrados: In a pressurized-face TBM, watertightness is provided by the tail seal(s) closing the gap between the completed segmental lining and the inside of the TBM tailskin. Tail seals consist of wire brushes impregnated with grease. A rough segment extrados increases the wear on the brushes, and an uneven surface increases grease consumption.
- Local irregularities: A raised point on a contact surface will result in concentrated loads at that location, increasing the likelihood of local spalling or cracking.
- Gasket groove: If a gasket is to be used to provide watertightness, then forming a groove that matches the gasket manufacturer's requirements is important if the design water pressures are to be achieved.

Waterproofing Gaskets

Gaskets are used between segments to act as seals and provide an essentially dry tunnel. Typically, EPDM gaskets are used. These rely on the contact pressure between compressed gaskets on adjacent segments to resist water pressure. EPDM gaskets should be primarily specified on a performance basis, with the required design water pressure included in the specification, along with minimum material properties. The design pressure is usually double the working pressure (the anticipated groundwater pressure) to cover the long-term stress relaxation of the gasket material and provide the required factor of safety.

Specific testing of each gasket, within a groove matching the design of the segment, should be carried out by the manufacturer. There is a standard global approach to testing, and the model specification included in Appendix C describes this, along with other gasket requirements.

In general, EPDM gaskets will not suffer deterioration if the segments with the gaskets installed are stored outside and exposed to the weather, including sunlight. However, this should be confirmed by the selected manufacturer in each case.

EPDM gaskets are susceptible to hydrocarbon contamination. Some manufacturers produce gaskets that either have a protective coating or use a different gasket material such as chloroprene, and these can be specified if it is determined that contaminated ground or groundwater will come into contact with the tunnel lining. Hydrophilic gaskets that swell on contact with water can also be used.

In addition to specification of the gasket system, it is common to specify the overall performance of the segmental tunnel lining in terms of watertightness. This can either be in the form of a measured inflow rate or as a visual requirement (e.g., no visible water ingress above springline, damp patches only [no flowing] below axis). The allowable tunnel inflow rate will vary according to the ground conditions and tunnel use, with transport tunnels generally having high watertightness requirements. Examples of typical values specified for measured inflow rates are provided in Appendix B.

Coatings

In aggressive soil or groundwater conditions, a coating is sometimes applied to the outer surface of the segments to improve concrete durability. This approach is not uniformly adopted, and there are strong counterarguments against using the coating, including the fact that it will not last for the typical design life of the segments and there is a high likelihood of damage to the coating during installation in the tunnel. Alternative measures may be more effective, such as improving durability through the concrete mixture design (e.g., low water-to-cement ratio and use of silica fume, slag, or fly ash) or the casting process (adequate curing).

If coatings are used, then the extrados and side faces should be painted with a solvent-free emulsion epoxy coating.

Connections

Connections between segments can be made by either bolts or dowels. Bolted connections typically consist of straight steel bolts that connect into plastic sockets cast into the segments. Inserts in the mold are used to form bolt pockets on the internal surface of the segments and to make a bolt hole through to the joint face. Bolts can be used on both the circumferential and longitudinal joints. They are placed after the segment has been correctly positioned by the erector and is held in place by the TBM shove rams with the gaskets compressed. The bolts should be tightened to a calculated maximum load, which allows for some spare load capacity so the bolts do not fail when the ram load is removed and the gasket load is applied to the bolt. After the complete ring is in place and final tolerance checks have been made, bolts should be retightened as necessary.

Bolts can be used in combination with inserts to provide a good ring build (i.e., to place the lining with the very tight tolerances that allow optimum lining performance). On the circumferential joint, these inserts include shear cones or supplemental dowels. On the longitudinal joint, they include a cylindrical guidance rod. Each of these inserts fits into preformed holes in the segments.

Dowels are made from high-quality plastic and fit tightly into preformed holes in the sides of the segments. They provide shear and some tensile capacity, although not as much as bolted connections. By forcing good alignment between segments, they also serve to help maintain construction tolerances. Dowels can only be used on the circumferential joint. As the segments are installed in the tunnel to form each ring, they are positioned using the erector arm and guided into place using the inserts. On the circumferential joint, the joint is closed and the gasket compressed using the TBM thrust rams. Dowels, if installed, will maintain joint closure by gripping against the sides of the preformed holes.

Connections should be designed and specified as a system, considering the gasket characteristics, required tolerances, and joint configuration (Gruber 2010). Specifications should include material properties, load and deformation requirements, and dimensional tolerances. The range of applicable material properties and tolerances can be obtained from discussions with manufacturers, but will vary from project to project, depending on the size and depth of the tunnel.

In many situations, bolting is only required for the initial assembly of the ring, since the tunnel lining is always in compression. Removal and reuse of the bolts can be considered in these situations, which has been done on many projects. When considering bolt removal, the extent of future movement of the tunnel and its impact on gasket sealing and lining stability should be assessed. This movement could occur because of effects such as time-dependent loading, changes in loading due to surface excavations, or surcharges and seismic events.

Load Distribution Pads

Even with the high tolerances specified for the concrete finish of the joint faces, small variations in the surface can result in stress concentrations at "high spots" under applied loads such as the ram loading. To avoid damage under this scenario, load distribution pads can be used. These pads placed in the segment joints are more compressible than the concrete, allowing the load to be spread more evenly.

Different types of pads are used, including timber, bitumen, and synthetic pads. Timber pads are generally stiffest but have the disadvantage of being thicker than the other options. They can also be used as packing. (To keep the face of the ring in the same geometric plane, "packing" out of the circumferential joint in one or more locations is sometimes needed.) However, use of packing should be limited in gasketed linings, as it will increase the joint width and consequently reduce the compression on the gasket.

Segment Inserts

Typically, a grouting insert is provided in each segment. As with all inserts cast into the segment, the use of metal should be avoided, with appropriate plastics being used for manufacturing the inserts. This insert can either be the whole depth of the segment or penetrate part way into the segment with the hole drilled out later if secondary grouting is carried out. The insert should be equipped with a nonreturn valve or allow for one to be inserted. Leakage can often occur around the outside of grout sockets, and a hydrophilic gasket ring should be used on the outside, particularly if all grout sockets penetrate the full segment depth.

Inserts are also required to allow the segment to be lifted by the TBM. If a vacuum erector is used, the segment will need to incorporate sockets for shear pins, and calculations will need to be submitted demonstrating the system's capacity. If a mechanical connection is required, a threaded insert can be cast in, or a threaded grout insert can be used. To demonstrate the capacity of the insert, manufacturer test data will need to be supplied.

Segment Markings

Segment markings can be used to identify each segment in the ring, help position the segment in the ring, and—if necessary—identify the owner, project, and/or tunnel diameter. Permanent markings can be formed using inserts in the mold on the intrados and also on the circumferential joint, allowing segment types to be identified in a stack of segments.

In recent years, labeling each segment with a bar code has been shown to be beneficial. Bar codes allow a segment to be traced back to the concrete batch and can be integrated with a quality control inspection process for molds and segments. Segments can be coded into stacks, and the technology can be used to produce documentation for delivery. The system can also be used to show storage yard locations. Data are fed to the storage computer by wireless technology, and the system is accessed by handheld bar code readers with screens designed to prompt the reader to provide the data required. Other technology exists to embed transmitter chips into the concrete product, but this is generally more expensive than other methods and has significant technological teething issues at the time of writing.

Transportation and Storage

During the design process, calculations of the effects of the anticipated means of transportation and storage will be carried out (e.g., means of stacking and lifting). The specification requirements for transportation and storage should be developed to match the scenarios used in the calculations.

After being demolded and fitted with a gasket, segments are typically stacked and stored at the precast facility until the concrete reaches the design strength. They are then transported to the project site. During this time, segments can be damaged if handled with the wrong equipment or in the wrong way. Segments are typically lifted using slings or specially designed lifting devices, with the lifting points identified so as to minimize flexural loads on the segments (typically at quarter points). The use of chains or lifting devices that will damage the concrete should be avoided. For segments at an early age, made with SFRC, or with a high slenderness ratio, vacuum lifting is often used, which treats the segment uniformly and minimizes bending moments.

Segments are normally delivered to the TBM on rail-mounted cars, although in larger-diameter tunnels rubber-tired vehicles can be used. The TBM backup will have the facilities to lift the segments and move them forward to the erector.

Special Requirements for Wastewater Tunnels

As noted in the introduction, one of the functions of concrete linings is to protect the underground structure from aggressive environments. In wastewater tunnels, corrosion protection is a key function of concrete.

Freshly placed concrete has a pH of roughly 12 to 14 due to the presence of calcium hydroxide ($CaOH_2$), which is soluble and comprises approximately 20% of the concrete. Ordinary portland cement will start to corrode at a pH of 6, but with minimal damage. Lower pH values will cause more severe corrosion, given the presence of oxygen. In wastewater tunnels, the surface pH of the concrete will be lowered slowly over time by the presence of carbon dioxide (CO_2) and hydrogen sulfide (H_2S). These compounds form weak acids when dissolved in water, reducing the alkalinity of the concrete until bacteria can grow at a pH of approximately 9. Depending on the condition, this process of lowering the pH of the concrete's surface can take many years or just a few months. At a pH value of 9, *Thiobacillus* bacteria will start to colonize the concrete surface, converting hydrogen sulfide to sulfuric acid only in the presence of oxygen.

A variety of corrosion-inhibiting products are available for use in connection with concrete tunnel linings. These products can be grouped into two categories: (1) supplementary cementitious materials, which reduce the corrosive effect of wastewater on concrete; and (2) additional liners that isolate the wastewater from the concrete surface.

Supplementary Cementitious Materials. The most commonly used concrete additive for corrosion protection is silica fume, a dry powder of pure silica, specifically amorphous (noncrystalline) silicon dioxide, produced as a by-product of silicon and ferrosilicon metal production. Silica is typically nonreactive with everything but the strongest acids. Researchers have found that one property of silica fume is the ability to react with calcium hydroxide, a by-product of concrete. This reaction reduced the volume of weak calcium hydroxide and replaced it with calcium-silicate hydrate, which produces an extremely strong concrete. In addition, the reduction of calcium hydroxide in concrete also reduced the permeability of the concrete, making the matrix extra dense and resistant to liquid penetration—reducing and delaying the effects of attack by sulfides and chlorides but not eliminating them. Silica fume is not a proprietary product. There are many other proprietary products (too many to mention) that work on a similar principle to silica fume.

A different approach is to modify the basic ingredients of the concrete itself. An example of this is polymer concrete (Maguire and Iskander 2008), which has been used in pipe manufacturing since the mid-1960s. During the manufacture of polymer concrete, a polymer resin is used to bond sand, aggregates, and filler to create a dense, corrosion-resistant material, which is lighter than comparable strength concrete. This makes polymer concrete a good material for pipe jacking. The corrosion mechanism in wastewater environments, which acts detrimentally on portland cement concrete, does not corrode polymer concrete. The various manufacturers of polymer concrete use different bonding agents and resins based on proprietary technology. Polymer concrete is widely used on pipes, but only one segmentally lined tunnel, a sewer project in Offenbach, Germany, has used this material because of cost and constructability issues.

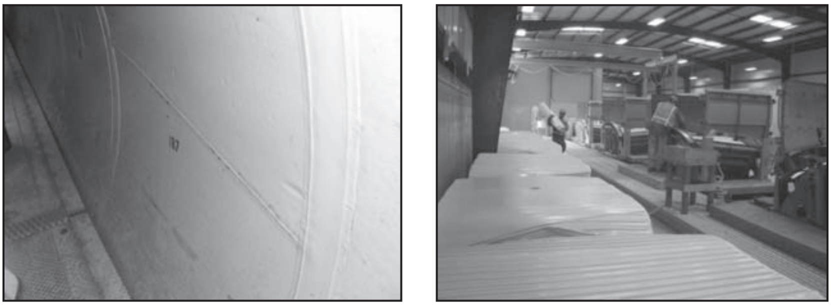
Liners. Liners work by isolating the wastewater from the concrete/segment surface. The most common liners are anchored polymeric sheet liner systems formed from either polyvinyl chloride (PVC) or high-density polyethylene (HDPE).

Anchored polymeric sheet liner systems or cast-in studliners have an anchoring system such as studs or ribs on one side and can be cast onto the face of precast concrete segments or CIP concrete. These materials have a good track record of corrosion resistance in wastewater environments, mostly in pipes, and both PVC and HDPE are highly resistant to sulfides.

The Deep Tunnel Sewage System in Singapore used a concrete segmental lining as the initial support. The tunnel was constructed by TBM, while the lining was bolted and gasketed to prevent water inflow. After tunneling was complete, an additional 9-in. (230-mm) thick concrete lining was cast in place with an HDPE studded lining placed 330 degrees around the tunnel to prevent H_2S corrosion of the concrete.

Cast-in studliners have been installed on precast concrete tunnel segments for the Upper Northwest Interceptor (UNWI) project in Sacramento, California (Figure 3.4). The casting process involves cutting and placing the PVC studliner to fit the concrete segmental molds and then adding the reinforcing cage. After this, the casting process proceeds as normal. After the studliner has been cast in the segment and the segments erected in the tunnel, joints that require welding are completed using an overlap on the segment liner or a plastic welding strip. The studliner is fusion welded. Finally, the studliner welds are tested to ensure that the welding is complete and watertight.

A similar process is used for an acid-resistant glass-reinforced plastic (GRP) face for precast concrete segments. The GRP face is placed in the segment mold during the casting process. Using



Courtesy of Jon Hurt.

Figure 3.4 Installed PVC segment lining on the UNWI project (left) and preparation of PVC liner for casting in segments (right)

TABLE 3.1 Concrete corrosion-resistant systems

	System	Type	Product Name Examples
One-pass Systems	Anchored thermoplastic lining	Lining	T-lock, AGRU Sure Grip
	Concrete faced with glass-reinforced plastic	Lining	Combisegments
	Precast polymer concrete	Lining	U.S. Composite Pipe, Future Pipe
	Acid-resistant concrete products, for use with precast segmental linings (or CIP linings)	Concrete additive	ConShield
	Microsilica (silica fume) concrete (segmental and CIP linings)	Concrete additive	Commonly used, not proprietary
Two-pass Systems	Liquid applied polymer-based protective lining	Coatings	Tnemec, Sauereisen
	Acid-resistant gunned cementations lining	Coating	SewperCoat
	Slip lining	Lining	Hobas, Krah
	Segmental sheet lining	Lining	Channeline
	Adhered PVC sheet linings (not recommended for underground use)	Lining	Linabond
	CIP linings (can add HDPE/PVC liners or additives)	Linings	Insituform, Inliner
	Spiral wound pipe	Pipe	Sekisui

a vacuum lifting device to prevent damage to the segments, these specialized segments are erected in the same manner as a normal segmental lined tunnel.

There are too many concrete corrosion-resistant systems to fully describe in this chapter. Table 3.1 lists a few of the available systems on the market. It is important to remember that a one-pass system of tunnel lining is generally much quicker and usually less costly to install than two-pass systems of corrosion lining.

Special Requirements for Segmental Linings for Temporary Use

In some cases, segments that do not form part of the permanent support (sometimes known as "junk" segments) are provided. A typical example would be in a water tunnel where the initial excavation is supported with segments, but the final liner is a welded steel pipe. In the case of junk segments, some relaxation in the specifications can be allowed, mostly related to long-term durability such as cover requirements and concrete quality. Segment tolerances can also be relaxed, and the use of gaskets can be omitted if ground and groundwater conditions around the tunnel are suitable. If gaskets are omitted from the segment, vertical segment molds (segment on its side) may be allowed, thus reducing segment surface finishing work.

KEY CONSTRUCTION ISSUES

Typically, segments are fed into the TBM in a fixed sequence, with each ring build starting with the segment opposite the key. When tapered segments are used, the ring will need to be built to a certain orientation, which is selected using the TBM guidance system. Prior to each segment being positioned, it should be inspected for any damage and to ensure that any required inserts or load distribution pads are in place.

To allow each segment to be placed, the TBM thrust rams are removed from the location where the new segment is to be positioned. The segment is then lifted into place using the erector arm, and its position checked to avoid steps and lips. The thrust rams are then extended onto the new segment to compress the gasket and hold the segment in place. The segment can then be connected to the adjacent ring using the fixing system adopted for the circumferential joint. It is important to minimize the number of thrust rams that are removed from the segments, both for the stability of the ring and to provide restraint to any pressure on the TBM face.

The same procedure is followed as each subsequent segment is placed. The erector arm is used to compress the gasket on the radial joint. To avoid the gaskets rubbing together and being damaged, a lubricant may be required on the gasket on this joint.

Grouting

Because of the overcut around the TBM shield, the thickness of the shield, and tolerances within the shield for segment erection, an annulus void of at least 4 in. (100 mm) is left behind the segments (unless an expanded lining is being used). To provide uniform support and loading on the segments, and to minimize ground movements and surface settlements, this void needs to be filled as soon as possible.

In soft ground tunnels, this grouting is typically carried out by pumping grout through small pipes in the TBM tailskin as the TBM advances forward—an operation known as tailskin grouting. To ensure that the annulus void is completely filled, the grout volume should be checked against both pressure and volume criteria. Grouting through ports in the segments is also possible, although this is not normally preferred as it delays the placement of grout, and each grout hole provides a penetration through the segments and consequently a potential leakage path.

In hard rock tunnels, a similar grouting process can be adopted, or an initial void filler of pea gravel can be installed, with grouting of the pea gravel carried out later.

Because the main function of the grout is to provide a relatively incompressible fill between the ground and segments, grouts are not normally required to have high strength. They can be weaker than the segment and, if applicable, the surrounding ground. A number of different types of grouts are available to deal with different scenarios (EFNARC 2005; Pelizza et al. 2010):

- Sand/fly ash grout: This is an inert mix, consisting of sand, water, and a filler material (typically fly ash). It is an economic backfill material, but ring movement and flotation of rings can occur if the grout remains fluid because of the low shear strength development. If there is any strength gain in the grout, it is limited and very slow. Use of an inert grout avoids any problems with blocking of grout pipes due to early setting of the grout but is not suitable in all situations (e.g., where water flows along the back of the lining toward the face of the tunnel, know as shunt flows).

- Cementious grout: This is a cement and water mix, often with additives such as bentonite and retarders to modify the set time and pumpability.
- Thixotropic (two-component) grout: This grout consists of a retarded, highly fluid component that can be kept in the tunnel for a considerable time before use and is easy to pump. The second component, an accelerator, is added at the injection point, causing the grout to gel almost immediately. This provides for fast acceleration of the grout, which helps to limit ovalization by locking the ring in place before it has time to float.

Specification of grouting materials should include requirements for initial test mixes, in order to demonstrate the grout's performance, and quality control checks to maintain uniformity of the constituent materials and the ongoing performance of the grout itself.

Dealing with flowing groundwater in open TBMs is challenging. Shunt flows have a tendency to wash out the annulus grout. Inflatable grout bags on the outside of the segments have been used to attempt to cut off groundwater flows, but if the rock is fractured, water can still flow toward the open TBM face. To ensure that the annular void is completely filled, secondary (or proof) grouting is often specified. In this case, holes are drilled through the segments at the grout hole locations to verify no voids are left around the tunnel lining. The holes can either be drilled in a regular pattern or based on targeted locations identified by a review of excavation volumes. If secondary grouting is specified, then the segment design needs to have considered the impacts of a localized high-pressure load, and the specification needs to include limits to avoid any critical pressure being exceeded.

Construction Tolerances

To avoid damage to the segments during erection and over the design life of the tunnel, tolerances for the constructed segments need to be specified. The following tolerances are normally specified:

- Plane of the leading face of the ring
- Maximum and minimum measured diameters
- Offset of actual tunnel center from design centerline
- Lips and steps between adjacent segments
- Gap between adjoining joint faces
- Roll of one ring relative to the adjacent ring

Examples of specified tolerances are provided in Appendix B.

In general, construction tolerances can all be measured manually using tapes and feeler gauges. However, this is often complex and challenging because of obstructions in the TBM. Automated systems are becoming available that provide a simpler means of measurement. When measuring segments, appropriate adjustments should be made for shrinkage and temperature.

Repairs

Casting imperfections or damage to segments during handling, transportation, or installation will require repairs. The specifications should contain criteria for repair, and a repair plan should be developed in advance of segment production to select appropriate repair materials and methods.

Problems that can occur during casting include honeycombing, where voids are formed on the concrete due to the failure of mortar flowing to fill the spaces between the coarse aggregate particles; and air bubbles, which coincide with the groove for the waterproofing gasket. During handling and transportation, damage can range from chipping of the corners to cracking through the entire section. Any segments damaged during handling or transportation should be repaired or replaced prior to being placed in the tunnel.

Damage should be assessed against the repair criteria. Defects that threaten the structural capacity of the segment or the ability to form a watertight joint should be rejected rather than repaired. Such damage would include cracks that extend through the complete cross section of the segment or pass through one or more of the joints. Since the performance of the EPDM gasket is highly dependent on the configuration of the gasket groove, any damage or defects that impact the groove generally require rejection of the segment unless it can be demonstrated that the repair will be able to accurately re-create the groove profile. Similarly, honeycombing on any joint face may compromise the watertightness by allowing a flow path around the gasket and would normally result in rejection of the segment.

Repair may also be necessary inside the tunnel. Segments may be damaged during installation behind the TBM. For example, edges may be damaged or the entire segment may be cracked as the TBM pushes forward. While a good ring build (within tolerances for plane and diameter) helps to minimize cracking damage, repair plans should be in place for damaged segments incorporated in the lining. Segments that incur damage that impacts the watertightness or structural capacity after erection should be repaired in the tunnel.

Repair methods include the following:

- Dry pack and epoxy-bonded dry pack mortar: Dry pack is a combination of portland cement and sand, with just enough water to hydrate the cement. Dry pack should be used for filling holes with a depth equal to or greater than the least surface dimension of the repair area. Holes should be sharp and square at the surface edges.
- Epoxy-bonded replacement concrete: Used for repairs between 1.5 and 6 in. (38 and 150 mm) thick, an epoxy bonding resin ensures a strong durable bond between the old and new concrete.
- Epoxy surface sealer: This is used for filling air voids/bugholes.
- Epoxy resin: This is used for filling cracks.

Repair costs for steel fiber-reinforced segments are typically lower than if only conventional rebar cages are used, for the following reasons:

- Fewer cracks typically arise during construction.
- Surface spalls may not need to be repaired since no rebar is exposed.
- Fibers projecting from spalls provide a superior bond with repairing mortar material.

QUALITY CONTROL

Quality control of the concrete is a critical part of the production routine of the precast factory.

Prior to acceptance and first use of the molds, they should be measured and checked against the required dimensions and tolerances. Typically, this can be carried out with conventional equipment, but for large or complicated segments a three-dimensional laser scan survey can be

Courtesy of Jon Hurt.

Figure 3.5 Trial rings to demonstrate good ring build

utilized. Once segment production starts, segment dimensions should be checked on a regular basis to ensure that the mold remains within tolerance.

In addition to verifying that tolerances are being achieved on individual segments, it is common to specify the assembly of trial rings in order to demonstrate that the segments fit together well and provide the required tunnel diameter (Figure 3.5). Typically, this will involve two or three rings of segments, built so that the rings are vertical, with adjacent rings staggered and using all the bolts, dowels, and alignment aids (but not gaskets). This allows the diameter of the assembled ring and any gaps between segments to be measured in multiple locations to demonstrate that required tolerances are being achieved prior to mass production of the segments. To verify continued quality, the top ring of the trial section could be replaced periodically during the casting period.

In addition to the regular concrete testing detailed in Chapter 2, maturity meters can also be used to monitor the development of concrete strength in accordance with ASTM C1074. Concrete permeability can be measured with a coulomb test in accordance with ASTM C1202.

Typically, segmental linings require limited monitoring, if any, after installation. If flotation of the segments in the fluid grout or very soft ground is a concern, then the level of the invert can be monitored. Since the majority of this movement is close behind the TBM, the first reading needs to be taken as soon as the TBM has advanced, and several readings need to be taken each day. If distortion of the lining is a concern, either due to the ground conditions (very shallow cover, for example), adjacent excavation, or a concentrated load, then convergence measurements of the tunnel can be taken. This can be done with survey prisms fixed around the lining, or hooks for use with a tape extensometer. Because this monitoring is difficult (or impossible) to do with the TBM trailing gear in the location, it should be carried out after the TBM has advanced away from the area.

CONCLUSION

Precast concrete segments have been used on many projects over the past 50 years to provide hundreds of miles of effective, durable, and safe tunnel linings. Segmental linings need to be considered as systems—a collection of components that can be assembled to provide a stable and watertight lining, compatible with the complex TBMs now being used for tunnel excavation.

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CHAPTER 4

Shotcrete

Robert Goodfellow, Pete Tatnall, and George Yoggy

In tunneling applications, shotcrete is a cementitious construction material that is sprayed from a nozzle either directly onto the excavated ground while the ground is still deforming soon after excavation as an initial lining or on top of previously placed shotcrete as a permanent lining. It exhibits early strength gain combined with high strength, low permeability, and high durability. Shotcrete has a wide range of possible applications. Dry-mix shotcrete has the ability to be placed far from any construction infrastructure, needing only water to be added to premixed ingredients supplied in bags. Placement of wet-mix shotcrete by a remote-controlled nozzle can give production volumes in excess of 15 yd^3/h (11.5 m^3/h).

INTRODUCTION

Shotcrete for ground control evolved in the early 1960s during the development of the New Austrian Tunneling Method (NATM) as a system for support in civil construction. Employed as a means of controlling ground behavior immediately after excavation, shotcrete distributes force in the excavated ground, providing stability as the excavation advances. Because of this primary purpose, one of the most important design and construction considerations when shotcrete is used is early strength development. As shotcrete technology has evolved, which it has done significantly in recent years, the focus has been on achieving this required early strength while also achieving as much long-term durability of the shotcrete as possible. Although two types of shotcrete involve delivery of dry and wet mixtures to the nozzle, most high-volume production shotcrete applications today use wet-mix shotcrete. Dry-mix shotcrete, however, still has many useful applications for locations and uses where delivery of a wet shotcrete mixture is logistically impractical—for example, small-volume applications for adits or deep exploratory excavations that are long distances from surface concrete delivery infrastructure.

For clarity, the two types of shotcrete are described:

1. Dry-mixture shotcrete (dry-mix): The dry-mix method involves placing the dry ingredients into a hopper and conveying them pneumatically through a hose to the nozzle. The nozzleman controls the addition of water at the nozzle. This requires a skilled nozzleman, especially in the case of thick or heavily reinforced sections. An advantage of the dry-mix process is that the water content can be adjusted instantaneously by the nozzleman, allowing more effective placement in overhead and vertical applications. The dry-mix process is useful in repair applications when it is necessary to stop frequently, as the dry material is easily discharged from the hose.

2. Wet-mixture shotcrete (wet-mix): Wet-mix shotcrete involves a concrete mixture containing all ingredients—including water—that has been thoroughly mixed prior to delivery to the nozzle. Compressed air is introduced at the nozzle to accelerate the mixture through the nozzle and onto the receiving surface. Wet-mix shotcrete generally produces less rebound and waste (when material falls to the floor) and also less dust compared to the dry-mix procedure. The greatest advantage of the wet-mix process is that larger volumes can be placed in less time.

In the early days of shotcrete application, dry shotcrete was used exclusively. Aluminate accelerators worked best for fast set and developing early strength but created so much heat by speeding the hydration process that the hardened concrete was unable to resist the action of carbonate exposure in the long term. After wet shotcrete was developed, it initially used accelerators in the aluminate and silicate "hot" chemical family to speed up the initial set and hydration of the concrete mixture. These chemicals were negative to durability, primarily due to heat generated preventing a completely stable hydration process. Although the shotcrete in most applications developed into structurally strong concrete, misuse and overuse of early admixture chemicals left residual, unhydrated, or "burned" cement that could not resist the elements of its environment so that the linings were not durable.

Today, chemicals are available that are low in alkalinity, more friendly to user and concrete, and—when introduced in a controlled manner—able to provide early strength as well as long-term durability. The admixtures used should be tested for compatibility with the base ingredients to make sure the use of these chemicals does not negatively impact the shotcrete quality.

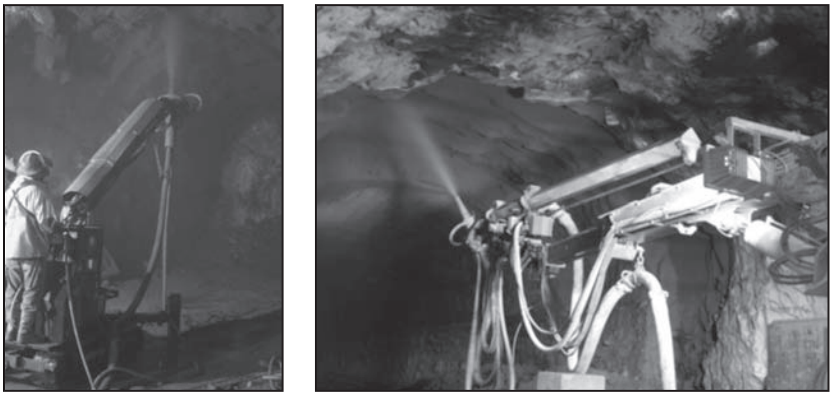
Shotcrete is now used with excellent results as a final lining for tunnels. It remains essential, however, to understand the significance and use of the accelerating admixtures and dosing requirements for a given mixture design and application requirement. Preconstruction testing and quality control before, during, and after application are critical. As noted, shotcrete technology (including both materials and equipment) has evolved significantly in recent years, resulting in the update or withdrawal of many existing standards and test methods. Our industry must recognize the new standards and technology available and use these in contract documents.

As a complete reference to the use of shotcrete underground, the new edition of the *Guide for Specifying Underground Shotcrete* (ACI 506.5R-09) is strongly recommended. It describes the manner in which shotcrete materials and placement should be specified, monitored, and reported in the United States.

DIFFERENCES BETWEEN ABOVEGROUND AND UNDERGROUND APPLICATIONS OF SHOTCRETE

There are notable differences between the application of shotcrete underground and aboveground. The following list outlines requirements underground that are not necessary in aboveground applications:

- Where shotcrete is used to provide rapid support, it needs to gain strength very quickly (i.e., early strength development may be fundamental to the application).
- It is particularly desirable to minimize shotcrete thickness in underground applications, because increased excavation dimensions have direct bearing on construction costs and potentially on short-term stability conditions as well.



Courtesy of George Yoggy.

Figure 4.1 Application of wet-mix shotcrete in tunneling by robotic spraying equipment

- Shotcrete may be placed directly against ground or against a continuous geotextile layer. Groundwater or seepage may unavoidably have direct contact with underground shotcrete during placement and possibly continuously during the shotcrete structure's life.
- Getting the shotcrete to the face of the excavation is complex.
- Shotcrete involves greater safety concerns than other types of concrete, as installation of ground support by definition requires entry into areas of unsupported ground to secure the stability of the tunnel opening.
- Underground applications of shotcrete typically involve significantly more overhead application than is required aboveground. With this comes higher rebound and more cleanup.
- The construction of a ring of support means that the chance of incorporating rebound in the invert is increased. All rebound must be cleared from the invert prior to installation of shotcrete lining.
- Underground work often happens 24 h a day, 7 d a week, and a reliable source for shotcrete is necessary at all times.
- Although shadowing (entrapping air into the final concrete section by poor spraying technique) can be a problem both above- and belowground, belowground structures may be supporting greater load and therefore often require more reinforcement. As such, it is important to design the excavation shape and to design and detail rebar so the possibility of shadowing is minimized.
- Robotic spraying equipment is more commonly used belowground (see Figure 4.1).
- Underground work is generally staged, which is unusual aboveground.
- Ground and ambient temperatures tend to be more consistent underground than near the surface.

MATERIAL CHARACTERISTICS

Shotcrete mixes differ from other concrete mixes in many ways. A fundamental cause of this difference is the interaction between the shotcrete and the ground onto which it is sprayed. Shotcrete is applied while the ground is still deforming but typically reaches its initial strength before the ground has stopped deforming. The shotcrete and the ground actively interact as the shotcrete is applied and hardens. Applications of concrete, such as cast-in-place (CIP) and precast concrete segments, are said to be passive because they do not involve this interaction. Another cause of differences in shotcrete mixes is the application process itself. During application, shotcrete is transitioned through a variety of openings of decreasing size, separated into individual particles before exiting the nozzle, and then accelerated dramatically from transport speed (3 fps or 1 m/s) to impact velocity (300 fps or 100 m/s). As such, the mixture differs from a CIP concrete mixture in many ways.

In shotcrete mixes, the largest particle size—that of the coarse aggregate—is reduced. The maximum size is ⅜ in. (9 to 10 mm), and the proportion is limited to approximately 20% of the total batch by mass or 17% by volume. These reductions result in the amounts of fine aggregate and cement in the mixture being increased, which in turn increases the mixing time and the shear energy required to distribute and blend the mixture.

There are other differences between shotcrete and CIP concrete as well. Fiber reinforcement is used with much greater frequency in shotcrete because it reduces and may eliminate the need for prior placement of rebar and therefore reduces concerns regarding rebar encapsulation and shadowing. Air entrainment is affected by the spray placement method and therefore must be considered differently when shotcrete is used rather than CIP. Shotcrete also requires more extensive preconstruction trials than CIP because site-, contractor-, and operator-specific factors have a greater influence on shotcrete performance than on CIP performance. More preconstruction trials often means longer lead time before placement can commence, as it is important to get these things right well in advance of starting the work.

Lastly, the surface roughness of shotcrete is greater (although finishing is possible) than for CIP. A good sprayed surface finish should be an orange-peel-like texture without wormholes or pockmarks. This can impact hydraulic considerations.

Considering all of these mixture characteristics, it is easy to see why insufficient and improper mixing is a common cause for problems with both the efficiency of shotcrete placement and its performance once it is placed. The following are mixture design guidelines that can help reduce problems with shotcrete mixtures underground:

- Cement content must be higher than the regular 350 to 400 lb/yd^3 (207 to 237 kg/m^3) in order to properly coat the increased surface area of the smaller aggregate. Typically, values in excess of 750 to 800 lb/yd^3 (445 to 475 kg/m^3) are necessary, and it is recommended that a minimum value be specified during design to make sure that the bid cost reflects a reasonable mixture design.
- Generally a ⅜ in. (9 to 10 mm) maximum aggregate size is used.
- The water-to-cementitious-materials (W/C) ratio is generally in the 0.4–0.45 range, promoting early strength gain. This also reflects the high cement content required to provide sufficient paste to cover the increased surface area of the smaller aggregate. Pumpability and strength gain concerns are usually addressed by use of water-reducing admixtures.
- Cement replacement with fly ash is not commonly used because of the requirements for early set times. Anticipated problems caused by high heat of hydration can usually be dealt with by admixtures.
- Cement replacement with condensed silica fume is more common and has been found to assist with shotcrete quality by helping to reduce rebound; improve adhesion and

cohesion, allowing the ability to build thicker layers on vertical and overhead surfaces; and reduce "washout" and increase density (McKelvey 1994).

- Admixtures are commonly used to promote early strength gain, maintain workability, and prevent premature hydration. Details about types of admixtures used in shotcrete are found in Appendix D.
- Air entrainment, if specified, is typically maintained around 8% during mixing but is reduced by impact velocity and compaction during placement—usually to around 3% to 4%.

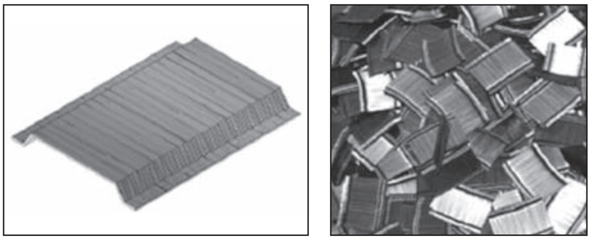
KEY DESIGN ISSUES

Shotcrete analysis and design cannot be considered separately from design issues related to tunnel construction because of the interaction between the ground and the shotcrete structure. Successful shotcrete design depends on a thorough understanding of how the ground and the shotcrete structure will behave together. Before the key design issues related to underground applications of concrete are discussed, readers are reminded of some basic guidelines related to shotcrete use:

- American Concrete Institute (ACI) documents are the primary resource for North American engineers. These documents undergo continual review and revision by committees of experts in the field, including designers, consultants, contractors, and product vendors.
- Standard test methods, specifications, and practices related to shotcrete are covered in ASTM documents, which are updated periodically to reflect new developments. The engineer must decide if an ASTM document is applicable based on project-specific considerations.
- Shotcrete use is widespread, and foreign references may also be of value to North American engineers. From time to time, design and construction approaches used in Europe and elsewhere are employed in North America. A notable example is in construction of shotcrete-lined tunnels in soft ground or weak rock.
- Shotcrete is not a design philosophy in the same way as, say, NATM and the Q-system, but in both of these examples, shotcrete has become an indispensible material for support and control of ground deformation during excavation and support of the opening.
- Shotcrete is used for a wide variety of ground support applications, and the purpose determines the design approach. Design approaches are based on one or more of three methods depending on the anticipated ground behavior: (1) a closed-form mathematical model of a theoretical ground condition; (2) a set of empirically developed equations based on observed ground behavior; or (3) numerical methods of modeling.

Fiber-Reinforced Shotcrete Design

Steel fiber-reinforced shotcrete lining design should follow the principles of ACI 506.1R-08 and ACI 544.4R-88 (reapproved in 2009) for both design and testing. The ACI acknowledges that while fibers have limited impact on ultimate direct tensile strength or compressive strengths, adding a critical mass of steel fibers similar to those shown in Figure 4.2 (greater than 0.4% by volume or alternatively greater than 50 lb/yd^3, or 30 kg/m^3) improves resistance to thermal and shrinkage cracking and the postcracking flexural strength of a concrete mixture.



Courtesy of Bekaert.

Figure 4.2 Steel fiber sets suitable for shotcrete mixtures break apart upon mixing and are evenly distributed within the mixture.

As part of the ultimate load capacity design, the structure should comply with established criteria for ductility and deflection. Additional design guidance can be obtained from Vandewalle (2005), which is referenced directly from the ACI guidelines. The use of macrosynthetic fibers should be allowed after testing proves the equivalence of these fibers with the performance of steel fibers. The development of macrosynthetic fiber material technology is an area of near-constant change, and reference to the latest project experience and codes is recommended.

Lattice Girders

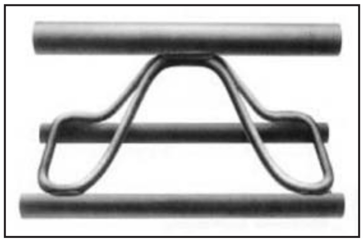
Lattice girders are frequently used for profile control in tunnel excavation and are necessarily substantial steel structures (Figure 4.3). These girders are often not included in design calculations because, when acting as compression rings, the lattice girders do not increase the stiffness significantly. When acting in flexure, though, it is common practice in geotechnical design to "capture" ground loading onto a stiff beam that then arches across a length of a less stiff structure. This design approach is common in open-cut excavation when designing soldier pile and lagging excavation support. It is also common in steel rib and timber lagging tunnel lining design. A similar design approach is considered applicable to the stiff lattice girder within a more lightly reinforced shotcrete shell.

While ACI 318-08 mandates a maximum allowable spacing for reinforcement bars for design, it is recommended that consideration of lattice girders be allowed in underground applications where desirable. Discussion on the effect of relative stiffness of tunnel linings with relation to ground conditions can be obtained from Goodfellow and Groves (2000).

Blasting Adjacent to Shotcrete Linings

The use of shotcrete in close proximity to the excavation heading is a common application, and many of those excavations are progressed using blasting. There is therefore a significant body of research on the resistance of freshly applied shotcrete to vibrations due to blasting. The question of damage to shotcrete from blast vibrations is frequently raised, and while it is intuitive from global experience that this damage is not significant (otherwise large areas of shotcrete would be rendered useless), hard evidence is often not immediately available. The following information seeks to briefly address the issue of blasting damage to shotcrete, and if additional information is required, the reader is referred to the source documents.

Research from Sweden is quoted by the International Tunnelling and Underground Space Association (ITA): "After 24 hours, shotcrete could resist vibrations up to 500 mm/s [20 in./s].



Courtesy of DYWIDAG-Systems International.

Figure 4.3 Three-bar lattice girder used in tunnel excavation and support

Tests done in the Southern Link tunnel (Malmo) [measured] vibrations of less than 80 mm/s [3 in./s], as close as 5 m [16 ft] from full blasting rounds at the tunnel face" (ITA 2010).

Shotcrete as a Permanent Lining

Considering how developments in shotcrete mixes have been furthered in recent years, and considering the demonstrated strength, density, and durability of shotcrete over many years, there is now no credible reason why shotcrete cannot act as a permanent lining in underground structures. This has been stated by the ITA in several of its publications (ITA 2010).

Knut Garshol, the animateur of ITA Working Group 12: Sprayed Concrete Use, wrote the following in his 2004 foreword that was presented again in the 2010 state-of-the-art report: "The acceptance of shotcrete for permanent linings is still today facing obstacles caused by the previous standard approach of using shotcrete exclusively for temporary support." This usage had the effect of ignoring long-term material behavior in design and placement of shotcrete. The results of this were (predictably) some durability issues with the material for many reasons. This behavior has led some to believe that shotcrete has inherently poor long-term material properties, but this belief is out of step with recent experience with shotcrete.

Garshol goes on to say, "On the contrary, the technology is available and today high quality and durable shotcrete is the norm and just a matter of decision-making and specification." The emphasis on specification is important and provides the basis for separating modern-day durable shotcrete mixtures from the high-baked mixtures of the past.

KEY CONSTRUCTION ISSUES

Construction issues of logistics dominate when using shotcrete effectively. The process of maintaining a coherent mixture and spraying semi-continuously over many hours is a formidable challenge. The following section summarizes the issues and provides discussion of solutions for the issues raised.

Mixing and Placement

In addition to achieving the requirements set forth in the design specifications, shotcrete must have the ability to be pumped from the location of delivery to the location of placement, often a distance of several thousand feet. Pumping technology has been addressed in previous chapters and need not be repeated here. However, adequate mixing is as important as pumping technology to ensure proper application from the nozzle to the wall. It was noted earlier in this chapter that

Courtesy of George Yoggy.

Figure 4.4 Concrete drum mixer specifically designed for underground application

more shear energy is required to mix shotcrete because of the smaller aggregate size and higher cement concrete in shotcrete mixes. Of the following three types of mixers, high-shear pan-style mixers give the best results when mixing shotcrete:

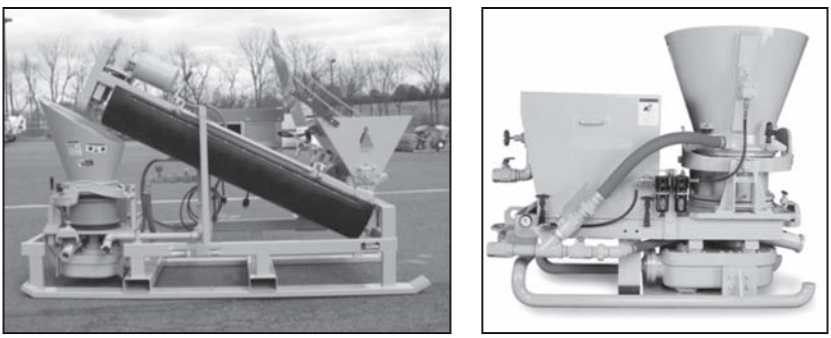
1. Standard ready-mix drum-type mixers (Figure 4.4): These mixers add 25% to 50% more revolutions to ensure adequate mixing. During mixing, the drum should be reversed to the point of discharge and then reversed to the mixing cycle, using slower and variable speeds for several revolutions before reaching normal mixing revolutions per minute. This cycle should be repeated three or more times to increase the mixing action of the materials.
2. Screw conveyors (augers): Screw conveyors are primarily transport tools that provide some mixing capacity. To improve and increase mixing, the auger should be established and maintained at an incline of 28 to 32 degrees to accommodate longer residence time for mating of materials and liquids and to allow “fall back” of material over the flights of the auger. All of these actions will improve the homogeneity of the mixture. Slower is better when employing this type of mixer.
3. Pan mixers: Horizontal, high-speed pan-type mixers are well suited for small aggregate mixes and will produce high-quality, homogeneous mixes with high fines designs (an example is shown in Figure 4.5). The strong, shearing action can fold and thoroughly mix materials in comparatively shorter times than can other types of mixers.

Pumps and Nozzle Spraying Equipment

Proper application of shotcrete requires the employment of equipment suited for the chosen process, logistics of supply and placement, volume to be placed in a given time, and specified performance. Conveying and spraying equipment and accessories for the dry shotcrete method differ from the wet method, and both the engineer and owner should familiarize themselves with equipment types and performance differences when planning a project. Equipment manufacturers that specialize in building and vending shotcreting machinery can be helpful in providing information regarding equipment operation and available choices.

Courtesy of Liebherr Concrete Technology.

Figure 4.5 Ring-pan concrete mixers



Courtesy of George Yoggy.

Figure 4.6 Typical dry-mixture shotcrete pumps, one shown with a predampener for the dry-mix materials (left)

Dry Shotcreting Equipment

Dry shotcreting equipment for tunnel construction generally consists of a rotor- or bowl-type continuous feed machine that receives material from a supply source and meters the mixture in a controlled manner through pipeline and hoses to the nozzle (Figure 4.6). The energy for transport and spraying is compressed air, and volume requirements must be matched to the feed and production rates of material for an application. A typical dry shotcreting setup requires 600 to 750 cfm (0.35 to 0.38 m^3/s) of air at 90 psig (720 kPa). Long conveying distances will require more air.

Spray nozzle configurations for the dry-mix method are similar from various sources and are designed to allow the introduction of water to blend with the materials as the mixture passes through. A mixing chamber allows turbulence to mix and produce a plastic consistency for discharge and deposition and compaction onto the receiving substrate.

Supply of mixtures to dry-mix shotcrete machines can be provided from a ready-mix truck, volumetric batching equipment, or mixers where the mixture includes sand moisture of 3% to 6%. Preblended, packaged mixtures designed for a specific performance are popular and efficient and provide a high degree of quality control. However, they require moisture to be added prior to discharging into the dry shotcrete machine.

Predampeners are machines that consist of mixing and metering augers and are fitted with spray bars that can introduce the required moisture to facilitate a prehydration of the cementitious component before the mixture passes through the system. This activity is essential to produce quality concrete, reduce rebound, and control dust.

Dry shotcrete systems suitable for tunnel shotcreting applications are rated at 3 to 8 yd^3/h (4 to 6 m^3/h) of production.

Wet Shotcreting Equipment

The common machine for conveying wet shotcrete mixtures is the twin piston, hydraulically powered, "swing" tube concrete pump. While all concrete pumps are similar in function, not all are suitable for shotcreting applications. Volume throughput for shotcreting operations generally ranges from 8 to 25 yd^3/h (6 to 19 m^3/h), which could indicate that a small machine is suitable. However, because the concrete employed must be conveyed through a number of line and hose reductions, the energy required for efficient transport demands a machine of adequate horsepower, hydraulic pressure, and piston capacity to suit the application. A machine rated for 50 yd^3/h (38 m^3/h) can be considered an average shotcrete pump, and capacity may be increased as conveyance distances increase.

A concrete pump well suited for shotcreting should have material cylinders that are close together and a valve system (swing tube or "S" tube configuration) that is capable of switching rapidly to reduce line pulsation, particularly for hand spray application.

Nozzle for Wet Shotcrete

The nozzle used for wet shotcrete application is critical to the quality of material placed (Figure 4.7). Concrete for the wet application is extruded through a hose under high pressure (i.e., more than 1,000 psi, or 6,900 kPa) at a rate of 3 to 4 fps (0.9 to 1.2 m/s). Upon entrance into the nozzle chamber, a high volume of air is introduced to intercept and "shred" the extruded mass back to individual particles as were originally mixed in the batching process. These individual components are then accelerated through and out of the nozzle assembly at nearly 100 times greater speed than the entrance speed—that is, over 300 fps (100 m/s). When accelerating chemistry is employed, the nozzle system must also be able to accommodate the introduction of the chemical in a controlled manner.

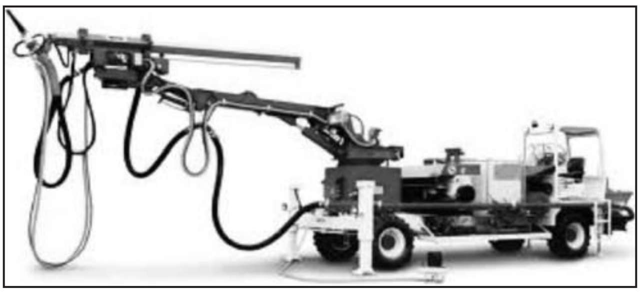
The nozzle system for proper application of concrete for ground support must be designed and manufactured to accommodate this physical process to ensure proper placement, compaction, and durability.

Robotics

Mechanical nozzle-manipulating equipment has evolved to meet a variety of demands for underground projects. Development of improved concrete mixtures, efficient pumping equipment for wet shotcrete applications, and larger excavated openings required taking the nozzle out of the operator's hands. Attaching a shotcrete nozzle to a mechanical arm or boom can overcome the fatigue factor of handheld systems, allow higher volume throughput, and accommodate improved

Courtesy of Allentown Shotcrete Technology, Inc.

Figure 4.7 Typical nozzle for wet shotcrete



Courtesy of BASF.

Figure 4.8 Wet-mixture shotcrete equipment with robotic nozzle

safety by supporting ground prior to personnel reentry (as shown in Figure 4.8). The term "robot" should be considered with caution since the proper operation and application still depend on the skill of the operator, which requires training and education. Since the increased throughput requires more air volume than can be manhandled, attaching the hose and nozzle to a mechanical device allows proper spraying and increased productivity. A manlift that provides a means of supporting the load of the hose and back pressure of the nozzle can also be a useful means of mechanizing and improving shotcrete efficiency and safety.

Nozzle-manipulating equipment can be configured to connect to a concrete pump in a relatively small footprint or can be a fully equipped carrying vehicle that incorporates an onboard air and accelerator supply as well as the concrete pumping system. Spray mobiles can be equipped to spray in large openings with a reach as much as 30 ft (9 m) or more. Choosing automated equipment for shotcreting applications should be done with counsel from a leading manufacturer of this type of equipment.

Testing

The objectives of shotcrete testing are similar to those of testing all other types of concrete, as described in Chapter 2. However, testing shotcrete has some unique aspects. In shotcrete application, preconstruction testing focuses on three major factors: (1) shotcrete mixture properties, (2) the mixture's ability to achieve design requirements under realistic field conditions, and (3) the ability of nozzlemen to place the mixture properly under realistic field conditions. It is important that the preconstruction trials be performed on-site with the same personnel, mixture design, and equipment that will be used during construction. Test panels are conventionally sprayed under representative conditions that mimic those expected during construction. That is, the nozzleman sprays panels fixed on a vertical wall and panels fixed overhead for application to tunnel support.

During construction, test cylinders, cubes, or beams should be obtained from shot material (either from in-situ cores or shot panels) according to the provisions of ASTM C1140. This will ensure the required consistency of sampling and that the designed shotcrete mixture meets specified requirements. Requirements for strength must be met, as must qualitative assessments of density. Assessing density includes looking for honeycombing, voids (especially around reinforcement or lattice girders), lamination, or sagging. Assessments of expected rebound will be made by the contractor to try to minimize wastage and better control costs. Table 4.1 provides descriptions with notes and references for the most common tests undertaken.

TABLE 4.1 Field testing methods for shotcrete application

Method	Description	References
ASTM C1604 Standard Test Method for Obtaining and Testing Drilled Cores of Shotcrete	ASTM 1604 provides procedures for obtaining cored specimens to determine the compressive strength of shotcrete during preconstruction and construction, and from older shotcrete structures. Shotcrete strength is affected by the shotcrete's location in the structure. Vertical, subhorizontal, and overhead elements of the shotcrete structure may show variability. Core strength is affected by core orientation relative to the direction of the shotcrete application. These factors should be considered in planning the locations for obtaining shotcrete samples and in interpreting strength test results. Core samples containing wire mesh or reinforcing bars should not be used for compressive strength testing. Sample acquisition may require a combination of core drilling, sawing, and grinding, which may have the potential to adversely affect the sample condition if care is not taken during sampling. Shotcrete samples obtained from test panels should follow the provisions of practice in ASTM C1140. The diameter of the cores should be at least 3 in. (75 mm), with the length being twice the diameter. Smaller core diameters may be used if permitted by the test specifier. When obtaining samples using the methods described in C1140, be sure the test panel is large enough for the number of cores or beams required for testing. The depth of the test panel should be 6 in. (150 mm). After the cores have been drilled, wipe off the surface water on the drilled cores and, within 1 h of drilling, seal the cores in separate plastic bags to prevent moisture loss.	ASTM C1604/C1604M-05. 2005. Standard Test Method for Obtaining and Testing Drilled Cores of Shotcrete. *Annual Book of ASTM Standards*. Vol. 04.02. ASTM C1140-03A. 2003. Standard Practice for Preparing and Testing Specimens from Shotcrete Test Panels. *Annual Book of ASTM Standards*. Vol. 04.02.
ASTM C78 Test Method for Flexural Strength of Concrete	Because most shotcrete will contain an accelerator, the shotcrete will not be cast in a mold. Beam samples from accelerated shotcrete should follow the procedures outlined in ASTM C1140. For unreinforced shotcrete C78 is appropriate for testing flexural strength. With fiber-reinforced shotcrete, the specifier may require both flexural strength and residual strength values for the shotcrete. If this is the case, ASTM C1609 should be used for testing.	ASTM C78/C78M-10. 2010. Standard Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading). *Annual Book of ASTM Standards*. Vol. 04.02. ASTM C1140-03A. 2003. ASTM C1609/C1609M-10. 2010. (see next page)

(Table continued next page)

TABLE 4.1 Field testing methods for shotcrete application (continued)

Method	Description	References
ASTM C1609/ C1609M Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete	Molded or sawn beams having a square cross section are tested in flexure using a third point loading configuration similar to ASTM C78 with the following exceptions: closed loop servo controlled testing system and roller supports that are free to rotate in their axes. The first peak strength characterizes the flexural behavior of the fiber-reinforced shotcrete up to the onset of cracking while residual strengths at specified deflections characterize the residual capacity after cracking. Beam samples from accelerated shotcrete are used in conjunction with ASTM C1609/ C1609M. The beams should be obtained following the procedures in ASTM C1140. Generally, 4 × 4 × 14 in. (100 × 100 × 355 mm) sawn beams are used in this test for shotcrete. The test panel should be 6 in. (150 mm) deep and large enough to saw from the test panel the number of beams required for testing. In many instances, several test panels of shotcrete samples may have to be produced. Since the test panels are normally made in the field, care should be taken to prevent evaporation of water from the panels. Panels should be covered and tightly wrapped (as soon after fabrication as possible) with a sheet of material meeting the requirements of ASTM C171. When sawing the beams in the field, the beams should be sawn larger than the 4 × 4 in. (100 × 100 mm) required for testing, tightly wrapped to prevent evaporation, and sent to the lab, which will saw the final samples. It is important that the beams be free from corrugations caused by uneven sawing. The beams should remain in a moist curing environment up to testing. Not all commercial laboratories are set up to run ASTM C1609. An approved cement and concrete reference laboratory for testing ASTM C1609 should be used when this test is required by the specifier.	ASTM C1609/ C1609M-10. 2010. Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading). *Annual Book of ASTM Standards*. Vol. 04.02. ASTM C78. 2010. ASTM C1140-03A. 2003. ASTM C171-07. 2007. Standard Specification for Sheet Materials for Curing Concrete. *Annual Book of ASTM Standards*. 04.02.
ASTM C1550 Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete	Shot round panels (3 in. [75 mm] thick × 32 in. [800 mm] diameter) of fiber-reinforced shotcrete are subjected to a central point load while supported on three symmetrically arranged pivots. Load and deflection are recorded simultaneously up to a specified central deflection. The energy absorbed by the panel is representative of the flexural toughness of the fiber-reinforced panel. This test reports a toughness value in joules. Because these panels are made in the field with accelerated shotcrete, care must be taken to make the top surface as smooth as possible. Many specifications call for flexural and residual strength of the shotcrete, but these values cannot be obtained using ASTM C1550. Not many commercial labs in the United States can perform this test, because the throat size of the testing apparatus is not large enough to accommodate the 32 in. (800-mm) diameter panel. This test procedure is currently done largely for quality assurance/control purposes only.	ASTM C1550-10a. 2010. Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel). *Annual Book of ASTM Standards*. Vol. 04.02.

NOTE: ASTM International standards can be found at www.astm.org.

Nozzleman Qualification

Another factor related to testing shotcrete is nozzleman certification. High-quality shotcrete application is more heavily dependent on the capabilities of the nozzleman than are other methods of concrete placement. Significant attention should be placed on the qualifications, training, and experience of all individuals directly involved in shotcrete placement.

The objective of nozzleman certification is to assure the specifier that the employee has the knowledge and capabilities required to perform the work properly (Dufour 2003). Specifiers are advised to require that nozzlemen demonstrate proficiency under simulated job-site conditions (e.g., with representative reinforcement and shooting from positions representative of those available during construction) (Dufour 2003).

Several industry organizations have created certification programs for nozzlemen. In North America, both the American Shotcrete Association (ASA) and the American Concrete Institute (ACI) play roles in the education and certification of personnel involved in shotcrete. ASA oversees training, while ACI oversees certification. The following are of note:

- While ASA previously provided certification, it no longer does so, and ASA certifications are no longer valid.
- ASA offers an underground shotcrete education program.
- The following document has been superseded: ACI 506.3R-91—*Guide to Certification of Shotcrete Nozzlemen* (ACI 1991).

Training and qualification are provided by ACI-qualified trainers for dry-mix process and wet-mix process shotcreting and for both vertical and overhead shooting. It should be noted that a percentage of ACI trainers are experienced in underground work. To be certified in overhead shooting, an examinee must also become certified for vertical shooting in the same process (wet or dry). ACI certification is good for 5 years from the date of completion of all certification requirements. Recertification for an additional 5 years can be obtained in the 6-year period following initial certification. These procedures are not onerous, and the specification writer is encouraged to include minimum training and qualification requirements wherever structural shotcrete is required.

Underground shotcrete application requirements, especially robotic shotcreting, however, may require education and training beyond present ACI offerings, and these should be carried out by qualified examiner/trainers and be specific to anticipated project requirements. Appendix C defines appropriate language for training and qualifications of shotcrete nozzle operators.

QUALITY CONTROL

The objectives of quality assurance (QA) inspection and quality control (QC) testing during construction are not necessarily the same as those of preconstruction testing. The quality of in-situ shotcrete, like concrete, is dependent on proper placement. Once the adequacy of the personnel and the mixture design is established during preconstruction testing, consistency of the applied material is the most important single factor for construction testing and the aim of the QC program during construction.

The best way to mitigate risks that arise during shotcrete application is for the parties, generally the construction personnel and inspectors, to communicate regularly and quickly. In addition, because changes in excavation sequence may be required quickly in response to changes in ground conditions and because these changes may affect design assumptions, it is important for the designer of the shotcrete lining, construction sequence, and heading dimensions to be present on-site full time during construction. Without a full-time presence it is difficult for the designer to retain functional control and responsibility for of the excavation and support results.

There are two types of testing of underground concrete: (1) destructive (cores taken from the tunnel lining directly) and (2) nondestructive (Schmidt hammer or penetration testing). Destructive testing is usually specified (as in Appendix D) as a requirement for sets of three cores drilled into the lining. The cores test for compressive strength and are visually inspected to confirm the density of the shotcrete application. Destructive testing is unquestionably valuable in checking whether the application continues to meet the requirements of the design and the contract documents. However, nondestructive testing has become more common recently and is preferable because it preserves the integrity of the lining, thereby preserving the lining's ability to support the ground and control and reduce ingress of groundwater. ACI 506.4R-94 (reapproved in 2004) introduces the topic of nondestructive testing by stating that "the development of in-place (nondestructive) test procedures for evaluating concrete structures has progressed to the point where the use of such procedures has become common." Given the benefits of nondestructive testing, it is the authors' opinion that it can be used to test shotcrete linings if the following conditions are met:

- The objectives of the construction testing are clearly understood by all sides.
- Sufficient calibration of the testing means can be established in the preconstruction testing, and good correlation is found between the nondestructive and destructive tests for the same properties.
- Sufficiently experienced inspection is available on-site to direct the testing and to react with sufficient speed and authority when deficiencies are found.

If all of the preceding conditions can be established, then a nondestructive testing program can be presented as an alternative to the specified testing program for approval by the owner's representative.

Several methods for shotcrete evaluation can be considered as part of a nondestructive testing program. Schmidt hammer testing of hardened concrete (according to the method described in ASTM C805/C805M and further described by Malhotra and Carino [2006]) provides a quick and easy way to check uniformity of placement, density, and relative strength across a wide area of shotcrete application. There are limitations to this test, and it should not be considered a substitute for strength testing of shotcrete cores. However, when applied properly in a scientific way, it remains an excellent method to check uniformity and the relative strength of one area of shotcrete with another, allowing for a significant potential reduction in the number of cores required. Penetration testing described in the Austrian *Sprayed Concrete Guideline* (Österreichischer Betonverein 1999) is recommended for checking early strength development. Needle penetrometers measure the force required to push a needle of specified dimensions into the freshly applied shotcrete. A Proctor penetrometer (ASTM C403/403M) can be used for this purpose. This device uses a calibrated spring to measure the resistance to penetration, from which material strength can then be calculated.

Later in the development of early strength, penetration tests described in ASTM C803/803M can be applied. A measured amount of energy is applied to a steel pin, and the penetration is measured directly. This test measures not only the surface hardness but also directly measures subsurface properties. This can be related to strength using a standard calculation.

Other in-place measurements include the use of an underreaming drill tool with an expandable insert that can be used for a pull-out test. No ASTM standard currently exists for such a test, but on-site calibrations and use may be applicable under certain circumstances. Sounding with a hammer is a low-cost and low-technology check for lamination of shotcrete or concrete. A "drummy" or hollow sound indicates substandard application, and these areas can then be marked and either repaired or replaced according to the site-approved submittals.

It should be noted that, according to ACI, permeability is an important parameter for shotcrete placement, as it indicates density of application and hence is a strong indication of strength. However, no permeability testing is recommended because of the lack of a standard test and reliability of results from those tests that do exist. For these reasons, it is the authors' recommendation that permeability not be a specified performance requirement of shotcrete linings.

CONCLUSION

Shotcrete material is a fast-developing technology with many valuable codes and references available from all over the world. The recently published ACI guideline (*Guide for Specifying Underground Shotcrete* [ACI 506.5R-09]) is a comprehensive resource for all owners and designers. The recommendations made in this chapter are further described in the Appendix D guidelines to specifiers.

In summary, the following recommendations are made:

- In general, the preferred approach is a performance specification with certain minimum parameters prescribed.
- It is sometimes helpful to provide certain mixture parameters (such as a minimum cement content) to make sure that a proper mixture is priced by the contractor.
- Specialized equipment should be specified for mixing and placement of shotcrete in a strong performance specification.
- Nozzlemen should be certified before placing shotcrete on-site.
- There is no structural or durability reason to avoid using shotcrete as a final lining.
- Blasting rarely has a negative impact on shotcrete applied nearby.
- Designers should be retained to observe construction on a full-time basis.
- Nondestructive testing should be used to its fullest extent during construction.

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CHAPTER 5

Admixtures

Ketan Sompura and Shashiprakash Surali

DEFINITION

An admixture is defined in the American Concrete Institute's *ACI Concrete Terminology* (ACI 2009) and ASTM C125 as "a material other than water, aggregates, hydraulic cement, and fiber reinforcement used as an ingredient of concrete or mortar, and added to the batch immediately before or during its mixing."

REASONS FOR USING CHEMICAL ADMIXTURES

Chemical admixtures are used in concrete or mortar to enhance their properties in fresh or hardened states.

In fresh concrete, chemical admixtures are used for the following:

- Decrease water content without changing workability
- Increase workability without increasing water content
- Retard or accelerate the rate of setting
- Entrain air or reduce density
- Reduce segregation
- Reduce rate of slump loss
- Improve pumpability, placeability, and finishability
- Modify the rate and/or amount of bleeding

In hardened concrete, chemical admixtures are used for the following:

- Accelerate early-strength gain
- Increase strength
- Increase durability
  - Increase resistance to freezing and thawing
  - Reduce scaling caused by deicing salts
  - Decrease permeability
  - Reduce expansion due to alkali–silica reaction
  - Inhibit corrosion of embedded reinforcement
  - Reduce drying shrinkage
- Improve aesthetics and produce colored concrete

TYPES OF CHEMICAL ADMIXTURES

The common types of admixtures and the relevant ASTM standards are listed.

Admixtures for Cast-in-Place Concrete and Precast Concrete

- Air-entraining admixtures conforming to ASTM C260
- Chemical admixtures conforming to the following:
  - Water-reducing admixtures: ASTM C494/C494M, Type A
  - Retarding admixtures: ASTM C494/C494M, Type B
  - Accelerating admixtures: ASTM C494/C494M, Type C
  - Water-reducing and -retarding admixtures: ASTM C494/C494M, Type D
  - Water-reducing and -accelerating admixtures: ASTM C494/C494M, Type E
  - High-range water-reducing admixtures: ASTM C494/C494M, Type F
  - High-range water-reducing and -retarding admixtures: ASTM C494/C494M, Type G
  - Specific performance admixtures: ASTM C494/C494M, Type S (Note: Specific performance admixtures include but are not limited to shrinkage-reducing, alkali–silica reaction-inhibiting, viscosity-modifying and antiwashout (for cast-in-place concrete only), rheology-controlling, and workability-retaining.
  - Corrosion-inhibiting admixtures: ASTM C1582/C1582M
  - Coloring admixtures: ASTM C979

Admixtures for Shotcrete

Admixtures for shotcrete should conform to ASTM C1141/C1141M. Many of the admixtures mentioned previously are also used in shotcrete. These are included in ASTM C1141/C1141M, which recognizes grades of admixtures used in shotcrete made by either of two processes:

1. Type I—Dry-mix shotcrete
   - Grade 1—Accelerating admixture, conventional
   - Grade 2— Retarding admixture
   - Grade 3— Pozzolanic admixture
   - Grade 4— Metallic iron admixture
   - Grade 5— Coloring admixture
   - Grade 6— Organic polymer admixture
   - Grade 7— Not applicable
   - Grade 8—Not applicable
   - Grade 9—Accelerating admixture, quick-setting
2. Type II—Wet-mix shotcrete
   - Grade 1— Accelerating admixture, conventional
   - Grade 2— Retarding admixture
   - Grade 3—Pozzolanic admixture
   - Grade 4— Metallic iron admixture

- Grade 5— Coloring admixture
- Grade 6—Organic polymer admixture
- Grade 7—Water-reducing admixture
- Grade 8—Air-entraining admixture
- Grade 9—Accelerating admixture, quick-setting

Each of the previous grades is further classified by identifying it according to the following classes:

- Class A—Liquid
- Class B—Nonliquid

ADMIXTURE DETAILS

The admixtures detailed in this appendix list various chemical admixtures included in the previous standards. Pozzolanic, metallic iron, and organic polymer admixtures listed in ASTM C1141/C1141M are not included here.

Air-entraining Admixtures

An air-entraining admixture is defined in ASTM C260 as "a material that is used as an ingredient of concrete, added to the batch immediately before or during its mixing, for the purpose of entraining air."

Composition. Air-entraining admixtures are usually formulated with water-soluble compounds such as salts of wood resins, synthetic detergents, salts of petroleum acids, salts of proteinaceous acids, fatty and resinous acids and their salts, and organic salts of hydrocarbons.

Details. An air-entraining admixture should be used when concrete is likely to be exposed to moisture and repeated cycles of freezing and thawing.

During cold weather, when the ambient temperature drops well below the freezing point, water present in concrete pores and microcracks freezes. Volume expansion of up to 9% can occur when water in concrete pores turns to ice. The ice formed exerts pressure and induces tensile stresses in the surrounding concrete if the degree of saturation is greater than 91.7%. Repeated cycles of freezing and thawing under such saturated conditions can cause severe dilation of the cement paste and eventual cracking of the concrete.

When an air-entraining admixture is added to concrete, the admixture causes the development of billions of small-size air bubbles in concrete during the mixing process. These bubbles provide relief from the pressure of freezing water and hence protect the concrete from cracking.

For cement paste to be protected against freezing and thawing damage, the spacing factor as determined by ASTM C457 should not exceed 0.2 mm (0.008 in.). Additionally, the surface area of the air bubbles should be more than 24 mm^2/mm^3 (600 $in.^2/in.^3$) of the air-void volume. ACI 212.3R states that the addition of high-range water-reducing admixtures to air-entrained concrete increases the spacing factor and decreases the surface areas of air-void systems beyond these limits; however, such admixtures do not reduce the freezing and thawing resistance of concrete.

Various ACI specifications provide the required air content for concrete depending on the severity of exposure.

Accelerating Admixtures

An accelerating admixture is defined in ASTM C494/C494M as "an admixture that accelerates the setting and early strength development of concrete."

Composition. Accelerating admixtures are usually made from soluble inorganic salts (such as chlorides, bromides, fluorides, carbonates, thiocyanates, nitrites, nitrates, thiosulfates, silicates, aluminates, and alkali hydroxides), and soluble organic compounds (triethanolamine and calcium formate). Calcium chloride is one of the best known accelerators for portland cement but is not used in reinforced concrete as it promotes corrosion.

Details. The setting and early strength development of concrete is a function of temperature and time. During cold weather, the rate of hydration of cement is slow, which increases the setting time and reduces its early-age strength. When accelerating admixtures are added to concrete, they react with the different components of cement, depending on their base chemical, and accelerate the rate of hydration of cement. Accelerating admixtures are also used in fast-track applications to increase early-age strength development, reduce the stripping time of forms, and, in cold weather, reduce the duration of curing or protection required for the concrete.

RETARDING ADMIXTURES

A retarding admixture is defined in ASTM C494/C494M as "an admixture that retards the setting of concrete."

Composition

Retarding admixtures can be categorized based on their chemical composition into five categories:

1. Hydrocarboxylic acids
2. Lignosulfonic acids and their salts
3. Sugars and their derivatives
4. Carboxylic acids and phosphorus-containing organic acids and salts
5. Salts of amphoteric metals, such as zinc, lead, and tin

The chemicals mentioned in categories 1 to 3 also possess water-reducing properties and can be classified as water-reducing and set-retarding admixtures, namely, ASTM C494/C494M, Type D.

Details

Retarding admixtures are used in concrete to offset the effects of fast setting of concrete due to high temperatures and other factors or to prolong the plasticity of concrete from its batching to placement. When retarding admixtures are added to concrete, they get absorbed onto the cement particles and slow down the early hydration of cement. Retarding admixtures also help to reduce the temperature of concrete in mass concrete placements and can avoid the formation of cold joints when concrete is placed in successive layers.

HYDRATION-CONTROL ADMIXTURES

Hydration-control admixtures (also known as extended set-control admixtures) are highly potent retarding admixtures that can delay the hydration of cement by several hours or days.

Composition

Hydration-control admixtures are based on materials such as carboxylic acids and phosphorus-containing organic acids and salts.

Details

In most applications, conventional retarding admixtures are used to delay the setting time of concrete. However, in certain applications, the use of hydration-control admixtures is preferred. The main advantage of a hydration-control admixture over a conventional retarding admixture is the degree of control over cement hydration in the concrete. If conventional retarding admixtures are added at higher dosages or outside their normal recommended dosage range, they can cause significant retardation of concrete's setting—which could be several hours or even days—and very slow development of concrete strength.

When hydration-control admixtures are used, the rate of increase in retardation with the increase in dosage is much lower when compared to a conventional retarding admixture. This provides the concrete producer improved control over the set time of concrete and avoids the unpredictability and other negative effects of retardation that occur with a normal retarding admixture.

Hydration-control admixtures work by inhibiting the nucleation process of calcium silicate hydrates and calcium hydroxide in concrete. Further, hydration-control admixtures, unlike some conventional retarders, do not cause false set at higher dosages.

Hydration-control admixtures are used in concrete when higher control over retardation is desired, concrete has to be transported for long distances, and for same day and overnight stabilization of returned plastic concrete. They are also used for stabilization of wash water for reuse as part of the mixture water. In tunneling and mining, when the concrete is pumped long distances, hydration-control admixtures are used to increase the workable life of concrete for several hours/days.

Hydration-control admixtures should meet the requirements of ASTM C494/C494M, Type B or D.

WATER-REDUCING ADMIXTURES

ASTM C494/C494M defines a water-reducing admixture as "an admixture that reduces the quantity of mixing water required to produce concrete of a given consistency."

Water-reducing admixtures consist of normal, mid-range, and high-range water-reducing admixtures. Normal and mid-range admixtures should meet the requirements of ASTM C494/C494M, Type A. Normal water-reducing admixtures are required to reduce the quantity of mixing water needed to produce concrete of a given consistency by at least 5% and for slumps of up to 4 in. (10 cm). Higher dosages of normal water reducers may provide more water reduction and slightly higher slumps; however, such dosages may result in retardation of concrete set, increased air content, and a decrease in early-age strength development.

High-range water-reducing admixture, also known as superplasticizer, is defined in ASTM C494/C494M as "an admixture that reduces the quantity of mixing water required to produce concrete of a given consistency by 12% or greater." A high-range admixture should meet the requirements of ASTM C494/C494M, Type F. High-range water reducers function best when used to produce concretes with slumps of 7.5 in. (19 cm) or greater.

Mid-range water-reducing admixtures fill the gap left between normal and high-range water-reducing admixtures. They are formulated to reduce the quantity of mixing water required to produce concrete of a given consistency by 5% to12% without the retardation that can occur when normal water reducers are used within this range of water reduction. Mid-range water reducers are typically used in the slump range of 4 to 8 in. (10 to 20 cm) and some mid-range water reducers can be used also as high-range water reducers.

Combinations of normal, mid-range, and high-range water reducers can be used to provide greater effectiveness and economy, particularly in applications where high dosages of high-range water reducers may be required.

Normal and high-range water-reducing admixtures with set-controlling characteristics are classified in ASTM C494/C494M as follows:

- Water-reducing and -retarding admixture—reduces the quantity of mixing water required to produce concrete of a given consistency and retards the setting of concrete. A water-reducing and -retarding admixture should meet the requirements of ASTM C494/C494M, Type D.
- Water-reducing and accelerating admixture—reduces the quantity of mixing water required to produce concrete of a given consistency and accelerates the setting and early strength development of concrete. A water-reducing and -accelerating admixture should meet the requirements of ASTM C494/C494M, Type E.
- High-range water-reducing and -retarding admixture—reduces the quantity of mixing water required to produce concrete of a given consistency by 12% or greater and retards the setting of concrete. A high-range water-reducing and -retarding admixture should meet the requirements of ASTM C494/C494M, Type G.

Composition

The materials most commonly used as water-reducing admixtures consist of one of the following:

- Lignosulfonic acids and their salts and modifications and derivatives of these
- Hydroxylated carboxylic acids and their salts and modifications and derivatives of these
- Carbohydrate-based compounds such as sugars, sugar acids, and polysaccharides

Similar materials and polycarboxylate ethers are also used in mid-range water reducers. Mid-range water reducers are widely used, because they have little impact on the setting time of concrete mixtures when used to obtain reductions in water content of about 5% to 12%.

Until recently, sulfonated naphthalene condensates, sulfonated melamine condensates, and modified lignosulfonates were the common chemicals used as high-range water-reducing admixtures. Today, polycarboxylate-based high-range water reducers are the most common types.

Details

When water is added to cement, the particles of cement flocculate and hold water in the flocs, thus reducing the amount of water available for workability. When a water-reducing admixture is added, the anions in it get adsorbed on the cement particles and impart a negative charge on them. This negative charge results in the repulsion of cement particles (electrostatic repulsion) that releases the water held in the flocs.

Polycarboxylate-based high-range water-reducing admixtures provide significantly improved cement dispersion over naphthalene- or melamine-based high-range water-reducing admixtures due to their dual mechanism of electrostatic and steric repulsion. Steric repulsion is caused by the side chains in the polycarboxylate molecules that physically keep the cement particles apart.

Water-reducing admixtures are used to achieve the required workability at lower water-to-cementitious-materials (W/C) ratios. High-range water-reducing admixtures can produce large reductions in concrete's water content. The reduced W/C ratio will result in higher strength and improved durability. Mid-range and high-range water-reducing admixtures are also used to produce concretes of high workability such as flowing concrete. Polycarboxylate-based high-range water-reducing admixtures are typically used to produce self-consolidating concrete.

Batch-plant addition of high-range water-reducing admixtures provides much better control and uniformity relative to on-site addition.

SPECIFIC PERFORMANCE ADMIXTURES

ASTM C494/C494M defines a specific performance admixture as "an admixture that provides a desired performance characteristic(s) other than reducing water content, or changing the time of setting of concrete, or both, without any adverse effects on fresh, hardened and durability properties of concrete as specified herein, excluding admixtures that are used primarily in the manufacture of dry-cast concrete products." Some of the specific performance characteristics included in this category of admixtures are shrinkage reduction, mitigation of alkali–silica reaction, and viscosity modification. Many other admixtures fall under this category, as detailed in the following sections.

Shrinkage-reducing Admixtures

Shrinkage-reducing admixtures are used to reduce the drying shrinkage of concrete and minimize the potential for drying shrinkage cracking.

Shrinkage of concrete is a result of induced capillary forces occurring during drying. Surface tension forces in the capillary pores of concrete generated during drying exert inward pulling force on the walls of the pores. This capillary tensile force leads to the shrinkage of concrete.

Shrinkage-reducing admixtures function by reducing the surface tension of water within the pores of concrete. This leads to a reduction in the capillary tension and pull on the walls of the pores and, consequently, a reduction in drying shrinkage. The main component of a shrinkage-reducing admixture is polyoxyalkaline alkyl ether.

Benefits of shrinkage-reducing admixtures include

- Reduced drying shrinkage at both early and later ages,
- Reduced curling and rate of curling in slabs, and
- Improved watertightness and durability.

The magnitude of drying shrinkage can be significantly reduced through the use of a shrinkage-reducing admixture. They reduce the drying shrinkage by 50% to 80% during the short term and 30% to 40% in the long term, depending on the dosage.

In order to use the most appropriate and economical dosage of a shrinkage-reducing admixture, it is necessary to specify the amount of drying shrinkage required. High dosages of shrinkage-reducing admixtures can negatively affect properties such as air content, bleeding, setting time, and strength of concrete. ASTM C157/C157M and ASTM C1581/C1581M are the test methods used to determine the drying shrinkage and assess the effectiveness of shrinkage-reducing admixtures.

Shrinkage-reducing admixtures should meet the requirements of ASTM C494/C494M, Type S.

Alkali–Silica Reaction-inhibiting Admixtures

Alkali–silika reaction (ASR) is a chemical reaction that occurs when the alkali hydroxides present in the pore solution of the concrete react with certain forms of reactive silica present in the aggregates to form an alkali–silica gel. This gel is harmless, but in the presence of moisture it swells and generates tensile stresses in the concrete, eventually causing the concrete to crack. The main source of alkalis (sodium and potassium) in fresh concrete is portland cement. However, some fly ashes may also contribute to the alkali content of concrete.

Admixtures used to minimize deleterious expansions in concrete due to ASR are based on lithium compounds such as lithium nitrite, lithium carbonate, and lithium hydroxide. The most commonly used lithium-based admixture is a 30% solution of lithium nitrate. Only lithium admixtures based on lithium nitrate are discussed in this section.

Lithium nitrate's mechanism of action in concrete is as follows: In the pore solution of the concrete, dissolved silica can react with alkali hydroxide (sodium or potassium) ions to form an alkali–silica gel that can absorb water and expand with deleterious effects on the concrete. If lithium ions are present in the pore solution in a sufficient ratio to the sodium and potassium ions, they preferentially react with the available silica to form relatively stable, insoluble lithium silicates. Normally, a ratio of lithium ion to the sum of the sodium and potassium ions of 0.74 on a molar basis is sufficient to minimize deleterious expansions of concrete to an acceptable level. A higher ratio may be required in case of very reactive aggregates.

Benefits of using lithium nitrate admixtures are the reduction in deleterious expansions in concrete due to ASR and the ability to use local aggregates. The dosage of lithium nitrate admixture should be calculated according to manufacturer's recommendations and is determined by calculating the total amount of alkalis contributed by cementitious materials per cubic yard of concrete. Performance tests such as ASTM C1260, ASTM C1567, ASTM C1293, or CRD-C662 should be conducted to determine the dosage of lithium nitrate required to minimize deleterious expansions in concrete to acceptable levels.

Lithium nitrate admixtures for ASR mitigation are expensive. However, the dosage and treated cost of lithium nitrate admixture can be reduced if used synergistically with appropriate levels of supplementary cementitious materials such as Class F fly ash, slag cement, metakaolin, and silica fume. The synergistic performance should be verified through testing.

Alkali–silika reaction-inhibiting admixtures should meet the requirements of ASTM C494/C494M, Type S.

Viscosity-modifying Admixtures

A viscosity-modifying admixture enhances concrete performance by modifying the viscosity and controlling the rheological properties of the concrete mixture. Viscosity-modifying admixtures are used in conjunction with high-range water-reducing admixtures to produce self-consolidating concrete. Materials used for viscosity-modifying admixtures include cellulose ether, welan, or diutan gum.

Viscosity-modifying admixtures should meet the requirements of ASTM C494/C494M, Type S.

Antiwashout Admixtures

Antiwashout admixtures increase the cohesiveness of concrete to be placed under water, thus reducing the washout of cementitious materials. The most common antiwashout admixtures are based on natural or synthetic gums and cellulose-based thickeners.

The U.S. Army Corps of Engineers has a standard specification for antiwashout admixtures (CRD-C661-06) and a test method to determine the effectiveness of an antiwashout admixture (CRD-C61).

Antiwashout admixtures should meet the requirements of ASTM C494/C494M, Type S or, if applicable, CRD-C661-06.

Rheology-controlling Admixtures

Rheology-controlling admixtures are a new type of admixture introduced in 2008. They can be defined as concrete additives that control the deformation response of a bulk concrete mixture to applied energy. A proprietary formula of rheology-modifying polymers is used in the manufacture of these admixtures.

Rheology-controlling admixtures improve the workability or flow of bulk concrete and improve the response to vibration without a significant change in slump. They do not affect any of the other properties of concrete.

While a rheology-modifying admixture is mainly used in dry-cast mixtures, a major application is in self-consolidating concrete.

Rheology-controlling admixtures should meet the requirements of ASTM C494/C494M, Type S.

Workability-retaining Admixtures

Workability-retaining admixtures are a new generation of admixtures that were introduced in the early 21st century. They can be defined as admixtures that allow concrete or mortar to maintain its fresh characteristics throughout the transporting, placing, consolidating, and finishing operations without adversely affecting the time of setting and hardened properties of concrete or mortar. A workability-retaining admixture is used as part of an admixture system and in combination with a water-reducing admixture. A proprietary formula is used in the manufacture of workability-retaining admixtures.

Benefits of using a workability-retaining admixture include

- Workability retention without retardation,
- Flexible levels of workability retention by adjusting dosage, and
- Improved early and late age strength.

Workability-retaining admixtures should meet the requirements of ASTM C494/C494M, Type S.

Permeability-reducing Admixtures

Permeability-reducing (waterproofing) admixtures improve concrete durability by reducing the water and moisture movement in the concrete. These admixtures reduce the penetration of water into the concrete and reduce the water transmission through concrete.

Permeability-reducing admixtures generally consist of materials from either of the following chemical families:

- Hydrophobic or water-repellent chemicals that include materials based on soaps and long-chain fatty acid derivatives, vegetable oils, and petroleum. These admixtures reduce concrete permeability by forming hydrophobic films within the capillary pores of the concrete.
- Inert fillers (talc, bentonite, siliceous powders, clay, hydrocarbon resins, and coal tar pitches), chemically active fillers (lime, silicates), and supplementary cementitious materials (fly ash, slag cement, silica fume). These materials fill the fine pores in concrete and reduce the overall porosity of the concrete.
- Crystalline materials consisting of proprietary active chemicals provided in a cement and sand carrier. These materials work by increasing the density of the calcium silicate hydrate and blocking pores by forming deposits.

Permeability-reducing admixtures can be utilized in concrete for all applications. Use of these admixtures offers several benefits, such as reduction in dampness, water migration, and leakage; resistance to freeze–thaw damage, corrosion, and carbonation; and efflorescence control. Permeability-reducing admixtures are generally used in concrete slated for water containment structures, below-grade structures, tunnels and subways, bridges and dams, and recreational facilities such as aquatic centers.

The selection of a permeability-reducing admixture depends largely on the service condition of the concrete structure. The effects of different permeability-reducing admixtures vary widely. Some of the tests used to evaluate the effectiveness of permeability-reducing admixtures against water ingress into concrete are DIN 1048 Part 5, BS EN 12390-8, CRD-C48, and ASTM C1585.

Shotcrete Accelerators or Quick-Setting Accelerators

Shotcrete accelerators are admixtures that are added to sprayed concrete at the point of its discharge (at the hose's nozzle) during the wet or dry shotcrete process. In the case of dry shotcrete, the accelerator could be a powder that may be preblended with the dry mix during manufacturing.

In wet shotcrete, depending on the pumping distance, concrete is pumped at moderate to high workability. It is difficult to place the shotcrete without shotcrete accelerators, as the concrete will not adhere to the substrate and can reduce rebound and prevent higher buildup. When a shotcrete accelerator is added to concrete, it quickly reacts with the cement (tricalcium aluminate component) and causes early stiffening of the cement paste. This enables better bonding of the concrete to the substrate, increases buildup, and significantly reduces rebound. The effect of different shotcrete accelerators can vary widely. Some shotcrete accelerators decrease only the set time of concrete, whereas others decrease the set time and also increase the early-age strength of concrete.

The main accelerator ingredient of shotcrete accelerator may consist of sodium aluminates, sodium and potassium hydroxide, or carbonates, triethanolamine, ferric sulfate, aluminum sulfate, and sodium fluoride. Old-generation shotcrete accelerators were alkali based, while new-generation accelerators are mostly alkali free.

The benefits of using alkali-free shotcrete accelerators include high early-strength development, faster construction, faster mining cycles due to early reentry, safe working environment for workers in tunnels and mines, reduction in rebound and dust, improved bond to substrate, overhead spraying, and reduction in groundwater pollution by alkalis' leaching. Applications for shotcrete accelerators include immediate support and final lining in tunnels, mines, and rock and slope stabilization.

Factors that can affect the performance of shotcrete accelerators are mixture proportions, cement type, temperature conditions, and shotcrete application.

Corrosion-inhibiting Admixtures

A corrosion-inhibitor is defined in *ACI Concrete Terminology* (ACI 2009) as "a chemical compound, either liquid or powder, usually intermixed in concrete and sometimes applied to concrete, and that effectively decreases corrosion of steel reinforcement."

The most common corrosion-inhibiting admixtures are calcium nitrite and amine/esters, which are effective in reducing the chloride-induced corrosion of steel in concrete. In addition, they also increase the chloride threshold of concrete.

Calcium nitrite–based corrosion-inhibiting admixtures function by forming a protective layer at the steel surface as a result of a chemical reaction with ferrous ions. Commercially available calcium nitrite–based corrosion-inhibiting admixtures are 30% solutions. These admixtures accelerate the setting time of concrete and therefore need a retarding admixture in conjunction. Typical dosage of calcium nitrite–based corrosion-inhibiting admixtures is 2 to 6 gal/yd^3 (10 to 30 L/m^3). These admixtures should meet the requirements of ASTM C1582/C1582M.

Amine/ester-based organic corrosion-inhibiting admixtures function by forming a protective layer at the steel surface and by reducing chloride ion ingress. The reduction in chloride ingress is caused by the admixtures' lining of walls of pores. These are used at a standard dosage of 1 gal/yd^3 (5 L/m^3).

A higher than normal dosage of air-entraining admixture may be needed to achieve the required air content in an air-entrained concrete with an amine/ester-based corrosion-inhibiting admixture. Depending on the ingredients and proportions of the concrete mixture, a decrease in compressive strength may be experienced, typically in the range of 5% to 10%. These deficiencies can be overcome with proper mixture proportioning.

Amine/ester-based organic corrosion-inhibiting admixtures should meet the requirements of ASTM C494/C494M, Type S, and ASTM C1582/C1582M.

Coloring Admixtures

Coloring admixtures are added to provide color to the concrete. Pigments for integrally colored concrete should conform to ASTM C979. Coloring admixtures can be used to provide a wide range of colors.

GUIDELINES FOR USE OF ADMIXTURES

Admixtures should be used in accordance with and at the dosages recommended by the manufacturer. Various factors affect their performance, including the characteristics of cement, supplementary cementitious materials and aggregates, concrete temperature, ambient temperature, and mixing and consolidation processes. When adequate information is unavailable or when a combination of admixtures is used—a common practice—it is necessary to conduct trials with project materials to determine the dosages of admixtures necessary to achieve the desired properties. Guidelines to this effect are provided in ACI 212.3R.

SPECIFICATION LANGUAGE

The relevant admixtures and text for inclusion in "Part 2 Materials" of a specification in CSI format can be chosen from the following list. Because materials technology changes so quickly, for product names and manufacturers the reader is advised to contact representatives of admixture material suppliers for the latest developments and materials for specification. Admixture product names may be included in specifications. It is recommended that if products are specified, they are not done so on a sole-source proprietary basis but that flexibility is provided for the contractor with an "or equal" or other similar designation to leave open the possibility of innovation and for new suppliers to enter the marketplace, enhancing competition.

Text for inclusion under "Part 2 Materials" of a specification:

2.1 Characteristics: Compatible with each other and free of intentionally added chlorides

2.2 Air-entraining admixture

A. Shall conform to ASTM C260

2.3 Water-reducing admixture

A. Shall conform to ASTM C494/ C494M, Type A

2.4 Mid-range water-reducing admixture

A. Shall conform to ASTM C494/C494M, Type A

2.5 High-range water-reducing admixture

A. Shall conform to ASTM C494/C494M, Type F (or ASTM C1017/C1017M, Type I)

2.6 Accelerating admixture

A. Shall conform to ASTM C494/C494M, Type C or E

2.7 Shotcrete accelerators or quick-setting accelerators

A. Shall conform to ASTM C1141/C1141M

2.8 Retarding admixture

A. Shall conform to ASTM C494/C494M, Type B or D

2.9 Hydration-control admixture:

A. Shall conform to ASTM C494/C494M, Type B or D

2.10 Workability-retaining admixture

A. Shall retain concrete workability without affecting setting time or early-age strength development

B. Shall conform to ASTM C494/C494M, Type S

2.11 Viscosity-modifying admixture

A. Shall conform to ASTM C494/C494M, Type S

2.12 Antiwashout admixture

A. Shall conform to ASTM C494/C494M, Type S

2.13 Rheology-controlling admixture

A. Shall conform to ASTM C494/C494M, Type S

2.14 Permeability-reducing admixture (waterproofing admixture)

2.15 Corrosion-inhibiting admixture

A. Shall conform to ASTM C1582/C1582M

2.16 Shrinkage-reducing admixture

2.17 Alkali–silica reaction-inhibiting admixture

A. Shall contain nominal lithium nitrate content of 30%

2.18 Coloring admixture

2.19 Other admixtures shall be approved by the engineer

REFERENCE

ACI (American Concrete Institute). 2009. *ACI Concrete Terminology*. Farmington Hills, MI: ACI.

NOTE: For full references for standards, visit Web sites for ASTM International (www.astm.org), German Institute for Standardization (DIN, www.din.de), British Standards Institution (BS, www.bsigroup.com), and U.S. Army Corps of Engineers (CRD, www.usace.army.mil).

APPENDIX A

Cast-in-Place Concrete Tunnel Lining Specifications

PART 1. GENERAL

1.1 SUMMARY

A. This section specifies the requirements of cast-in-place (CIP) concrete tunnel lining, including concrete materials; mixture proportioning requirements; furnishing, erecting, and removing formwork; waterstops; placement procedures; finishes; and curing and protecting CIP concrete.

B. Related sections

1. *Call out section number and title*
2. *Call out section number and title*
3. *Call out section number and title*
4. *Call out section number and title*

1.2 REFERENCE STANDARDS

A. American Concrete Institute (ACI)

1. ACI 117 *Specification for Tolerances for Concrete Construction and Materials*
2. ACI 207.1 *Guide to Mass Concrete*
3. ACI 301 *Specifications for Structural Concrete*
4. ACI 304.2R *Placing Concrete by Pumping Methods*
5. ACI 305R *Hot Weather Concreting*
6. ACI 306R *Cold Weather Concreting*

B. American Society for Testing and Materials (ASTM)

1. ASTM C31 *Test Methods for Making and Curing Concrete Test Specimens in the Field*
2. ASTM C39 *Test Method for Compressive Strength of Cylindrical Concrete Specimens*
3. ASTM C94 *Specification for Ready-Mixed Concrete*
4. ASTM C40 *Test Method for Organic Impurities in Fine Aggregates for Concrete*

COMMENTARY

This specification is for CIP tunnel linings only and not for elevated slabs, columns, or beams that might be placed underground such as in a subway station.

Related sections could include reinforcement, rock support, waterproof membrane installation, control of water, waterstops, and construction joints.

Delete the reference standards that do not apply.

5. ASTM C117 *Test Method for Materials Finer Than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing*
6. ASTM C127 *Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate*
7. ASTM C128 *Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate*
8. ASTM-C131 *Test Method for Resistance to Degradation of Small Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine*
9. ASTM C136 *Test Method for Sieve Analysis of Fine and Coarse Aggregates*
10. ASTM C138 *Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete*
11. ASTM C142 *Test Method for Clay Lumps and Friable Particles in Aggregates*
12. ASTM C143 *Test Method for Slump of Hydraulic Cement Concrete*
13. ASTM C150 *Specification for Portland Cement*
14. ASTM C172 *Practice for Sampling Freshly Mixed Concrete*
15. ASTM C231 *Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method*
16. ASTM C260 *Specification for Air-Entraining Admixtures for Concrete*
17. ASTM C309 *Specification for Liquid Membrane-Forming Compounds for Curing Concrete*
18. ASTM C494/C494M *Specification for Chemical Admixtures for Concrete*
19. ASTM C566 *Test Method for Total Evaporable Moisture Content of Aggregate by Drying*
20. ASTM C595/C595M *Specification for Blended Hydraulic Cements*
21. ASTM C618 *Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete*
22. ASTM C989 *Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars*
23. ASTM C1064/C1064M *Test Method for Temperature of Freshly Mixed Hydraulic-Cement Concrete*
24. ASTM C1074 *Standard Practice for Estimating Concrete Strength*

25. ASTM C1077 *Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation*
26. ASTM C1107/C1107M *Specification for Packaged Dry, Hydraulic-Cement Grout (Non-shrink)*
27. ASTM C1116/C1116M *Specification for Fiber-Reinforced Concrete and Shotcrete*
28. ASTM C1260 *Test Method for Potential Alkali Reactivity of Aggregates*
29. ASTM C1293 *Test Method for Determination of Length Change of Concrete Due to Alkali–Silica Reaction*
30. ASTM C1567 *Test Method for Determining the Potential Alkali–Silica Reactivity of Combinations of Cementitious Materials and Aggregates (Accelerated Mortar-Bar Method)*
31. ASTM C1602/C1602M *Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete*
32. ASTM D75/D75M *Practice for Sampling Aggregates*
33. ASTM D2419 *Test Method for Sand Equivalent Value of Soil and Fine Aggregates*

C. U.S. Army Corps of Engineers

1. CRD C572 *U.S. Army Corps of Engineers Specification, Polyvinyl Waterstops*

D. National Ready Mixed Concrete Association

1. *National Ready Mixed Concrete Association, Quality Control Manual—Plant Certification*

1.3 QUALITY ASSURANCE

A. Contractor shall provide the services of Independent Testing Laboratory (ITL) meeting requirements of ASTM C1077 to demonstrate conformance with specified requirements.

The requirement on a large tunnel project to have outside laboratory perform independent testing of placed concrete is normal.

B. Conform to requirements of ACI 301, except as modified herein.

ACI 301 provides extensive specifications for structural concrete.

C. Complete pre-placement checkout form prior to placing concrete.

1.4 SUBMITTALS

A. Field quality control data

B. Shop drawings and product data

1. Concrete mixture designs and test reports.

Designs and reports may require substantial lead time if locally established mixes have no tunnel history.

2. Formwork drawings showing details of construction, number of tunnel form sections, form joints, access openings, and design calculations. Submit __ days before commencing concrete operations.

   Steel tunnel forms are a long lead time item and must typically be submitted at least 3 months in advance of delivery to job. Submittals should include form geometry, tolerances, bracing, and anchoring. Areas left blank are to be considered carefully for project-specific needs by specifier.

3. Form installation and concrete placement method including: description of the method of form installation and concrete placement for underground work, including method and procedure for setting, connections, inspection, stripping and moving tunnel forms; and method of concrete conveyance
4. Panning and invert drain material

   This applies to rock tunnels without waterproofing systems.

5. Embedded item placement drawings
6. Curing procedure
7. Manufacturer's data
   a. Waterstops
   b. Admixtures
   c. Curing compounds
   d. Bonding and repair materials
   e. Pozzolanic and cementitious materials
   f. Fibers (where used)

      This could be any material fiber—steel, macrosynthetic, or microsynthetic.

8. Certified test reports
   a. Before delivery of materials or concrete, submit certified reports of the tests specified herein.
   b. To certified test reports on previously tested materials, add manufacturer's certified statement that previously tested material is the same type, quality, manufacture, and make as that proposed for use in this work.
   c. Certified test reports are required for the following:
      1) Cement
      2) Aggregates
      3) Supplementary cementitious materials
      4) Chemical admixtures
      5) Curing compounds
      6) Bonding and repair materials
      7) Concrete mixture designs
      8) Field trial mixes
9. Rock excavation contact cleaning preparation plan

   This applies in bare rock tunnels, not in shotcreted areas.

10. Daily reports and records of concrete placement, including but not limited

to concrete volumes, stationing of concrete placement, locations and quantities of panning material installed, lengths of pours, time of pours, slump and cylinder sample preparation, concrete delivery tickets showing batch weights, batch times, and discharge times.

1.5 MASS CONCRETE

A. Classify tunnel lining concrete as mass concrete where the tunnel lining has enough volume to require that measures be taken to mitigate heat generation from hydration of cement and attendant volume change to minimize cracking. Analyze concrete linings that exceed 3 ft 0 in. (0.9 m) in thickness to see if additional measures are required.

Mass concrete is traditionally defined in North America as having thickness greater than 3 ft (0.9 m), triggering special measures to limit thermal differential cracking. Depending on the concrete mixture and the placement situation, special measures may not be required or may be limited if thorough analysis of heat generation is performed.

B. Ensure that mass concrete conforms to all requirements of this specification section except that Type II, low-heat cement shall be used, with a low C3S and C3A.

Low-heat cement is difficult to obtain and is usually a special-order item that requires cement supplier to allocate separate storage space. Cement manufacturer will not make low-heat cement for quantities required to concrete most tunnels. It is better to use supplementary cementitious materials such as Type F fly ash or slag cement.

C. Contractor may prepare thermal calculations that show whether concrete will experience internal temperature differentials greater than 35°F (1.6°C) and temperature exceeding 60°F (15°C). If the contractor can show that requirements for temperature control can be achieved by other means such as use of fly ash, slag cement, or precooling of concrete, requirements to use low-heat cement may be waived by engineer.

ACI 207.1R and the Portland Cement Association describe protective measures that prevent excessive temperature generation in mass concrete as well as temperature differential between internal concrete and surface of concrete.

D. Submit separate mixture designs along with contractor's calculations and written thermal control plans to control temperature and temperature differentials where mass concrete is required.

Contractor has opportunity to analyze and determine whether cooling measures normally associated with mass concrete are necessary. These are typically done with semi-adiabatic calorimeter and vendor software.

PART 2. PRODUCTS

2.1 READY-MIXED CONCRETE

A. Supplied by National Ready Mixed Concrete Association (NRMCA) certified batch plant

NRMCA has an established and practical certification process commonly used for ready-mix batch plants.

2.2 MATERIALS

A. Concrete materials

1. Cement

a. ASTM C150/C150M, Type II

b. Nonfalse setting and low alkali, containing less than 0.60% alkalis

Note that most manufacturers' Type II cement also satisfies requirements for Type I cement, but the reverse is not always the case.

c. Requirement for low-alkali cement shall be waived if test results

The need or otherwise for special cement is discussed in Chapters 1 and 2.

provided by the supplier show that aggregate source consists of nonalkali reactive aggregate when tested in accordance with ASTM C1293 or C1567.

2. Aggregates
   a. General
      1) Fine and coarse aggregates
         a) Meet ASTM C33/C33M gradation.
         b) Handle fine and coarse aggregates as separate ingredients.
         c) All aggregates shall be nonreactive
         d) Wash before use.
         e) Perform specified tests prior to commencing concrete work.

Check local history of aggregates for alkali–silica reactivity (ASR). New source may not have documented history. Extensive ASR testing per ASTM C1260, C1293, and C1567 may be required.

      2) Variations from specified gradations in individual tests acceptable if average of three consecutive tests is within specified limits and variation is within permissible variation listed by ASTM C33
   b. Fine aggregate
      1) Hard, dense, durable particles of either sand or crushed stone regularly graded from coarse to fine meeting requirements of ASTM C33
      2) Fine aggregate content of concrete mixtures not exceeding 40% by weight of total aggregate content
      3) Sand equivalent value no less than 75 when tested in accordance with ASTM D2419
      4) Fine aggregate for each mixture design from a single source

This does not prohibit use of aggregate from more than one location. Separate mixture design for each separately sourced aggregate needed.

   c. Coarse aggregate
      1) Hard, dense, and durable gravel or crushed rock free from injurious amounts of soft and friable particles, alkali, organic matter, and other deleterious substances and meeting requirements of ASTM C33 class designation __

ASTM C33 specifies classes of aggregates depending on end use of concrete.

1 in. (0.25 mm) is a good size for a pumpable concrete mixture. Most tunnel liner concrete is pumped into final location.

      2) Before and during field trial mixes, minor adjustments to gradation in Section 2.2.A.2.c.1 to produce specified concrete

3) Coarse aggregate for each mixture design from single source

Use of aggregate from more than one location is not prohibited. Separate mixture design for each separately sourced aggregate is needed.

3. Pozzolanic materials
   a. Pozzolan supplied for all mixture designs shall be from same single source

Fly ash or slag cement is commonly used to lower peak heat development and improve long-term strength. Typically one or the other is used but not both.

   b. Fly ash: Class F conforming to ASTM C618

Type F fly ash is less reactive than Type C and normally more consistent.

4. Ground granulated blast-furnace slag cement
   a. Slag cement supplied for all mixture designs shall be from same single source.
   b. Grade 100 or Grade 120 slag cement shall conform to ASTM C989.
5. Chemical admixtures

Use of hydration-control admixtures such as Delvo and Recover can be useful for long delivery chains where maintaining slump is important.

   a. General
      1) Compatible with cement and with other admixtures
      2) Admixtures containing intentionally added chlorides unacceptable

Where possible, minimize the use of superplasticizer in long (>300 ft or 91 m) pumplines. If pumping is stopped for long, there is a better chance of resuming pumping if mixture is not reliant on superplasticizer for slump.

      3) Admixtures used in accordance with manufacturer's recommendations and added separately to concrete mixture
   b. All chemical admixtures shall meet requirements of ASTM C494/C494M.
   c. Air-entraining admixture shall meet requirements of ASTM C260.
6. Water
   a. Meet requirements of ASTM C1602/C1602M.

Previous requirement was that mixture water had to be potable water. The pressure to reuse wash water has led to development of ASTM C1602. Beware of using wash water without extensive testing as water can change set time characteristics.

   b. If reclaimed water is used, provide ASTM C1602 data and define process control method in place by concrete supplier.
   c. Potable water may be used without additional testing.
7. Fiber reinforcement (if required):

Addition of fibers will normally decrease slump and increase need for water or superplasticizer.

   a. Specify material such as steel or synthetic.
   b. Fibers should meet requirements of ASTM C1116/C1116M.

ACI 544.3R contains guidelines for specification of fiber-reinforced concrete.

   c. Specify applicable performance characteristics.
8. Forms
   a. Design forms to resist loads expected in accordance with contractor's means and methods.

ACI 347R contains guidelines for minimum formwork pressures under various circumstances.

b. Describe form finish required based on final use of exposed concrete.

B. Concrete

1. Classification and use

a. Concrete classification: If more than one mixture design is planned for use in tunnel lining concrete, specify type of use, location, and class designation in a table.

2. Mixture proportioning

a. Concrete mixture proportions: If more than one class of concrete, specify proportions in a table.

b. Maximum size aggregate typically 1 in. (25 mm) or less for nonmass concrete).

c. Maximum water/cementitious material ratio (typically 0.42 to 0.46).

d. Percent replacement by pozzolan by weight of cementitious material (typically 15% to 20%)

e. Percent replacement by slag cement by weight of cementitious material (typically 15% to 50%)

f. Air content (0%–3% entrapped for no exposure to freeze–thaw, 4%–6% entrained for freeze–thaw exposure)

g. Maximum slump specified with or without high-range water-reducing admixture (typically 5 in. [127 mm] without and 8 in. [203 mm] with superplasticizer)

h. Self-consolidating concrete (SCC) shall be allowed with slump exceeding specified maximum if proof of performance without segregation is provided

i. Minimum 28-d compressive strength of ____ psi.

j. Where specifically approved by engineer, high range water reducer may be added in concrete mixes:

1) This increases workability and facilitates concrete placement in areas of high reinforcement congestion.

2) Maximum slump of 8 in. (203 mm) should not be exceeded for all concrete containing the high-range water-reducing admixture (superplasticizer) at point of placement.

3) For all concrete mixes utilizing a high-range water reducer

Most tunnel linings have steel form finish not exposed to public view. Do not overspecify finish. Public areas such as subway stations where exposed concrete serves as architectural finish will require detailed finish standard.

Some projects may have just one classification for tunnel. Other projects may have different classifications for portals, shafts, tunnel linings, mass concrete fills.

For pumped concrete, 1 in. (25 mm) is recommended.

It is better to not specify cementitious content of CIP mixtures and let trial mixtures show minimum required content. While U.S. codes are silent, European codes suggest cement content in the range of 474–540 lb/yd^3 (281–320 kg/m^3) and fly ash content in the range of 85–135 lb/yd^3 (50–80 kg/m^3).

European specifications typically allow 0.50 to 0.55. Water-to-cementitious-materials (W/C) ratios below 0.46 require use of superplasticizer for pumped concrete. A lower W/C content is recommended here to balance high strength with strong preference for high durability for underground concrete.

15% to 20% Type F fly ash mixes can be designed using normal procedures. Content could be increased above 20% if qualifying performance data is provided.

15% to 50% slag cement mixes can be designed using normal procedures. Content could be increased above 50% if qualifying performance data is provided.

Non-air-entrained concrete has higher early strength for faster form stripping and can be used in areas without freeze–thaw exposure. It usually has entrapped air in 0%–3% range.

Slump varies with maximum aggregate size. SCC mixes can have slumps up to 10 in. (254 mm) and slump flow up to 32 in. (813 mm).

Concrete tunnel linings usually have low stresses, and the 4,000-psi (27.5-MPa) requirement is the typical minimum. Special serviceability requirement may require higher strengths. High-slump concrete must resist segregation in the pumpline and at point of placement. Plugs in the pumpline will result if mixture segregates or bleeds in pumpline. Concrete does travel behind the form and must remain coherent mass. Higher-slump concrete is often required where reinforcing steel density is high and opportunities for internal vibration are limited.

producing a concrete mixture with slump greater than 5 in. (127 mm), if segregation develops, reduce slump.

This is common in steel forms but there is even greater need in wood forms to which form vibrators are not normally attached.

C. Waterstops

1. Specify material.
2. Specify size.
3. Specify acceptable manufacturer.
4. Specify maximum head.
5. Protect waterstops from damage during construction.
6. Waterstop selection shall be based on rated head of application.

D. Curing compound

1. Materials for curing concrete shall meet requirements of ASTM C309, C1315, or C171.

ASTM C309 is for liquid curing compounds.ASTM C1315 is for liquid curing and sealing compounds. ASTM C171 is sheet materials for curing.

2. Curing compounds shall be compatible with follow-on architectural finishes (if applied).

E. Panning material

1. Continuously extruded dimpled sheet material. The minimum compressive strength of panning material normal to sheet should be 2,500 psf (0.12 MPa) at 10% deflection of dimples.

Panning material applies only to tunnels with exposed bare rock with no waterproofing system.

F. Mixing and transportation facilities

Backup requirement reasonable when the concrete supplier has several batch plants in area with same aggregate supplies. This is not always possible when batch plant is at remote job site or if plant is on job site and independent from local suppliers.

1. General
   a. Facility design and layout to be reviewed by engineer.
   b. Provide for transport of concrete from mixer to point of placement without segregation.
   c. Engineer shall have access to batch plant for necessary inspections.
2. Design
   a. Consistently produces uniform product meeting specifications
   b. Has adequate capacity to meet maximum demand
   c. Has backup in case of system failure
   d. Is easily inspected for cleanliness and wear

When developing new mixture, do not skip the lab trial batch process and proceed to field trials. Knowledge of what mixture looks like and how it should behave before proceeding to field trials is needed.

G. Conveying equipment

1. General
   a. Ensure that material in contact with concrete does not contaminate concrete (such as aluminum).
   b. Provide discharge facilities at end of conveying equipment, which

prevents segregation of concrete constituents.

2. Maintenance
   a. Maintain equipment in proper operating condition.
   b. Clean after each placement is completed.

2.3 SOURCE QUALITY CONTROL

A. Concrete control tests

1. Mixture design
   a. Before beginning concrete work, determine proper proportions of materials for each strength and class of concrete.
   b. Ensure that mixture consists of specified cement, pozzolan or cement slag, admixtures, aggregate, and water.
   c. Ensure that methods for selecting and adjusting proportions of ingredients are in accordance with ACI 211.1.
   d. Proportion all mixture designs in accordance with Section 5.3, "Proportioning on the Basis of Field Experience and/or Trial Mixtures," of ACI 318.
   e. Submit mixture designs on each class of concrete.
   f. If trial batches are used, have mixture design prepared by ITL and achieve a compressive strength 1,200 psi (8 MPa) higher than specified 28-d strength. Increase this over design to 1,400 psi (10 MPa) when concrete strengths over 5,000 psi (34 MPa) are used. If more than 20% pozzolan or 50% slag is utilized for cement replacement, 56-d or 90-d strengths shall be used in place of 28-d strengths.
   g. Accompany the proposed mixture designs with complete standard deviation analysis of trial mixture test data as per ACI 214R.
   h. Ensure that certified reports of each mixture design state whether items reported comply with specifications and show:
      1) Design strength and average required strength per ACI 318, Section 5.3
      2) Maximum slump
      3) Weights and test results of ingredients

Trial batches for the record are normally done by ITL. Contractor may have substantial involvement or may do trial batches to determine optimum mix.

Take minimum of three each 4 × 8 test cylinders or two each 6 × 12 test cylinders. In addition to tests, technician should also calculate W/C ratio and yield. Yield should be in range of 27 ft^3/yd^3 + 0.5 ft^3/yd^3. Later testing than 28 d could take place as 56 d or 90 d.

4) Other physical properties necessary to check each mixture design.

2. Field trial mix

   a. After acceptance of mixture designs and prior to concrete placement, establish field proportions for classes of concrete required, based on accepted design mixes.

   b. Manufacture field trial concrete using equipment to be used for work.

   c. Make adjustments in design mixes to provide dense, homogeneous, durable concrete with good workability and finishing qualities.

   d. The ITL shall take four sets of three standard test cylinders from the field trial mixes for each concrete design and test in accordance with ASTM C39. Samples will also be tested for slump (ASTM C143), air content (ASTM C231), density (ASTM C138), and air and ambient temperature (ASTM C1064) by an ACI Level I field technician.

   e. Perform compression strength tests for each set of cylinders at 7 d, 14 d, 28 d, and at later ages as specified.

   f. Notify engineer one week in advance of field trial mixture work.

   g. Perform field trial mixture work in presence of engineer and ITL.

   h. Do not place production concrete prior to field trial mixture requirements being met as specified.

PART 3. EXECUTION

3.1 INSTALLATION

A. Concrete

1. Use ready-mixed concrete conforming to applicable portions of ASTM C94.

2. Proportion materials by weight.

   Only weigh batch plants can be NRMCA certified.

3. Weigh and introduce pozzolan or ground granulated blast furnace slag (GGBFS) into mixer with cement and other components of concrete mix. Pozzolan or GGBFS should not be introduced into wet mixer ahead of other materials or with mixing water.

4. Introduce water at time of charging mixer to provide that specified slump or water-to-cementitious material ratio is not exceeded.

5. Arrange with the ITL for inspection as required to comply with these specifications.
6. Do not allow retempering of concrete that has set.
7. Only allow addition of water at site to ready-mix concrete as outlined in ASTM C94 under the following conditions:
    a. Maximum W/C ratio has not been exceeded.
    b. Maximum allowable mixing and agitation time (or drum revolution) are not exceeded. (300 revolutions or 1½-h mixing time are limitations.) Extended agitation with chemical admixtures requires preapproval.
    c. Concrete is remixed for at least half the minimum mixing time or number of revolutions in accordance with ASTM C94.
    d. Maximum allowable slump is not exceeded.
8. Redosage with specified high-range water-reducing admixture (superplasticizer) may be done with prior approval of engineer regarding dosage and time periods delivered to site. Discharge concrete within 90 min after introduction of mixing water when ambient temperature is below 75°F (24°C) and within 60 min when ambient temperature is 75°F (24°C) or above where concrete batched loses slump. Discharge requirements may be extended with use of hydration stabilizers if proof of performance is provided during field trials.

B. Tunnel formwork

1. Conform to ACI 347R.
2. Ensure all form joints are mortar tight.
3. Ensure that it can be cleaned, oiled, and moved quickly to new locations.
4. Design and construct all formwork systems to provide only lines and delineations indicated.
5. Thoroughly clean forms previously used of all dirt, mortar, and foreign matter before being used.
6. Provide with openings (windows) of not less than 4 ft^2 (0.37 m^2) each at not more than 8-ft (2.4-m) centers at each springline and at crown, the crown opening to alternate on either side of the tunnel centerline. Pumping ports may be substituted for crown openings.

Most owners and engineers will require that ITL take samples and make tests on fresh concrete. The point where sample is taken is often a matter of dispute. Long tunnels normally do not have a location where the test cylinders can be safely stored. As long as no water is added to mixture, the last transfer point before entering tunnel is reasonable location to take samples. Air and slump tests can be taken closer to point of placement if desired.

Some batch tickets state maximum amount of additional water that can be added. Otherwise calculation must be done by hand.

7. Maintain plane and true surfaces where exposed; take special care at all cold joints to match adjacent pour surfaces.
8. Construct forms with spacing of supports and ties such that they will support maximum pressure exerted by fluid concrete with actual deflection not to exceed 1/360 times the support spacing.
9. Provide openings in forms to facilitate cleaning and inspection prior to concrete placement, where necessary.
10. Ensure that forms design limit free fall of concrete to less than 6 ft (1.8 m), and locate openings to facilitate placement and consolidation of concrete.
11. Ensure that reusable steel form collapses easily without damaging concrete.
12. Remove tunnel arch forms for tunnels smaller than 20 ft (6 m) in diameter when compressive strength reaches minimum of 600 psi (4 MPa). Establish concrete compressive strength by using maturity meter following ASTM C1074 or by other means approved by engineer.
13. Remove tunnel arch forms for tunnels larger than 6.1 m (20 ft) in diameter after compressive strength reaches minimum of 84 kg/cm (1,200 psi). Establish concrete compressive strength by using maturity meter following ASTM C1074 or by other means approved by engineer.
14. For tunnel arch forms larger than 20 ft (6 m) in diameter, allow earlier form removal if contractor can show by structural analysis that the established concrete compressive strength at time of form removal is adequate to resist construction applied loading with adequate design factor of safety. Establish concrete compressive strength by using maturity meter following ASTM C1074 or by other means approved by engineer.

European codes call for removing forms when the concrete has compressive strength of 218 to 435 psi (1.5 to 3 MPa). Depending on the country, forms must be stripped when strengths reach 435 to 725 psi (3 to 5 MPa). From a practical testing standpoint, 600 psi (4 MPa) is the minimum point at which a definite "break" occurs when performing compressive strength tests.

ASTM C1074 maturity meter method provides a reliable indication of concrete compressive strength when calibrated to particular mixture design's strength gain characteristics.

ACI 347R Section 6.6.4 recommends minimum stripping time of 12 h for exposed surfaces and 8 h for bulkheads. Then stripping times should be reduced in 30-min increments based on site experience.

C. Preparation for concrete placement

1. Remove unexcavated materials, timber lagging, blocking, and wedges that project within the structure cross section shown on plans.
2. Remove timbers outside structure cross section that are not required for direct support of ground during concrete placement.
3. Remove steel supports outside structure cross section that are not required

for direct support of ground or anchoring during concrete placement.

4. Remove all loose material on excavated surfaces except loose material and detached rock wedged behind lagging or blocking that cannot safely be removed.

When establishing standard of cleanliness, designer should consider design requirements for the lining. Concrete is not required to bond to rock. Normally tunnel walls and invert should be free of loose material or muck that would prevent concrete from being placed to full depth or thickness.

5. Remove loose or soft material from tunnel invert. Compacted granular muck below the minimum excavation limit may be left in place with engineer's approval.
6. Perform such other work as may be required to provide that surfaces in contact with fresh concrete are free from standing or running water, mud, dried mortar or grout, loose material, oil, organic debris, frost, or ice.

D. Panning installation

1. At a minimum, panning coverage includes all intermediate area along circumference of tunnel wall from invert drain to highest point of observed water movement. Panning should be installed full circle only when inflows are coming into tunnel directly at crown and cannot be controlled by panning on one side of tunnel only.

This applies to bare rock tunnels without waterproofing system installation.

2. Installation of upper area panning during the excavation phase is permitted.
3. All inflows should be covered by a lap of 6 in. (15 cm). Overlapping of adjacent panning sheets is required to extent necessary to satisfactorily contain water inflows.
4. Panning material should be attached to walls securely by pins and washers or other devices proposed by contractor, in a manner acceptable to engineer and that will provide integrity of support and minimize risk of bursting, leaking, or deformation.

E. Invert drain installation

1. Remove groundwater so as to prevent hydrostatic pressure from damaging uncured concrete and to prevent groundwater from eroding fresh concrete.

This applies to bare rock tunnels without waterproofing system installation.

2. Separately collect excess flows occurring inside completed lining upstream from point of concrete placement and divert through or around fresh concrete being placed.

F. Installing steel reinforcement: Conform to Section 03200–Concrete Reinforcement

G. Conveying and placing concrete

1. Ensure that everything from mixer to forms is in accordance with ACI 301.
2. Discharge concrete within 90 min of adding water unless mixture contains hydration stabilizer tested by preparing trial batches, in which case limitation on time to discharge is established by trial batch.
3. Ensure that methods of conveying concrete to point of final deposit prevent segregation or loss of ingredients. At initial point of entry from pipe into tunnel forms, concrete should not move laterally more than 30 ft (9 m).
4. Place concrete
   a. General
      1) Place in accordance with ACI 301 and ACI 304R.
      2) Prior to form erection and concrete placement, clean all surfaces against which concrete will be placed.
      3) Prior to placing concrete, apply coating of release agent on forms in accordance with manufacturer's requirements.
      4) Install all grout and vent pipe as approved for contact grouting.
      5) Continuously deposit concrete until section of approved size and shape is completed.
      6) Do not place concrete in water.
      7) Do not permit water to run over concrete until concrete is sufficiently hardened to prevent erosion.
      8) Ensure that surface of hardened concrete upon which fresh concrete is to be placed is rough, clean, sound, and damp. Before placement of plastic concrete, hardened surface shall be cleaned of all laitance and foreign substances (including curing compound), washed with clean water, and wetted thoroughly.
      9) If placing concrete by slickline over top arch of form, end of discharge line shall be buried in fresh concrete during placement. Line shall be buried a minimum of 5 ft (1.5 m). Mark end of discharge line so as to readily indicate depth of burial at all times.

10) Contractor shall inform engineer at least 24 h in advance of times and places at which he or she intends to place concrete. The limits of each concrete pour (placement) shall be determined by contractor and shall be acceptable to engineer. All concrete within predetermined limits shall be placed in one continuous operation.

b. Place concrete by pumping

1) General

a) Use pumping equipment designed to handle types, classes, and volumes of concrete to be placed without segregation.

b) If pumped concrete does not produce required end results, discontinue pumping operation and rectify problems before concreting proceeds.

2) Pumping equipment

a) Have alternate standby pump available within 30-min notice to avoid cold joints in structural elements being placed.

This is a practical requirement only when pumping from surface or pumping where rubber-tired portal access is available. It is not likely that a pump can be quickly replaced in single track heading deep underground. In some situations, contractor would do better to have alternate method to clear concrete and save the slickline. In long tunnels, a second pump available large enough to pump concrete through slickline may not be available.

b) Ensure that minimum diameter of hose conduits is in accordance with ACI 304.2R.

c) Replace pumping equipment and hose conduits that are not functioning properly.

d) Do not permit aluminum conduits for conveying concrete.

c. Hot weather concrete placement

Comply with ACI 305R guidelines.

1) Use concrete materials and proportions that have records in field use under hot weather conditions.

Delivery of concrete to tunnel job site in hot weather can be critical if batch plant is more than 20 min away. Temperatures of the rock and forms in the tunnel are typically in the 50–60°F (10–15°C) range, even with hot weather outside.

2) Provide measures to cool concrete if high temperature, low slump, flash set, cold joints, or shrinkage cracks are encountered.

In areas with known extreme temperature conditions, having separate concrete mixes for different temperature ranges is suggested.

a) By utilizing cool ingredients before mixing

b) By adding flake ice or well-crushed ice of a size that melts completely during mixing for all or part of mixing water

c) By using liquid nitrogen injection

3) Use concrete consistency that permits rapid placement and effective consolidation.

4) Transport, place, consolidate, and finish concrete with the least delay.

5) Plan work, as much as possible considering all other requirements, to avoid adverse exposure of concrete to environment; schedule placing operations during times when weather conditions are favorable.

6) Protect concrete against moisture loss at all times during placing and during its curing period.

7) Ensure that temperature of concrete when placed:

a) Does not exceed 90°F (32°C)

b) Does not exceed conditions that cause difficulty in placement due to loss of slump

8) Do not allow water additions in excess of the proportioned maximum water content to compensate for loss of workability.

It is better to use a hydration stabilizer such as Delvo or Recover than to keep adding water.

d. Cold weather concrete placement (less than 50°F [10°C])

Comply with ACI 306R guidelines.

1) Do not place concrete on substrates that are below 32°F (0°C) or contain frozen material.

Ventilation can pull cold air far inside tunnel. Rock temperatures normally stay in the 50–60°F (10–15°C) range, but form may require insulation if very cold air is being drawn into tunnel.

2) Keep temperature minimums for concrete discharged at site:

In areas with known extreme temperature conditions, having separate concrete mixes for different temperature ranges is suggested.

a) At least 40°F (4°C) air temperature at point of placement

b) Minimum concrete temperature of 60°F (15.5°C) for section least dimension less than 12 in. (30 cm).

c) Minimum concrete temperature of 55°F (13°C) for section least dimension greater than 12 in. (30 cm).

3) Maintain all materials, forms, reinforcement, subgrade, and any other items that concrete may come in contact with free

of frost, ice, or snow at time of concrete placement.

H. Consolidating concrete

1. Consolidate concrete in accordance with ACI 301. SCC concrete does not need to be consolidated.
2. Attach form-type vibrators rigidly to form and operate at 8,000 rpm or more. Coordinate location and operation of vibrators, position, and discharge rate to ensure that crown of arch remains full during placing operation. Utilize immersion-type vibrators through access windows in forms as necessary to ensure proper consolidation of concrete.
3. Suspend concrete placing until proper consolidation can be achieved.
4. Number of vibrators and size of vibrator heads shall be in accordance with manufacturer's recommendations for work conditions.
5. Keep spare vibrators of each size and type at site during all concrete-placing operations.
6. Do not troll vibrators to pull concrete ahead within forms.
7. Do not place vibrators between reinforcing and form.
8. At each insertion, use sufficient duration to consolidate concrete, but not to cause segregation or bleeding.
9. Work concrete thoroughly around reinforcement and embedded fixtures and into corners of forms, eliminating all air or aggregate pockets, that may cause honeycombing, pitting, or planes of weakness.
10. If, during placing operation, there is delay of more than 15 min, vibrate into previous lift just prior to placing new concrete.

I. Curing

1. Acceptable curing conditions are to keep relative humidity of tunnel air above 90%. If these conditions cannot be maintained for 6 d after placement, then cure concrete by keeping concrete continuously moist for 6 d or applying curing compound immediately after form removal.
2. Absorptive mats or fabric may be used to retain moisture during the curing period.

Use and type of vibrator should conform to ACI 309R. This applies to steel forms only. Wood forms cannot take vibration.

Immersion-type vibrators are of limited utility in heavily reinforced tunnels. Vibrator can only reach from side window down to the toe of form. Vibrator is difficult to insert into concrete and can be easily trapped in reinforcement. Usually vibrator shaft is cut off and left in concrete when head is trapped.

Vibration of joint between invert slabs and tunnel sidewall with immersion-type vibrators is possible provided engineer makes clearance allowances when designing reinforcement. Alternatively, engineer can allow contractor freedom to shift bar locations at form windows.

3. Coordinate repairs or treatment of concrete surfaces so that interruption of curing is not necessary.
4. Minimize temperature variations of air in contact with concrete surface to _____.
5. Apply and maximize coverage quantities of curing/sealing compound per manufacturer's instructions.
6. Do not use spray-on type membrane on construction joints and other surfaces where bond is required or where surface repairs are to be made.

J. Protection

1. Protect concrete from injurious action by flowing water, frost, and mechanical injury.
2. Do not allow concrete to dry out from time it is placed until expiration of curing periods.
3. Protect finished concrete surfaces from gouging, chipping, excessive heat, overstress, and other damage during construction.

K. Construction joints

1. General
    a. Place concrete in each unit of construction continuously.
    b. Before new concrete is placed on or against concrete that has set, retighten forms and clean the surface of set concrete.
    c. Concrete surfaces shall be wetted but not saturated immediately prior to placing of fresh concrete.
2. Preparation
    a. Produce rough surface of exposed fine aggregate at construction joints.
    b. Clean construction joints either by high-pressure water jetting, wet sandblasting, or combination thereof. Clean splashed concrete from projecting waterstops.

L. Embedments (if required)

1. Unless otherwise indicated, set and secure in forms, prior to concrete placement, all frames, special castings, channels, angles, or other materials that are to be embedded in concrete.
2. Embed in concrete, as indicated on drawings, all anchor bolts and inserts.

3. Use nailing blocks, plugs, strips, and the like necessary for attachment of trim, finish, and similar work.
4. Temporarily fill voids in sleeves, inserts, and anchor slots with readily removable material to prevent entry of concrete into voids.

M. Expansion joints (if required)

1. Install expansion joints as indicated on drawings.
2. Do not extend reinforcement or other embedded metal items bonded to concrete through expansion joints.
3. Waterstops required in expansion joints are indicated on drawings.

N. Waterstops

1. Securely hold waterstops in position during placing of concrete.
2. If, after placing concrete, waterstops are materially out of position or shape, remove surrounding concrete, reset waterstop, and replace concrete as indicated.
3. Protect from damage during construction.
4. Use expansive waterstops at junction of new and existing concrete or as indicated on drawings; install per manufacturer's recommendations.
5. Locations
   a. Expansion and construction joints in structures
   b. As indicated on drawings
6. Field splices are acceptable only in straight sections.
7. Fabricate crosses, tees, and other shapes prior to delivery to site.

O. Grouting of panning material and invert drain

This applies only in bare rock tunnel with no waterproofing system.

1. Do not perform grouting of panning material and invert drain until concrete in tunnel liner has cured for at least 7 d.
2. Grout invert drain using low applied pressure to cut off inflows, followed by grouting of panning material.

P. Repair of surface defects in newly placed concrete

Best appearance is achieved when concrete is patched immediately after form stripping.

1. Use for repairing surface voids, spalls, rock pockets, tie holes, and minor honeycombing in formed concrete.
2. Perform repairs within 24 h of form removal.

3. Neatly remove fins and encrustations from exposed surfaces.
4. Remove defective concrete to sound concrete before repair is performed.
5. Clean areas to be repaired.
6. Repair damaged or defective area with initial cut of 1 in. (2.5 cm) deep made with masonry saw around damaged area.
7. Do not allow feather edging of replaced materials.
8. Place and pack approved concrete repair material in layers as prescribed in Section ___.

   Reference Specification section here for concrete patching and finishing
9. Solidly compact each layer over entire surface by use of hardwood stick or hammer.
10. Cure approved concrete repair material by approved methods.
11. Protect finished surfaces from stains and abrasions.
12. Ensure that finishes are equal in workmanship, texture, and general appearance to that of adjacent concrete.
13. Correct concrete with honeycombing, which exposes the reinforcing steel, or with defects, which affect structural strength.

3.2 FIELD QUALITY CONTROL

A. General

1. Have field sampling and testing performed by ITL.
2. Sample aggregates at random locations from stockpiles. Take samples of concrete from predetermined locations and at such times to represent the quality of materials and work throughout the contract.
3. Sample aggregates not less than 30 d prior to the use of such aggregates in the work in order to verify gradation, cleanliness, and specific gravities and absorption.
4. Minimum number of samples and tests are specified in Section 3.2.D.
5. Provide safe and suitable facilities for obtaining samples.

B. Inspection

1. Notify engineer 24 h in advance that form and reinforcement will be ready for inspection.
2. Complete pre-placement inspection form prior to placing concrete.

3. Ensure that inspection notes loose bars, formwork either not properly aligned or not properly sealed, and loose wire, which might interfere with appearance of finished surfaces.
4. Where items such as anchors, fastenings, conduit, piping, and so on are included, accept placement prior to placing any concrete.

C. Sampling: Sample materials as follows.

1. Aggregates
   a. General
      1) Sample fine and coarse aggregates in accordance with ASTM D75.
      2) Take samples at discharge gates of storage bins.
      3) Obtain samples at concrete batch plant at frequency specified.
      4) Repeat sampling when source of material is changed or when unacceptable deficiencies or variations from specified requirements of materials are found in testing.
   b. Tag and identify sources for aggregate samples.
   c. Coarse aggregate
      1) Take sample weighing between 50 and 60 lb (23 and 27 kg) after batch plant is brought up to full operation.
      2) Take samples so that a uniform cross section, accurately representing materials on belt or in bins, is obtained.
      3) Take samples for sieve analysis of coarse aggregate, and specific gravity and absorption.
   d. Fine aggregate
      1) Take samples as specified for coarse aggregate.
      2) Take samples for sieve analysis of cleanliness, specific gravity, and absorption.
      3) Take samples of sand when it is moist.

Use visual inspection to determine if a washed sieve analysis should be run to determine the percent passing a #200 sieve.

2. Concrete
   a. Obtain samples of plastic concrete in accordance with ASTM C172.
   b. At start of initial concrete pumping placements, provide testing of concrete samples at point of delivery

Sampling tunnel concrete at point of placement is difficult and sometimes impossible. Better to establish upfront the air entrainment loss at point of placement and take samples at nearest convenient point. Concrete with more entrained air has lower compressive into form and compare with nearest convenient sample collection point:

1) Determine if any significant changes in unit weight, slump, air content, and other mixture characteristics occur.

2) Make appropriate adjustments to concrete mixes delivered to site to correct variations in mixture characteristics as specified.

c. Take concrete samples for slump, air content, unit weight, temperature, and test cylinders at convenient point of collection after final delivery characteristics of mixture have proved acceptable for unit weight, slump, and air content.

strength, so the sample taken before air loss is conservative.

Check for leaking gaskets at pumpline joints if losing significant amounts of entrained air. Slump loss in pumping measured between the pump inlet hopper and the hose discharge should not exceed 1.5 in. (4 cm).

D. Testing

1. Aggregate

a. Make minimum of one test of coarse aggregate per 400 yd^3 (306 m^3) of concrete.

b. Make a minimum of one test of fine aggregate per 200 yd^3 (153 m^3) of concrete.

c. Confirm continuing conformance with specifications for gradation, cleanliness, and sand equivalent.

d. Require minimum of one test per day of each aggregate.

e. Take aggregate moisture tests prior to start of batching and at sufficient frequency thereafter to ensure uniform batch-to-batch consistency.

2. Concrete

a. Strength tests

1) Verify compressive strength by testing standard cylinders of concrete samples taken at job site.

2) Represent concrete placed with standard cylinders ___ in. in diameter × ___ in. long.

3) Cast one set of cylinders of each class of concrete for each 100 yd^3 (76.5 m^3) poured per day in each separate structure of each class of concrete. If concrete mixture contains more than 20% pozzolan or 25% slag and will be tested at 90-d, cast ___ standard cylinders.

4) Cast, handle, and cure cylinders in accordance with ASTM C31.

Standard test cylinders are usually 6 in. (15 cm) diameter × 12 in. (30.5 cm) long or 4 in. (10 cm) diameter × 8 in. (20 cm) long. The 6 × 12 cylinders are the old standard, with 4 × 8 cylinders becoming more common. Specifications typically require that 6 × 12 concrete cylinders be broken in groups of two while 4 × 8 cylinders are broken in groups of three. Some prefer to break test cylinders in groups of three regardless of size.

Using 6 × 12 cylinders, a group of four cylinders would be cast per 100 yd^3 (76.5 m^3) and 6 cylinders if 90-day testing is performed.

Using 4 × 8 cylinders, a group of six cylinders would cast per 100 yd^3 (76.5 m^3) and nine cylinders if 90-day testing is performed.

Storage boxes cannot be kept at the tunnel lining location as location changes every day.

5) Take additional cylinders when error in batching is suspected.	
6) For the first 24 h after casting, keep cylinders moist in storage box constructed and located so that its interior air temperature is between 60°F and 80°F (15.5°C and 26.6°C).	
7) At the end of 24 h, transport cylinders to testing laboratory.	
8) Test specimens for compressive strength in accordance with ASTM C39.	
9) Make tests at 7, 28, and 90 d from time of casting, as appropriate.	
10) Test ___ standard test cylinders from each group of ___ at end of 7 d and ___ cylinders at end of 28 d.	When using 6 × 12 test cylinders, cast four. Test two at 7 d and two at 28 d. When using 4 × 8 test cylinders, cast six. Test three at 7 d and three at 28 d.
11) For concrete mixes with more than 20% pozzolan or 25% GGBFS, modify compression testing such that ___ standard cylinders from each set of ___ will be tested at the end of 7 d, ___ cylinders will be tested at the end of 28 d, and ___ cylinders will be tested at the end of 90 d.	When using 6 × 12 test cylinders, cast six. Test two at 7 d, two at 28 d, and two at 90 d. When using 4 × 8 test cylinders, cast nine. Test three at 7 d, three at 28 d, and three at 90 d.
12) Where compression tests are to be used to determine when forms may be removed, make at least ___ additional standard cylinders and cure on-site in accordance with ASTM C31. Compression cylinder tests will be waived if strength is estimated using ASTM C1074 maturity meter method.	This method is not indicative of in-place concrete and will result in form stripping times beyond what is necessary. Use this method only when no maturity meter data is available. Delete paragraph 12 if using maturity meter ASTM C1074. Test-cylinder strengths will typically lag behind in-place strengths at early ages (less than 72 h). Typical would be to take two 6 × 12 cylinders or three 4 × 8 cylinders.
13) Require additional tests until sufficient number of satisfactory tests have been performed to establish early strength of concrete in accordance with ACI 214R. Waive additional testing if strength is estimated using ASTM C1074 maturity meter method.	
14) Ensure that each final strength test result is average of strengths of ___ standard test cylinders at 28 d (or 90 d for concrete with more than 20% pozzolan or 25% slag).	With 6 × 12 cylinders, use the average of two cylinders. With 4 × 8 cylinders, use the average of three cylinders.
15) Ensure that average of any three consecutive 28-d or 90-d strength test results, as appropriate, of cylinders representing	

each class of concrete for each structure is equal to or greater than specified strength and not more than 10% of the strength test results to have values less than specified 28-d or 90-d strength for total job concrete.

16) Ensure that no individual strength test results are less than specified strength by more than 500 psi (3.5 MPa).

Acceptance criteria are different if mixture design strength is above 5,000 psi (34.5 MPa). ACI 318-08 is the reference document for acceptance criteria.

17) Evaluate 28-d strength test results (or 90-d strength for concrete with more than 20% pozzolan or 25% slag) in accordance with ACI 214R.

18) If 28-d test (or 90-d strength for concrete with more than 20% pozzolan or 25% slag) results fall below specified compressive strength for class of concrete required for any portion of the work, make adjustments in proportions, water content, or both, as necessary. Report changes and adjustments in writing to engineer.

19) If compressive test results indicate concrete in place may not meet structural requirements per the concrete class, make tests to determine if structure or portion thereof is structurally sound in accordance with ACI 301 criteria. Tests may include, but not be limited to, cores in accordance with ASTM C42 and any other analyses or load tests acceptable to engineer.

b. Tests for consistency of concrete

1) Perform slump tests whenever standard cylinders are cast.

2) Take samples for slump determination from concrete during placing.

3) Specify slump when measured in accordance with ASTM C143.

Take additional slump test when pumping concrete long distances (>300 ft [91 m]).

4) Make additional tests at beginning of concrete placement operation and at subsequent intervals to ensure that specification requirements are met.

c. Tests for temperature

1) Take whenever standard cylinders are cast.

2) Measure temperature in accordance with ASTM C1064.

3) Measure at additional frequency intervals if necessary during hot or cold weather conditions until satisfactory temperature control is established.

d. Tests for air content

1) Take whenever standard cylinders are cast.

2) Measure in accordance with ASTM C231.

e. Tests for density and yield of concrete

1) Take whenever standard cylinders are cast.

2) Measure in accordance with ASTM C138.

E. Sampling and testing reports

1. Include in test reports sufficient information to identify design mixture used, the stationing or location of concrete placement, and quantity placed.
2. Note slump, air content, temperature of concrete, ambient temperature, and unit weight.

NOTE: For more information about standards and publications cited here, visit the American Concrete Institute Web site (www.concrete.org) and ASTM International Web site (www.astm.org).

APPENDIX B

Precast Concrete Segmental Lining Specifications

PART 1. GENERAL

1.1 SUMMARY

A. This section specifies requirements for bolted, gasketed, precast concrete segments for final tunnel lining.

1.2 RELATED SECTIONS

A. *Call out section number and title.*

B. *Call out section number and title.*

C. *Call out section number and title.*

D. *Call out section number and title.*

1.3 REFERENCE STANDARDS

A. Standards referenced shall be most current versions.

B. American Concrete Institute (ACI)

1. ACI 211.1 *Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete*
2. ACI 214R *Evaluation of Strength Test Results of Concrete*
3. ACI 301 *Specification for Structural Concrete*
4. ACI 305.1 *Specification for Hot Weather Concreting*
5. ACI 306.1 *Specification for Cold Weather Concreting*
6. ACI 309R *Guide for Consolidation of Concrete*
7. ACI 318/318M *Building Code Requirements for Structural Concrete*
8. ACI 350/350M *Code Requirements for Environmental Engineering Concrete Structures and Commentary*

COMMENTARY

This specification is for precast concrete segmental linings only and not for other precast elements that might be placed underground, such as in subway station.

This section is complete specification that deals with concrete requirements, manufacturing process, and installation of lining. An alternative approach is to have these sections in separate specifications. Experience has shown that utilizing generic project concrete specification for segment production can lead to requirements that do not apply to precast concrete.

Related sections include reinforcement, gaskets, and tunnel excavation.

Delete the reference standards that do not apply.

9. ACI 503.4 *Standard Specification for Repairing Concrete with Epoxy Mortars*
10. ACI 517 *Accelerated Curing of Concrete at Atmospheric Pressure*
11. ACI 533R *Guide for Precast Concrete Walls*

C. American Society for Testing and Materials (ASTM)

1. ASTM A82 *Standard Specification for Steel Wire, Plain, for Concrete Reinforcement*
2. ASTM A123 *Standard Specification for Hot Dip Galvanizing*
3. ASTM A185 *Standard Specification for Steel Welded Wire Reinforcement*
4. ASTM A325 *Standard Specification for Structural Bolts*
5. ASTM A496 *Standard Specification for Steel Wire, Deformed for Concrete Reinforcement*
6. ASTM A497 *Standard Specification for Steel Welded Wire Reinforcement, Deformed for Concrete*
7. ASTM A615 *Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*
8. ASTM A820 *Standard Specification for Steel Fibers for Fiber-Reinforced Concrete*
9. ASTM C31 *Standard Practice for Making and Curing Concrete Test Specimens in the Field*
10. ASTM C33 *Standard Specification for Concrete Aggregates*
11. ASTM C39 *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*
12. ASTM C42 *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*
13. ASTM C94 *Standard Specifications for Ready Mixed Concrete*
14. ASTM C136 *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates*
15. ASTM C138 *Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete*
16. ASTM C143 *Standard Test Method for Slump of Hydraulic-Cement Concrete*
17. ASTM C150 *Standard Specification for Portland Cement*

18. ASTM C157 *Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete*
19. ASTM C171 *Standard Specification for Sheet Materials for Curing Concrete*
20. ASTM C172 *Standard Practice for Sampling Freshly Mixed Concrete*
21. ASTM C231 *Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method*
22. ASTM C260 *Standard Specification for Air-Entraining Admixtures for Concrete*
23. ASTM C309 *Liquid Membrane-Forming Compounds for Curing Concrete*
24. ASTM C494 *Specification for Chemical Admixtures for Concrete*
25. ASTM C618 *Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete*
26. ASTM C989 *Standard Specification for Ground Granulated Blast-Furnace Slag*
27. ASTM C1064 *Standard Test Method for Temperature of Freshly Mixed Hydraulic-Cement Concrete*
28. ASTM C1074 *Standard Practice for Estimating Concrete Strength by the Maturity Method*
29. ASTM C1116 *Standard Specification for Fiber-Reinforced Concrete*
30. ASTM C1202 *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*
31. ASTM C1240 *Standard Specification for Silica Fume*
32. ASTM C1293 *Standard Test Method for Determination of Length Change of Concrete Due to Alkali–Silica Reaction*
33. ASTM C1315 *Standard Specification for Liquid Membrane-Forming Compounds Having Special Properties for Curing and Sealing Concrete*
34. ASTM C1567 *Standard Test Method for Determining the Potential Alkali–Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method)*
35. ASTM C1602 *Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete*
36. ASTM C1609 *Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading)*

37. ASTM D75 *Standard Practice for Sampling Aggregates*
38. ASTM D2419 *Standard Test Method for Sand Equivalent Value of Soils and Fine Aggregate*
39. ASTM E1529 *Standard Test Methods for Determining Effects of Large Hydrocarbon Pool Fires on Structural Members and Assemblies*

D. American Welding Society (AWS)

1. AWS D1.4 *Structural Welding Code—Reinforcing Steel*

1.4 DEFINITIONS

A. Circumferential joint: Segment joint face between adjacent rings

B. Gasket: Water-sealing system consisting of continuous, deformable, elastomeric (EPDM) gasket attached to each segment mating face to provide permanent water-tightness for erected tunnel lining

C. Packing: Load-distributing material attached to segment joints, to distribute compressive stresses across joints

D. Radial joint: Segment joint face radial to center of ring

E. Segment stripping: Removal of cast concrete segment from mold after initial curing at predetermined minimum concrete compressive strength

F. Tunnel lining: Precast concrete segments with steel reinforcement, stirrups, ties, etc., and appurtenances fixed together to produce a series of complete rings. Appurtenances may include gaskets, plugs, bolts, dowels, alignment rods, ball sockets, grommets, packing, etc.

1.5 SUBMITTALS

The submittals listed here are based on the design-bid-build process with full design for segmental lining provided by contractor. If contractor has design responsibility, then additional submittals should be added.

A. Furnish submittals in accordance with Section 01300—Contractor Submittals.

B. Qualifications

1. Tunnel lining manufacturer
2. Tunnel lining manufacturer's supervisor
3. Segment mold manufacturer
4. Segment reinforcement manufacturer
5. Segment designer

C. Shop drawings, including:

1. All segment types, layout, number of segments per ring, taper configuration, key configuration, concrete grade and type, packing, segment identification information, reinforcement, mechanical joint connection assemblies, dowels, bolts, connectors, gasket grooves,

gaskets, grout/threaded lifting sockets and plugs, inserts, and accessories necessary for manufacture, transportation, erection and performance of lining system

2. Key dimensions, spacing, and layout of elements including plans, sections, elevations, and segment configurations for changes in alignment and grade

3. Complete details of all forms, including number of molds, details for securing embedded items in place during casting, form geometry, and fabrication tolerances

D. Concrete mix designs and test reports, including:

1. Constituents
2. Water tests
3. Trial mixes
4. Results of tests on trial mixes

E. Detailed work plans including drawings, procedures, QC plans, and descriptions for manufacturing, casting, curing, stripping, handling, transporting, storing, fixing of gaskets, and packing. Information shall include but not be limited to:

1. Location and layout of facilities for casting, curing, and storing segments

2. Segment marking to be used—that is, segment type, date cast, and serial number, including form identity

3. Production schedule for segment manufacture

4. Curing process for segment casting to 28 d after casting, including following:

   a. Methods to protect segments from direct sunlight and weather if stored outside

   b. Measures to control shrinkage and temperature cracking of segments

5. Manufacturer's certification confirming that products conform to relevant ACI and ASTM standards

6. Methods to verify that design strength of segments has been attained prior to stripping and use on-site

7. Manufacturer product data and test data demonstrating compliance with this specification for all materials and inserts used in manufacturing, handling, and assembly of tunnel lining as well as any admixtures, curing compounds, bonding/repair materials, pozzolanic/cementitious materials, and

fibers that will be used in proposed concrete mix

8. Two samples of each manufactured insert or component
9. Segment repair and rejection criteria and procedures at factory, on-site, and in stockyard

F. Results of dimensional checks on molds, segments, and trial ring assemblies

G. Assembly procedures, including sequencing of ring erection, bolting, adjusting, packing, jacking, grouting, and details of installing segments without causing segment damage and within ring build tolerances, for making alignment/grade corrections/adjustments and for tunneling on curves

1. Methods to verify proper assembly in tunnel, including details of planned ring-build compliance checks and corrective actions
2. Details of mechanisms by which packing and thrust jacks will be kept in their intended positions on lining segments to avoid segment damage

H. Rejection criteria and procedures for tunnel liner inspection prior to installation

I. Procedure and products for filling holes made for temporary construction hangers, tunnel rails, sleepers, etc.

J. Procedure and products for repair of concrete segments

K. Daily tunnel report shall include:

1. Sequentially numbered accounting of segment rings erected in tunnel correlated to tunnel station line, level, roll, key location, bar codes of segments, and notations of segment damage
2. Records of any nonconformance of tunnel liner and remedial actions carried out

Designer qualifications are not required if segments are fully designed prior to bid.

L. Weekly manufacturing report shall include:

1. Summary of weekly production

M. Weekly tunnel report shall include:

1. Cumulative flow through entire tunnel

N. As-built survey record: Provide monthly record, including as-built survey record of measurements of every fifth segment ring of diameter measured at invert, springline, and approximately 45 degrees to axis of each ring erected. Record station of ring at center of segment at invert. Make measurements with precision of 0.01 in. (0.25 mm)

1.6 QUALITY ASSURANCE

A. Precast segment manufacturer qualifications

1. Tunnel lining manufacturer shall be qualified company regularly engaged in manufacture and fabrication of precision bolted, gasketed precast concrete tunnel lining segments of similar dimensions and tolerances to those specified, and which has provided precision tunnel rings in last five years for at least three large public projects comparable to work of this contract in size and type.

The requirement for accordance to AWS D1.4 is only for structural welds. If this is not included in the design and only welding is run-of-the-mill tack welds for assembly of cages, this may be unnecessary. Most precasters have their own crews assembling and welding cages.

2. Manufacturing shall be under supervision of tunnel lining manufacturer's supervisor, who shall be fully qualified and experienced in manufacture of precision, bolted, gasketed precast concrete tunnel lining segments of similar dimensions and tolerances and shall have minimum of 10 years' experience on similar projects.

3. Tunnel liner designer shall be licensed civil engineer currently registered in the State of __________ with a minimum of seven years of recent design experience in underground construction and in design of precision bolted, gasketed precast concrete tunnel lining segments.

There are a range of performance requirements for watertightness. If project has single tunnel diameter, it may be more convenient to specify in terms of gpm/1,000 linear feet of tunnel rather than by square feet.

Depending on design responsibility, it may be necessary to specify design assumptions for thrust load, erection loads, etc., and make contractor verify design against actual loads.

4. Manufacturer shall provide training of its personnel for manufacture of segmental lining to cover following:

   a. With the mold manufacturer to ensure proper usage of molds and required tolerance checks

   b. With reinforcement manufacturer to ensure proper fabrication of reinforcement cages and required clearance checks prior to casting

5. Manufacturer shall be certified with NPCA (National Precast Concrete Association) and/or PCI (Precast/Prestressed Concrete Institute) (Group A, Category A1).

B. Segment reinforcement manufacturer qualifications

1. Manufacturer shall have 5 years of experience in manufacture of reinforcement cages for precast segmental linings of at least three projects of similar size.

2. Facility and welders used for welding reinforcing bar cages shall be certified in accordance with AWS D1.4.

C. Segment mold manufacturer qualifications

1. Mold manufacturer shall have 5 years of experience in manufacture of molds for mechanically connected precast segmental linings of similar dimensions and tolerances as those required by this specification.

D. Access for inspections

1. Access shall be provided to segment manufacturers' premises to allow engineer to review quality of manufacturing. Provide all necessary assistance to engineer on each visit.

1.7 PERFORMANCE REQUIREMENTS

A. Watertightness

1. No active drips or seeps above springline of tunnel
2. Maximum inflow at a single point less than 0.1 gpm (0.006 L/s)
3. Tunnel inflow less than 5 gpm per 1,000,000 ft^2 (0.3 L/s per 92,900 m^2) of tunnel lining

PART 2. PRODUCTS

2.1 SEGMENT MOLDS

A. Segment molds shall be steel construction with machined mating surfaces, be of strong and rigid construction, and provide segments to dimensions and tolerances required.

B. Completed molds shall be measured to tolerance that provides standard that is at least half that required for segments.

C. Molds shall provide smooth and true casting surfaces that are free from irregularities or blemishes.

D. Joints in the molds shall be watertight.

E. All inserts to form bolt pockets, holes, grout holes, or similar items shall have sufficient strength and appropriate coefficient of thermal expansion as to maintain construction tolerances of segments.

2.2 CONCRETE MATERIALS

A. Cement

1. ASTM C150, Type II.
2. Non-false-setting and low alkali, containing less than 0.60% alkalis.
3. Requirement for low-alkali cement can be waived if test results provided by supplier show that aggregate source consists of nonalkali reactive aggregate when tested in accordance with ASTM C1293 or C1567.

Material requirements, particularly for concrete constituents, should be reviewed against availability of materials in local area.

Although many projects have used common specifications for concrete for both cast-in-place and segmental linings, it is recommended that the segmental specification has stand-alone concrete specification tailored to needs of precast concrete segments.

Note that most manufacturers Type II cement also satisfies requirements for Type I cement, but the reverse is rarely the case.

Different classes of cement are discussed in Chapters 1 and 2.

B. Aggregates

1. Fine and coarse aggregates
    a. Meet ASTM C33 gradation
    b. Handle as separate ingredients
    c. Shall be nonreactive
    d. Wash before use.
    e. Perform specified tests prior to commencing concrete work.
    f. Variations from specified gradations in individual tests are acceptable if average of three consecutive tests is within specified limits and variation is within permissible variation listed by ASTM C33.
2. Fine aggregate
    a. Hard, dense, durable particles of either sand or crushed stone, regularly graded from coarse to fine, shall meet requirements of ASTM C33.
    b. Fine aggregate content of concrete mixtures shall not exceed 40% by weight of total aggregate content.
    c. Sand equivalent value shall be no less than 75 when tested in accordance with ASTM D2419.
    d. Fine aggregate for each mix design shall be from single source.

The single-source requirement does not prohibit the use of aggregate from more than one location. A separate mix design for each separately sourced aggregate is needed.

3. Coarse aggregate
    a. Hard, dense, and durable gravel or crushed rock free from injurious amounts of soft and friable particles, alkali, organic matter, and other deleterious substances shall meet requirements of ASTM C33 class designation 4S.

ASTM C33 specifies classes of aggregates depending on end use of concrete.

    b. Coarse aggregate shall have maximum size of 1 in. (25 mm).
    c. Before and during field trial mixes, minor adjustments may be made to the aggregate gradation to produce specified concrete mixture properties.
    d. Coarse aggregate for each mix design shall be from single a source.

The single-source requirement does not prohibit the use of aggregate from more than one location. A separate mix design for each separately sourced aggregate is needed.

C. Pozzolan

1. Pozzolan supplied for all mix designs shall be from same single source.

D. Fly ash

1. Class F conforming to ASTM C618

Fly ash or ground granulated blast furnace slag (GGBFS) is commonly used to lower peak heat development and improve long-term strength. Typically one or the other is used but not both.

E. GGBFS

1. GGBFS supplied for all mix designs shall be from same single source.
2. Grade 100 or Grade 120 GGBFS shall conform to ASTM C989.

Type F fly ash is less reactive than Type C and normally more consistent.

F. Silica fume shall conform to ASTM C1240.

G. Admixtures

1. General
   a. Admixture shall be compatible with cement and other admixtures.
   b. Admixtures containing intentionally added chlorides are not acceptable.
   c. Admixtures shall be used in accordance with manufacturer's recommendations and added separately to concrete mix.
2. All chemical admixtures shall meet requirements of ASTM C494/C494M.
3. Air-entraining admixture shall meet requirements of ASTM C260.

H. Water

1. Water shall meet requirements of C1602/C1602M.
2. If reclaimed water is used, ASTM C1602 data shall be provided and control method in place by concrete supplier shall be defined.
3. Potable water may be used without additional testing.

The previous requirement was that mix water had to be potable drinking water, but pressure to reuse wash water has led to development of ASTM C1602. Beware of using wash water without extensive testing as water can change set time characteristics.

2.3 STEEL FIBER REINFORCEMENT

A. Steel fiber reinforcement shall be deformed steel fiber Type I in accordance with ASTM A820. Fibers shall be produced from cold drawn wire.

B. Fibers may be collated with fast-acting water-soluble glue or may be uncollated individual fibers.

C. Fibers shall be stored in dry sealed containers until required for use. Fibers shall be kept free from corrosion, oil, grease, chlorides, and deleterious materials that may reduce the bond between fibers and concrete.

D. Required minimum length of fibers is 2 in. (50 mm).

E. Required minimum aspect ratio of fibers is 50:1.

F. Required minimum tensile strength of fibers is 150,000 psi (1,034 MPa).

G. Fiber type shall be selected on basis of compliance with this specification and on suitability and ease of use in batching, mixing, and concrete placement processes proposed, as demonstrated by trial batches.

H. Fibers that can be uniformly distributed in the concrete and do not tend to form fiber balls during batching and mixing shall be used.

2.4 POLYPROPYLENE FIBERS

A. Liners shall incorporate monofilament polypropylene fibers conforming to ASTM C1116.

B. Liners containing a minimum of 1.75 lb/yd^3 (1.0 kg/m^3) of polypropylene fibers of length no greater than 0.5 in. (12 mm) and diameter no greater than 32 μm shall be deemed to comply with fire-loading requirements.

C. Contractor may propose alternative fiber size and dosage, subject to verification of fiber performance under fireloading as follows:

1. Saturated samples used in proposed concrete mix shall be tested by exposing one face only to a fire-loading equivalent to ASTM E1529 hydrocarbon fire for 2 h.
2. Test pieces shall be subjected to maximum design working stress during test.
3. Area of sample exposed to fire test shall not be less than 1,500 in.^{2} (0.97 m^2) and thickness shall be equivalent to proposed thickness of lining.
4. Test shall demonstrate that spalling of surface exposed to fire load is no greater than 1 in. (25 mm) in any area of specimen.

2.5 CONCRETE MIXES

A. Classification and use

1. Specifier to add text if required.

Some projects may have just one classification for tunnel. Other projects may have different classifications for segments under different loading conditions.

B. Mix proportioning

1. Maximum size aggregate: _________ (typically 1 in. [25 mm] or less)
2. Minimum cementitious content: _______ (typically 475 to 675 lb/yd^3 [282 to 400 kg/m^3])
3. Maximum water/cementitious ratio: ___ (typically 0.35 to 0.40)

If more than one mix design is planned for use in precast concrete segments, type of use, location, and class designation in table shall be specified.

If more than one class of concrete, proportions shall be specified in table.

4. Silica fume: between 5% and 7% of cement, by weight

Silica fume, which improves durability of segments, should be included if required.

5. Percent replacement by pozzolan by weight of cement (typically 15% to 20%)

Using normal procedures, 15% to 20% Type F fly ash mixes can be designed. Content could be increased above 20% if qualifying performance data are provided.

6. Percent replacement by GGBFS by weight of cement (typically 15% to 50%)

   Using normal procedures, 15% to 50% GGBFS mixes can be designed. Content could be increased above 50% if qualifying performance data are provided

7. Air content (0%–3% entrapped for no exposure to freeze–thaw, 4%–6% entrained for freeze–thaw exposure)

   Non-air-entrained concrete has higher early strength for faster form stripping and can be used in areas without freeze–thaw exposure. Concrete usually has entrapped air in the 0%–3% range.

8. Maximum slump specified with or without high-range water-reducing admixture (typically 5 in. [127 mm] without and 8 in. [203 mm] with superplasticizer)

   Slump varies with maximum aggregate size. Self-consolidating concrete mixes can have slumps up to 10 in. (254 mm) and spreads up to 32 in. (813 mm).

9. Sufficient dosage of steel fibers to meet minimum requirements for steel fiber-reinforced concrete (SFRC):

   Delete if SFRC is not used. Specify steel fiber requirements to match design assumptions, in terms of test results from ASTM C1609

   a. Average $f_{150,0.75}$ of four beams sampled from each batch exceeding _____ psi
   b. Any individual $f_{150,0.75}$ not less than _____ psi

   Dosage rates to meet performance requirements will vary between different steel fiber products and will need to be selected based on discussions with fiber suppliers.

10. Minimum 28-d compressive strength ______ psi.

    Concrete mixes for segments are normally at least 6,000 psi (41 MPa).

11. Where specifically approved by engineer, high-range water reducer may be added in concrete mixes.
    a. This increases workability and facilitates concrete placement in areas of high reinforcement congestion.
    b. A maximum slump of 8 in. (203 mm) shall not be exceeded for all concrete containing the high-range water-reducing admixture (superplasticizer) at point of placement.
    c. For all concrete mixes utilizing a high-range water reducer producing concrete mix with slump greater than 5 in. (127 mm), if segregation develops, slump shall be reduced.

2.6 EQUIPMENT

A. Mixing and transportation facilities

1. General
   a. Design and layout shall be reviewed by engineer.
   b. Transport of concrete shall be provided from mixer to point of placement without segregation.
   c. Engineer shall have access to batch plant for necessary inspections.
2. Design
   a. Consistently produces uniform product meeting specifications
   b. Has adequate capacity to meet maximum demand

c. Has backup in case of system failure

d. Is easily inspected for cleanliness and wear

B. Conveying equipment

1. General

a. Material in contact with concrete shall not contaminate concrete (such as aluminum).

b. Discharge facilities shall be provided at end of conveying equipment, which prevent segregation of concrete constituents.

2. Maintenance

a. Maintain equipment in proper operating condition

b. Clean after each placement has been completed.

2.7 REINFORCEMENT STEEL

A. For tied cages, steel bar reinforcement shall be deformed, high-yield bars conforming with ASTM A615, ASTM A82, ASTM A497, or ASTM A496.

For welded cages, steel bar reinforcement shall be deformed, high-yield bars conforming with ASTM A185.

2.8 CURING COMPOUNDS

A. Materials for curing concrete shall meet requirements of ASTM C309, ASTM C1315, or ASTM C171.

ASTM C309 is for liquid curing compounds. ASTM C1315 is for liquid curing and sealing compounds. ASTM C171 is sheet materials for curing.

2.9 SHRINKAGE LIMITATIONS

A. Maximum concrete shrinkage for specimens cast in laboratory from trial batch, as measured at 21-d drying age or at 28-d drying age, shall be 0.036% or 0.042%, respectively.

B. Maximum concrete shrinkage for specimens cast in factory shall not exceed trial batch maximum shrinkage requirement by more than 25%.

2.10 CONCRETE PERMEABILITY

A. Minimum requirement for concrete permeability is maximum coulomb test value of 1,000 coulombs at 56 d, measured in accordance with ASTM C1202.

The coulomb test value should be selected based on approach taken to achieve durability and design life requirements. The 1,000-coulomb value is typical requirement but may be lowered to 750 or 500 under certain conditions; e.g., where concrete must act instead of epoxy coating of rebar.

2.11 TOLERANCES

Actual tolerances should be selected based on exact needs of project.

A. Segment manufacturing tolerances

1. Dimensions of individual precast concrete segments shall be within the following tolerances:

These manufacturing tolerances are written for permanent segmental tunneling gasketed lining to be used behind pressurized face tunnel boring machines (TBMs). In this case, outside surface of segment needs to be relatively smooth and level to avoid causing excessive wear on TBM tail seals and high grease consumption. For open-face TBMs using "junk" segments, a more relaxed specification can be used.

a. Circumferential length: $\pm\frac{1}{16}$ in. (±1.6 mm)

b. Thickness: $\pm\frac{1}{16}$ in. (±1.6 mm)

c. Width: $\pm\frac{1}{32}$ (±0.8 mm)

d. Flatness of sides (circumferential and longitudinal joints): 0.06 in. (1.5 mm) (feeler gauge not passing beneath 36 in. (1 m) long straight edge

Tolerances on circumferential length, and width and flatness of sides are to ensure that joints faces on adjacent segments are aligned and make uniform contact to avoid damage.

e. Width of gasket sealing groove: –0, $+\frac{1}{32}$ in. (–0, +0.8 mm)

f. Depth of gasket sealing groove: $\pm\frac{1}{64}$ in. (±0.4 mm)

Tolerances on width and depth of gasket sealing groove are to ensure that gasket is correctly constrained. These tolerances should be confirmed with gasket manufacturer.

g. Mismatch of gasket sealing groove at corners: $\pm\frac{1}{16}$ in. (±1.6 mm)

h. Warp of segments (distance between opposite corners on intrados): $\pm\frac{1}{32}$ in. (±0.8 mm)

It may be beneficial to use a drawing to illustrate tolerances.

B. Concrete cover

1. Where reinforcing steel is used in segments, minimum clear cover of 1¼ in. (30 mm) shall be provided from outermost surface of embedded reinforcement and closest outer surface of concrete and to all cast-in inserts.

2. In critical or congested zones identified on design drawings, cover of ¾ in. (20 mm) shall be provided.

Concrete cover quoted is in accordance with ACI 318/318M Section 7.7.3 with segments treated as wall elements.

Additional requirements can be included as required by ACI 318/318M Section 7.7.6 for corrosive environments or ACI 350/350M as applicable.

2.12 SEGMENT MARKINGS

A. Catalog of segment markings

1. All segments shall be marked with indented or embossed uppercase lettering on inner face to include the following information:

a. Position or piece number of segment in ring

b. Installation/orientation direction

c. Unique mold identification reference

B. For reinforced segments, cast dimples shall be used to identify areas in segment where no reinforcing steel is located and where holes may be drilled into lining as required.

Additional markings that may be required include internal diameter of ring and segment alignment marks.

2.13 SEGMENT TRACEABILITY

A. Each segment shall be identified on both inside surface and circumferential joint edge using durable UPC bar code label identifiers. Updated data base shall be

As described in Chapter 3, bar code labeling has been shown to be beneficial and allows segment to be traced from start of its life (concrete batch and mold) through to final installation.

maintained at manufacturing plant capable of being accessed and updated throughout production and at storage areas. Capability shall be provided for creating stacks and delivery and loading documents from system. Following data shall be available from bar code:

1. Serial number
2. Segment type
3. Mold number used in manufacture
4. Date and time of casting, including cycle and shift
5. Quality control inspection record, pre- and postmanufacture
6. Cross reference to concrete testing

This requirement is optional and should be considered on a project-by-project basis. If bar codes are not used, it is recommended to require some other method that records the same information.

2.14 GASKETS

A. For gasket requirements, see (model specification)

2.15 INSERTS

A. Bolts and bolt inserts

1. All permanent bolts and washers shall conform to ASTM A325 standard specifications.

In some cases plain steel bolts and washers are sufficient, and in some instances hot-dipped galvanized are needed, depending on exposure conditions. ASTM A123 is the appropriate standard specification for hot dip galvanizing.

B. Grouting sockets

1. All permanent grout/threaded lifting sockets, threaded plastic inserts, grout plugs, and related embedded items shall function under conditions of chemical attack, biological degradation, temperature, packing, and displacements in completed structure and shall account for any long-term relaxation in materials during 100-year design life.
2. Grout plugs shall include a hydrophilic washer.
3. Grout sockets and plugs shall be capable of resisting groundwater pressure without leaking, including any increase in pressure resulting from secondary grouting.

C. Lifting sockets

1. Lifting sockets, where used, shall be subjected to pullout tests. A minimum of two trial tests before production shall be carried out. The test shall demonstrate a safety factor of 3 against failure at design pullout load. The design pullout load shall be equal to the dead load of the largest segment in ring.

D. Concrete spacers

1. Concrete spacers should have the same minimum compressive strength as concrete mix design.

2. All spacers should be saturated with clean water for at least 24 h prior to use.
3. Spacers should be in a moist condition and not appreciably dried out after being fixed to reinforcement cages before concrete is cast.

2.16 STRESS DISTRIBUTION PACKING

A. Stress distribution packing shall be one of following:
   1. Combination of bituminous and elastomeric material with maximum thickness of $\frac{5}{32}$ in. (4 mm)
   2. Marine grade plywood $\frac{5}{32}$ in. (4 mm) thick

Outline specification has not been included for epoxy coating of steel. Epoxy coatings are only likely to be required in extreme circumstances, and other measures to improve durability of concrete are more likely to be effective (such as reduced water/cement ratio and tighter permeability requirements).

PART 3. EXECUTION

3.1 PREPARATION FOR SEGMENT CASTING

A. Thoroughly clean and inspect molds before each use.

B. Coat molds with nonstaining release agent before each use in accordance with manufacturer's recommendations.

C. Fix concrete spacers or chairs so that reinforcement is held firmly in correct position within formwork with all cover as specified. Spacers or chairs shall be rigidly fixed to reinforcement to prevent displacement. Do not use spacers in circumferential or longitudinal joint regions—the areas up to a distance of 0.5 in. (12 mm) from joint surface

D. Ensure that tie wire does not intrude into minimum concrete cover of segments. All wire ends shall be turned inward into segment body.

E. Check that reinforcement cages, plastic inserts, lifting socket, plugs, and other embedments are properly positioned to specified tolerances and clearances with required cover prior to each casting.

3.2 SEGMENT CASTING

A. Segments shall be manufactured under cover in controlled conditions and protected against adverse weather, heat, cold, and humidity.

B. Concrete batching should conform to ASTM C94.

C. Placing of concrete shall conform to applicable requirements of ACI 301, ACI 305.1, and ACI 306.1. Concrete that is found to not conform to these requirements shall be rejected.

D. Concrete shall be consolidated in accordance with ACI 309R for complete contact with molds and embedded items. Consolidate concrete adjacent to side molds and along entire length of molds to ensure a smooth surface finish.

E. Segment extrados surface shall be finished by steel float, with only minimum of surface working consistent with requirement to achieve smooth level uniform surface.

3.3 DEMOLDING

A. Do not remove segment from mold until the concrete has achieved strength of at least 2,000 psi (14 MPa).

Demolding strength should be calculated as part of design and updated as required. Typical value is 2,000 psi (14 MPa), although values as low as 1,500 psi (10 MPa) have been used,

3.4 QUALITY CONTROL

A. Mix design

1. Before beginning concrete work, proper proportions of materials shall be determined for each strength and class of concrete.
2. Mix shall consist of specified cement, pozzolan or GGBFS, admixtures, aggregate, and water.
3. Methods for selecting and adjusting proportions of ingredients shall be in accordance with ACI 211.1.
4. Proportion of all mix designs shall be in accordance with Section 5.3, Proportioning on the Basis of Field Experience and/or Trial Mixtures, of ACI 318/318M.
5. Mix designs on each class of concrete shall be submitted.
6. If trial batches are used, mix design shall be prepared by independent testing laboratory and achieve compressive strength of 1,400 psi (9.5 MPa) higher than specified 28-d strength. If more than 20% pozzolan or 50% slag is utilized for cement replacement, then 90-d strengths shall be used in place of 28-d strengths.

The 90-d test is required to verify that concrete with high GGBFS content does not suffer from dip in strength with time.

7. Proposed mix designs shall be accompanied with complete standard deviation analysis of trial mixture test data, as per ACI 214R.
8. Certified reports of each mix design shall state whether items reported comply with the specifications and shall show:
   a. Design strength and average required strength per ACI 318/318M Section 5.3
   b. Maximum slump
   c. Weights and test results of ingredients

d. Other physical properties necessary to check each mix design

B. Field trial mix

1. After acceptance of mix designs and prior to concrete placement, field proportions for classes of concrete required shall be established, based on accepted design mixes.
2. Field trial concrete shall be manufactured using equipment to be used for work.
3. Adjustments shall be made in design mixes to provide a dense, homogeneous, durable concrete with good workability and finishing qualities.
4. Four sets of three standard test cylinders from the field trial mixes for each concrete design shall be tested in accordance with ASTM C39. Samples shall also be tested for slump (ASTM C143), air content (ASTM C231), density (ASTM C138), and air and ambient temperature (ASTM C1064) by an ACI Level I field technician.
5. Compression strength tests shall be performed for each set of cylinders at 7 d, 14 d, 28 d, and 90 d.
6. Two samples (4 × 4 × 11-in. prisms [100 × 100 × 280 mm]) shall be prepared, cured, dried, and tested for shrinkage tests in accordance with ASTM C157.
7. SFRC
   a. Four 6-in. (152-mm) beams shall be prepared for testing at 28 d to ASTM C1609.
   b. Failed specimens shall be checked to ensure random distribution and alignment of steel fibers. If fiber alignment or distribution is noticeably nonrandom so as to improve testing results, repeat tests.
   c. Fiber content shall be demonstrated by taking random samples of 0.25 ft^3 (7,000 cm^3) of concrete mix and separating out fibers. Frequency of testing shall be one set of three samples per mixing unit per 24 h. Collect, dry, and weigh fibers. Fiber content is deemed acceptable if the following are satisfied:
      1) Average fiber content from set of three samples is greater than design fiber content.
      2) No individual sample has fiber content less than 80% of design fiber content.

8. Engineer shall be notified one week in advance of field trial mix work.
9. Field trial mix work shall be performed in presence of engineer.
10. Production concrete shall not be placed prior to field trial mix requirements being met as specified.

C. Concrete testing during production

1. General
   a. Aggregates shall be sampled at random locations from stockpiles. Samples of concrete shall be taken at predetermined locations and at such times to represent quality of materials and work throughout contract.
   b. Aggregates shall be sampled not less than 30 d prior to use of such aggregates in work to verify gradation, cleanliness, and specific gravities and absorption.
2. Aggregates
   a. Fine and coarse aggregates samples shall be in accordance with ASTM D75.
   b. Samples shall be taken at discharge gates of bins feeding weigh hopper.
   c. Samples shall be obtained at concrete batch plant at frequency specified herein.
   d. Sampling shall be repeated when source of material is changed or when unacceptable deficiencies or variations from specified requirements of materials are found in testing.
   e. Aggregate samples shall be tagged and their sources identified.
3. Coarse aggregate
   a. Samples weighing between 50 and 60 lb (23 to 27 kg) shall be taken after batch plant is brought to full operation.
   b. Samples shall be taken so that uniform cross section, accurately representing materials on belt or in bins, is obtained.
   c. Samples shall be taken for sieve analysis of coarse aggregate and specific gravity and absorption.
4. Fine aggregate
   a. Samples shall be taken as specified for coarse aggregate.

b. Samples shall be taken for sieve analysis of cleanliness, specific gravity, and absorption.

c. Samples of sand shall be taken when sand is moist.

5. Concrete

   a. Samples of plastic concrete shall be obtained in accordance with ASTM C172.

   b. At start of initial concrete pumping placements, testing of concrete samples shall be provided at point of delivery into the form and compared with nearest convenient sample collection point to determine if any significant changes in unit weight, slump, air content, and other mix characteristics occur.

   c. Appropriate adjustments shall be made to concrete mixes delivered to site to correct variations in mix characteristics as specified.

   d. Concrete samples for slump, air content, unit weight, temperature, and test cylinders shall be taken at convenient point of collection after final delivery characteristics of mix have been shown to be acceptable for unit weight, slump, and air content.

6. Testing

   a. Aggregate

      1) Minimum of one test of coarse aggregate per 400 yd^3 (306 m^3) of concrete shall be taken.

      2) Minimum of one test of fine aggregate per 200 yd^3 (153 m^3) of concrete shall be taken.

      3) Continuing conformance with specifications for gradation to ASTM C136, cleanliness, and sand equivalent shall be confirmed.

      4) Minimum of one test per day of each aggregate shall be required.

      5) Aggregate moisture tests shall be made prior to start of batching and at sufficient frequency thereafter to ensure uniform batch-to-batch consistency.

   b. Concrete strength tests

      1) Compressive strength shall be verified by testing standard cylinders of concrete samples taken at job site.

Standard test cylinders are usually 6 in. (152 mm) in diameter × 12 in. (304 mm) long or 4 in. (101 mm) in diameter × 8 in. (203 mm) long. The 6 × 12 cylinders are the old standard, with 4 × 8 cylinders becoming

2) Standard cylinders ___ in. in diameter × ___ in. long shall represent concrete placed.
3) One set of ___ standard cylinders shall be cast per shift of each class of concrete. If concrete mix contains more than 20% pozzolan or 25% slag and will be tested at 90 d, cast ___ standard cylinders.
4) Casting, handling, and curing of cylinders shall be in accordance with ASTM C31 and in line with handling and curing of actual segments.
5) Additional cylinders shall be taken when error in batching is suspected.
6) For the first 24 h after casting, cylinders shall be kept moist in storage box constructed and located so that its interior air temperature will be between 60°F and 80°F (16°C and 27°C).
7) At the end of 24 h, cylinders shall be transported to testing laboratory.
8) Testing of specimens for compressive strength shall be in accordance with ASTM C39.
9) Tests shall be made at 7, 28, and 90 d from time of casting, as appropriate.
10) ___ standard test cylinders from each group of ___ shall be tested at end of 7 d, and ___ cylinders shall be tested at end of 28 d.
11) For concrete mixes with more than 20% pozzolan or 25% GGBFS, the compression testing shall be modified such that ___ standard cylinders from each set of ___ shall be tested at the end of 7 d, ___ cylinders tested at end of 28 d, and ___ cylinders tested at end of 90 d.

more common. Specifications typically require that 6 × 12 concrete cylinders be broken in groups of two while 4 × 8 cylinders are broken in groups of three. Some prefer to break test cylinders in groups of three regardless of size.

Using 6 × 12 cylinders, one would cast a group of four cylinders per shift and six cylinders if 90-d testing is to be performed.

Using 4 × 8 cylinders, one would cast a group of six cylinders per shift and nine cylinders if 90-d testing is to be performed.

When using 6 × 12 test cylinders, cast four. Test two at 7 d and two at 28 d.

When using 4 × 8 test cylinders, cast six. Test three at 7 d and three at 28 d.

When using 6 × 12 test cylinders, cast six. Test two at 7 d, two at 28 d, and two at 90 d.

When using 4 × 8 test cylinders, cast nine. Test three at 7 d, three at 28 d, and three at 90 d.

12) Where compression tests are to be used to determine when segments may be demolded, then at least ___ additional standard cylinders shall be made and cured on-site in accordance with ASTM C31.
13) Additional tests shall be required until a sufficient number of satisfactory tests have

This is not a method that is indicative of in-place concrete and will result in segment demolding times beyond what is necessary. Use this method only when no maturity meter data is available. Delete Paragraph 12 if using maturity meter ASTM C1074. Test-cylinders' strengths will typically lag behind in-place strengths at early ages (less than 72 h). Typical would be to take two 6 × 12 cylinders or three 4 × 8 cylinders.

When using 6 × 12 cylinders, use average of two cylinders.

been performed to establish early strength of concrete in accordance with ACI 214R. Additional testing is waived if strength is estimated using the ASTM C1074 maturity meter method.

14) Each final strength test result shall be the average of the strengths of ___ standard test cylinders at 28 d (or 90 d for concrete with more than 20% pozzolan or 25% slag).

When using 4 × 8 cylinders, use average of three cylinders

15) Average of any three consecutive 28-d or 90-d strength test results, as appropriate, of cylinders representing each class of concrete for each structure shall meet average compressive strength requirements of ACI 318.

16) No individual strength test results shall be less than specified strength by more than 500 psi (3.5 MPa).

17) The 28-d strength test results (or 90-d strength for concrete with more than 20% pozzolan or 25% slag) shall be evaluated in accordance with ACI 214R.

18) If 28-d test (or 90-d strength for concrete with more than 20% pozzolan or 25% slag) results fall below specified compressive strength for class of concrete required for any portion of work, adjustments shall be made in proportions, water content, or both, as necessary. Report changes and adjustments in writing to engineer.

19) If compressive test results indicate concrete segment(s) may not meet structural requirements per concrete class, further tests shall be made to determine if segment can be accepted. Tests may include but not be limited to cores in accordance with ASTM C42 and any other analyses or load tests acceptable to engineer.

c. Tests for consistency of concrete

1) Slump tests shall be performed whenever standard cylinders are cast.

2) Samples shall be taken for slump determination from concrete during placing.

3) Slump shall be as specified when measured in accordance with ASTM C143.

4) Additional tests shall be made at beginning of concrete placement operation and at subsequent intervals to ensure that specification requirements are met.

d. Tests for temperature of concrete

1) Whenever standard cylinders are cast

2) Measured in accordance with ASTM C1064

3) Measured at additional frequency intervals if necessary during hot or cold weather conditions until satisfactory temperature control is established

e. Tests for air content

1) Whenever standard cylinders are cast

2) Measured in accordance with ASTM C231

f. Tests for unit weight and yield of concrete

1) Whenever standard cylinders are cast

2) Measured in accordance with ASTM C138

g. Tests for shrinkage

1) On a monthly basis, two samples (4 × 4 × 11-in. [100 × 100 × 280 mm] prisms) shall be prepared, cured, dried, and tested for shrinkage tests in accordance with ASTM C157.

h. Sampling and testing reports

1) Test reports shall include sufficient information to identify design mix used, stationing or location of concrete placement, and quantity placed.

2) Slump, air content, temperature of concrete, ambient temperature, and unit weight shall be noted.

D. Concrete permeability testing

1. Samples shall be tested in accordance with ASTM C1202.

2. For first 100 rings manufactured, one cylinder shall be tested for each 20 rings cast.

3. After first 100 rings, one test shall be performed for each 150 rings cast, using cylinder samples.

E. Steel fiber testing

1. Two 6-in. (152-mm) beams shall be prepared from each week's production for testing at 28 d to ASTM C1609.

Delete if SFRC is not used.

2. Failed specimens shall be checked to ensure random distribution and alignment of steel fibers. If fiber alignment or distribution is noticeably nonrandom so as to improve testing results, repeat tests.

3. Fiber content shall be demonstrated by taking random samples of 0.25 ft^3 (7,000 cm^3) of concrete mix and separating out fibers. Frequency of testing shall be one set of three samples per mixing unit per 24 h. Collect, dry, and weigh fibers. Fiber content is deemed acceptable if the following are satisfied:

   a. Average fiber content from set of three samples is greater than design fiber content.

   b. No individual sample has fiber content less than 80% of design fiber content.

F. Segment dimensions

1. Labor, equipment, templates, and facilities necessary for inspecting manufactured segments shall be provided, as well as steel profile templates and calibrated measuring instruments for verification of manufacturing tolerances.

Molds can gradually lose required tolerances over time as they are repeatedly reused. A regular cycle of checking is essential to control quality.

2. Tolerance-measuring system shall be implemented to account and adjust for thermal, moisture, and ambient temperature influences.

3. All measurements shall be checked for first segment cast in any mold. Check all measurements to ensure that dimensions and tolerances of segments are maintained. At minimum, check one segment from each mold per week. If any measurement of any segment is found to be out of tolerance:

   a. Segment shall be indelibly marked with the word "DISCARD" and then discarded.

   b. Previous 25 segments produced from that specific mold shall be measured for compliance with required tolerances. If any segments are found to be out of tolerance, indelibly mark segment with the word "DISCARD," and then discard segment.

c. If any measurement of previous 25 segments is found to be out of tolerance, process shall be repeated of discarding defective segment and measuring next 25 previous segments until no defective segments are discovered.

4. Any necessary resources to facilitate independent testing and inspections by owner, including master and working templates, gauges, and calipers adequate to determine accuracy and tolerances in manufacture, shall be provided.

5. Record of all units cast in each mold shall be kept. Withdraw from service any mold that becomes distorted or that casts faulty units until it is proved to be corrected.

3.5 CURING

A. All segments shall be cured using either moist curing, curing compounds, curing at elevated temperatures, or combination of these systems.

B. Segments shall be cured and protected during storage in accordance with ACI 533.

C. If steam curing is used:

1. After segments are cast and have attained preset time, segment forms shall be placed in enclosure or chamber large enough to allow complete circulation of steam.

2. Segments shall not be removed from forms until required stripping strength is attained, as determined by test cylinders.

3. Enclosure or chamber ambient temperature that does not exceed 100°F (38°C) shall be provided for first 2 h of curing; and temperature maintained between 90°F and 150°F (32°C and 65°C) until required stripping strength is attained.

4. Temperatures shall be continuously monitored during curing.

5. Cooling rate shall be controlled to limit temperature differential to avoid thermal cracking.

6. Approved curing compound shall be applied on all surfaces immediately after removal of segments from steam curing.

3.6 TRIAL RINGS

A. Prior to beginning full production of precast concrete segments, a demonstration section shall be erected. The section should comprise two complete precast concrete

Some projects use three rings.

If the segments form an expanded lining, then use a single test ring only segment trial rings, without gaskets or packings but with all other appurtenances including bolts, ball sockets, and alignment rods. If packings are designed to be used at radial joints, gap should be simulated using two spacers of appropriate thickness.

B. Trial rings shall be built on a flat, level base, one above the other and rotationally offset by one or more fixing.

C. The following dimensions shall be checked against corresponding tolerances on each ring of demonstration section:

1. Inside diameter (ID) measured at four evenly spaced locations around ring: 0.2% ID or 0.24 in. (6 mm) maximum.
2. Lip between adjacent segments on ID: 0.06 in. (1.5 mm).
3. Gap between joints with bolts tight and no packing: 0.02 in. (0.5 mm) (feeler gauge not passing).
4. All bolt holes aligned such that bolts can be inserted and fully tightened.

D. If trial rings do not meet tolerances, manufacturing adjustments shall be made as necessary and trial ring assembly repeated. Full production of segments shall commence only after successful completion of demonstration lining trial assembly.

Specified testing frequency varies between different sources. The British Tunnelling Society and Asian specifications recommend assembling a new ring every 0.5% of production. North American specifications reviewed had test frequency between 100 and 500 castings per mold. Other specifications have lower requirements (e.g., one per month for first two months and then at engineer's direction).

E. Base ring of demonstration section shall be retained as master ring for duration of casting.

Refer to separate specification in Appendix C.

F. At intervals not exceeding 1 in every 200 castings from each mold:

1. Ring shall be assembled on top of master ring from segments picked at random.
2. Trial ring dimensions shall be checked to ensure they are within specified tolerances.
3. If trial rings do not meet tolerances, associated molds shall not be used until manufacturing tolerances have been adjusted as necessary and new trial ring assembly meets required tolerances.

3.7 GASKET INSTALLATION

A. For gasket installation, see ___________

3.8 HANDLING/STACKING

A. Segments shall be stacked on level base, supported at quarter points with wood blocking, sleepers, or similar to avoid damage. When stacked, position segments should be in safe and stable manner and with blocking in vertical alignment with horizontal deviation of no more than 6 in. (15 cm).

B. Lifting equipment that minimizes handling stresses on segments by either lifting segments at quarter points or providing continuous lifting force over at least 50% of segment length should be provided. Do not allow chains, wire ropes, etc., to bear against segment surface while lifting.

C. Segments shall not be transported from manufacturing facility until they have attained design strength. This should be verified by cylinder tests on concrete cured with segments or correlation with maturity curves.

D. Segments shall be inspected after delivery to site, and defective or damaged segments repaired or removed in accordance with Figure B1.1. Indelibly mark segments to be rejected with the word “DISCARD” and then discard segment.

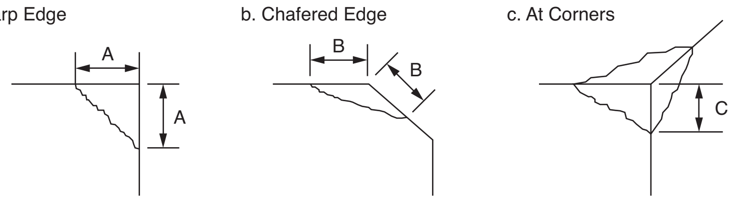
3.9 INSTALLATION

A. Tunnel lining segments, bolts, sockets, compression packing, and gaskets shall be inspected prior to being transported underground and again before assembly. Any element with defects or damage should be removed and repaired, or replaced per approved repair procedures.

B. Segments shall be installed within the following tolerances, and measured after grouting:

1. Surface of leading face of each ring shall not depart at any point from plane surface normal to longitudinal axis of ring, accounting for taper, by more than 0.12 in. (3 mm).
2. Rate of change of plane shall not be greater than 0.12 in. (3 mm) in abutting segments.
3. Maximum and minimum measured IDs in any single ring shall be within 0.5% of design internal diameter, up to maximum of 2 in. (50 mm).
4. Horizontal tunnel alignment tolerance of tunnel centerline shall be ±2 in. (50 mm).
5. Vertical tolerance of tunnel centerline shall be ±2 in. (50 mm).
6. Lips and steps between abutting segments (at circle or radial joints) shall not be greater than 0.2 in. (5 mm).
7. Roll of one ring relative to adjacent ring shall not differ by more than 0.2 in. (5 mm) from relative design positions. Maximum total allowable roll of any ring is ±1.6 in. (41 mm) from design position.
8. Inner gap between adjacent radial faces or circumferential segment faces measured at depth of 2 in. (50 mm) from intrados shall be less than 0.28 in. (7 mm).

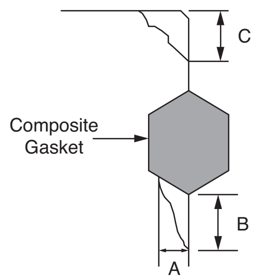
Edge Repair



Guideline for Edge Repair

Action	A, mm	B, mm	C, mm
No repair	< 5	< 5	< 5
Repair with cementitious mortar	5–50	5–50	5–70
Reject	> 50 or rebar exposed	> 50 or rebar exposed	> 70 or rebar exposed

Recesses



Guideline for Recesses Repair

Action	A, mm	B, mm	C, mm (where applicable)
No repair	< 2	< 5	—
Repair with cementitious mortar	2–10	5–30	0–10
Reject	> 10	> 30	> 10

NOTE: All visible blowholes on glued surfaces to be filled with cementitious mortar.

Guideline for Blowholes Repair

Action	Gasket Surface, mm	Epoxy Surface, mm	Intrados, mm
No repair	—	—	< 2 wide
Repair with cementitious mortar	All	All	> 2

Source: Adapted from Singapore Land Transport Authority specification (www.lta.gov.sg).

Figure B1.1 Guidelines for precast segment repair

C. Thrust jacks shall be left locked onto previously built ring until adjacent segment in next ring is placed. Only enough thrust jacks should be removed to allow installation of one segment of ring at any one time.

D. Surface of tail shield, especially invert, should be kept clean and clear of all loose materials, soil, ponding water, or obstructions prior to assembly of each ring. Joint surfaces of segments shall be clean and cleared of deleterious matter before assembly.

E. Longitudinal joints shall be staggered in adjacent rings so there are no cruciform joints.

F. Gaskets shall be checked for cleanliness and position. Lubricate gaskets using soft soap recommended by gasket manufacturer. Apply sufficient load to segment to compress gasket.

G. Bolts shall be tightened as appropriate to maintain contact of joint faces and proper compression of gaskets. Use bolts in all intended positions and in manner designed.

H. Variable-width packing between rings shall not be used to adjust line and grade or negotiate curves. Utilize taper of rings to negotiate curves and grade, and to correct alignment.

I. Temporary supports or hangers for utility lines shall only be connected during construction of tunnel to tunnel lining in manner and at positions that do not detract from strength, durability, or watertightness of tunnel lining. All temporary fixings shall be removed upon completion of construction and repair of lining.

J. Permanent supports, fixings, or hangers for chemical feed lines shall be connected to tunnel lining at positions that do not detract from strength, durability, or watertightness of tunnel lining.

K. Temporary bolts shall only be removed after backfill grout reaches minimum specified compressive strength.

3.10 REPAIR OF DEFECTS

A. Damage identified after demolding at segment factory or after delivery to site shall be repaired in accordance with Figure B1.1, ACI 503.4, and approved repair procedures.

B. If tunnel lining exhibits damage, cracking, spalling, or other irregularities after installation, lining shall be repaired per approved repair procedures.

NOTE: For more information about standards and publications cited here, visit the American Concrete Institute Web site (www.concrete.org), the ASTM International Web site (www.astm.org), and the American Welding Society Web site (www.aws.org).

APPENDIX C

EPDM Gaskets for Precast Concrete Tunnel Lining Specifications

PART 1. GENERAL

1.1 SUMMARY

A. This specification section describes labor, materials, tools, equipment, and incidentals necessary for EPDM (ethylene propylene diene monomer) gaskets for precast concrete tunnel linings, in accordance with contract documents.

B. Related sections

1. Call out section number and title on a project-specific basis.

1.2 REFERENCE STANDARDS

A. American Society for Testing and Materials (ASTM)

1. ASTM D395 *Standard Test Methods for Rubber Property—Compression Set, Method B*
2. ASTM D471 *Standard Test Method for Rubber Property—Effect of Liquids*
3. ASTM D573 *Standard Test Method for Rubber—Deterioration in an Air Oven*
4. ASTM D1149, *Standard Test Methods for Rubber Deterioration-Cracking in an Ozone Controlled Environment*

B. International Organization for Standardization (ISO)

1. ISO 37 *Rubber, Vulcanized or Thermoplastic—Determination of Tensile Stress–Strain Properties*
2. ISO 48 *Rubber, Vulcanized or Thermoplastic—Determination of Hardness (Hardness Between 10 IRHD and 100 IRHD (International Rubber Hardness Degrees))*

C. German Institute for Standardization (DIN)

1. DIN ISO 3302-01 *Rubber—Tolerances for Products—Part 1: Dimensional Tolerances*
2. DIN ISO 3384 *Rubber, Vulcanized or Thermoplastic—Determination of Stress Relaxation in Compression at Ambient and at Elevated Temperatures*

COMMENTARY

This example specification provides detailed requirements for EPDM gasket manufacturing, testing, and use. It was developed by gasket manufacturers to provide robust specifications while avoiding inappropriate or nonstandard requirements.

Depending on project design approach and procurement method, this specification may be used by design team as basis to develop part of the technical specifications. Alternatively, design team might specify only performance-based requirement of leakage criteria and design life.

There are many blank spaces in this specification that must be filled with project specific detail by the specifier.

Related sections will include precast concrete segmental linings specification.

3. DIN ISO 11346 *Rubber, Vulcanized or Thermoplastic—Estimation of Life-time and Maximum Temperature of Use*

D. German Research Association for Underground Transportation Facilities (STUVA)—gasket recommendations

STUVA News' *Tunnel* magazine provides several useful references on the testing and use of gaskets in tunnel linings, including *Tunnel* issue Aug. 8, 2005, and Feb. 2, 2006. See www.tunnel-online.info.

1.3 DEFINITIONS

A. Bearing offset: difference in alignment between two adjacent segments. This is measurement of step between segments and provides measure of reduction of contact surface between adjacent gaskets.

B. Design pressure = testing pressure. It is double the working pressure to provide safety factor and allow for long-term relaxation. This is pressure for which gasket is selected, tested, and supplied.

C. Gasket differential gap: difference between complete gasket compression (zero gap) and actual gasket compression. This is measurement of incomplete closure between two joint faces.

D. Working pressure: expected amount of hydrostatic pressure on structure.

1.4 QUALITY ASSURANCE

A. Manufacturer of EPDM gaskets shall have been regularly engaged in manufacture of such products for tunnels with hydrostatic heads equal to or greater than that of this contract for period of no less than 5 years.

Contract referred to in Section 1.4 is the project to which this specification is applied.

This requirement is intended to ensure that gaskets are supplied by qualified manufacturers. A new manufacturer without the required corporate track record would need to demonstrate that it employs key individuals in technical and production areas with appropriate experience.

This requirement may need to be modified in projects that are at high hydrostatic heads with little or no precedent.

1.5 SUBMITTALS

A. Contractor shall submit the following to the engineer:

1. Details of proposed gasket manufacturer
   a. Manufacturer's name, address, and contact information
   b. Manufacturer's technical contact person
   c. Manufacturer's project reference list
   d. Details of proposed gasket profile, including test data to demonstrate compliance with specified requirements for
      1) Hardness
      2) Tensile strength
      3) Elongation at break
      4) Compression
      5) Ozone resistance
      6) Accelerated aging
      7) Stress relaxation
      8) Water absorption

This information is required to demonstrate that corporate experience matches requirements specified in Section 1.4.

9) Oil absorption
10) Long-term performance
11) Gasket groove loads
12) Required groove design
13) Load-deflection graph
14) Water tightness graph
15) Long-term stress relaxation graph

e. Details of gasket adhesive
f. Details of gasket lubrication, as suggested by manufacturer

2. Manufacturer's recommendations/ quality control manual
   a. Storage procedures
   b. Proper handling procedures
   c. Gasket frame installation
   d. Repair instructions
   e. Special instructions for corners and intersections
   f. Installation equipment (if any)
   g. Health and safety data/material safety data sheets (MSDSs)
3. Shop drawings, including as minimum
   a. Details of each gasket frame with individual gasket marking system

      Acceptable marking systems include color-coding and text.
   b. Details of special joint angles and other construction details

PART 2. PRODUCTS

2.1 MATERIALS

A. Elastomeric gaskets—general

1. Ethylene propylene diene monomer materials are referred to by using the acronym EPDM.
2. Gaskets shall be dense elastomeric synthetic rubber type, free of pitting, porosity, blisters, and other imperfections; manufactured as continuous frame with mitered, moulded gasket corners and vulcanized (no glued corner assemblies permitted) to provide uniform gasket thickness along entire length of mating surfaces.
3. Gaskets shall be manufactured by extrusion to form a profile with appropriate spaces within section to enable gasket to be fully compressible within the groove formed in concrete segments. Gasket shall still be capable of further compression when its top

surface is level with the top of the groove. Percentage of groove filled should not exceed 100% of cross section of groove area at full closure of segments. Compression packing thickness must be considered in groove design process in order to meet specifications previously mentioned.

4. Extruded section shall be joined to form a rectangular gasket frame that is stretch fitted into grooves of the concrete segments. Corner joint shall be designed so the watertightness requirements are achieved without excessive load being concentrated on gasket corners or on gasket groove of concrete segments.

Different manufacturers use different approaches to meet this requirement. One approach is to have corner joint injection moulded and formed from rubber material consisting of lower durometer than extruded profile.

5. Material from which gaskets are to be manufactured shall withstand any chemical attack from the ground or groundwater and shall not be subject to any biological degradation, based on conditions baselined in the geotechnical baseline report (GBR).

Owner is responsible for providing information in GBR about potential contaminants that may impact long-term performance of sealing profile.

If known contaminants are present—e.g., hydrocarbons—then test requirements should be added to Section 2.1B to demonstrate gasket durability.

6. Profile material shall consist of compound based on EPDM able to withstand long-term stresses and strains without detriment to specified performance.

7. Gasket cross section shall be dimensioned to suit the groove as detailed for mating faces of the segmental tunnel linings in accordance with owner's specified construction tolerances. Manufacturing tolerances shall be according to DIN ISO 3302-01, tolerance class E2.

B. EPDM gasket properties

1. EPDM gaskets shall meet the specified minimum requirements included in Table D1.1.

C. Long-term performance

1. Material furnished and supplied under this section shall be guaranteed to perform according to technical provisions and to manufacturer's published specifications, whichever is greater or more stringent.

2. Materials shall be designed for 100-year lifespan, based on the long-term stress relaxation curve determined by testing for 100-year performance

Lifespan should be defined in accordance with project requirements.

2.2 PERFORMANCE

A. Performance requirements specified in this section should be achieved when using the following criteria:

Differential gap of 0.20 in. (5 mm) and bearing offset of 0.40 in. (10 mm) are suggested tolerances for gasket performance. Different tolerances can be specified if required by owner.

1. Actual gasket groove details to be used on segment

TABLE D1.1 Minimum EPDM gasket properties

Value	Standard	Requirement
Elongation at break, min.	ISO 37	300% for hardness class 56-65 200% for hardness class 66-75 175% for hardness class 76-85 Hardness class (IRHD) to be determined using ISO 48
Tensile strength, min.	ISO 37	9 MPa
Deformation: Short term (22 h 158°F [70°C]) Long term (70 h 73°F [23°C])	ASTM D395 Method B	<25% compression (German Research Association for Underground Transportation Facilities [STUVA] value)
Ozone resistance	ASTM D1149	No surface cracking of untensioned specimen under 7× magnification
Accelerated aging: 168 h, 158°F (70°C)	ASTM D573	• Change of hardness less than +6 (IRHD) • Change of tensile strength less than 15% (STUVA value) • Change of elongation less than 30%
Stress relaxation test: Elevated temperature at 158°F (70°C) for 3 months. 60-min measured load set equal to 100%	DIN ISO 3384	Minimum residual load should be better than 55% after 3 months of continuous testing and calculation, according to WLF (Williams–Landel–Ferry) formula determining residual load after lifespan of project (typically 100 years).
Water absorption: 48 h at 158°F (70°C) Use distilled water for standard test.	ASTM D471 Use distilled water for standard test.	Change in weight less than 10%
Oil absorption: 70 h at 158°F (70°C)	ASTM D471	Change in weight less than 110%
Long-term performance	DIN ISO 11346	Determine residual load after lifespan of the project

Source: STUVA recommendation as used on an international basis.

2. Differential gap of 0.20 in. (5 mm) and bearing offset of 0.40 in. (10 mm) along circumferential and radial joints

3. Maximum working pressure: _____ bar

As defined earlier, working pressure is actual maximum hydrostatic pressure and is a project-specific value that needs to be input by the specifier.

B. Gasket groove loads: Demonstrate through laboratory testing (and engineering analysis as required) that the EPDM gasket will not exert a load deflection force of more than _____ per linear meter on gasket groove of concrete tunnel liner. Test should be performed at range of gasket deflections to provide load-deflection force graph. Gasket load should be measured in testing immediately after gasket is compressed.

Insert appropriate force based on discussions with manufacturers using project-specific details.

C. Gaskets shall be tested for watertightness as follows:

1. Gasket-mounting fixture of testing device shall be steel plates machined to proper gasket profile dimensions.
2. Testing device shall be formed as T-joint test rig.
3. Gaskets shall not be glued in.

4. Flat corners shall be verified as sealing correctly.
5. Test rig shall be able to simulate range of conditions of displacement and joint gap, including worst combination to be encountered in completed structure.
6. During test, water pressure should be increased in increments of 1 bar and held at each value for 5 min.
7. Final pressure tested shall be at least the design pressure.

   As previously defined, design pressure is double the working pressure.
8. This pressure shall be maintained for 24 h during which no leakage shall occur at gasketed faces.
9. Tests shall be carried out at normal ambient (surrounding) temperature.
10. Watertightness graph detailing gap and offset variables, based on testing with actual groove configuration, shall be produced based on testing.

PART 3. EXECUTION

3.1 GENERAL

A. Technical assistance: Representative of EPDM manufacturer shall be available once to segment manufacturer's plant, in order to provide installation and testing instructions to contractor's or segment manufacturer's personnel. EPDM manufacturer's representative shall also provide training instructions to contractor's or segment manufacturer's personnel before and during initial startup of EPDM installation.

B. General competence and workmanship: Segment manufacturer shall demonstrate to engineer's satisfaction proven ability and competence to install approved EPDM gasket in accordance with manufacturer's instructions.

3.2 INSTALLATION

A. Surface inspection: Surfaces to receive EPDM gaskets shall be inspected for defects that could result in poor performance of gasket. Defects should be corrected prior to installation of EPDM gasket.

B. Preparation: Surfaces to receive EPDM gaskets shall be cleaned and free of oil, water, laitance, and other deleterious substances in accordance with EPDM gasket manufacturer's instructions.

C. Adhesive: Gasket shall be fixed into the groove cast in segmental tunnel linings prior to erection using only adhesives recommended by EPDM gasket manufacturer.

D. Lubricant: Gasket faces to the key segment radial joints shall be lubricated prior to segment erection with product approved by EPDM gasket manufacturer and by engineer. Lubricant shall be organic soap or approved equal.

E. Mounting of gasket frame to the segment: Mounting shall be in accordance with EPDM gasket manufacturer's suggested instructions.

F. Any gasket frames or compression packing found to be out of compliance shall be repaired or replaced prior to segment installation.

3.3 OUTSIDE STORAGE OF COMPLETED SEGMENTS

A. Length of time in which completed segments with gasket frames attached may be stored in unprotected manner shall be in accordance with gasket manufacturer's requirements.

NOTE: For full references for standards, visit Web sites for ASTM International (www.astm.org), German Institute for Standardization (DIN, www.din.de), and International Organization for Standardization (ISO, www.iso.org).

APPENDIX D

Shotcrete Specifications

This sample specification for shotcrete is based on the three-part system found in ACI 506.2 and ACI 506.5R. The assumptions for this sample specification are a drill-and-blast tunnel where fiber-reinforced shotcrete is the primary ground support. The ground support may include rock bolts, lattice girders, and so forth, but they are not part of this specification section.

PART 1. GENERAL	COMMENTARY
1.1 SUMMARY	
A. This specification includes requirements for furnishing all labor, materials, testing, and equipment, and performing all operations necessary for application of initial support, dry- or wet-process shotcrete to the specified thickness as required in contract documents and specified herein.	Specifier should include whether shotcrete is for initial or final linings, and whether shotcrete is restricted to dry or wet process.
B. This specification is written in inch/pound units.	Specifier should select units.
1.2 DEFINITIONS	
A. Accelerating admixture: admixture that causes increase in rate of hydration of hydraulic cement and thus shortens setting time, increases rate of strength development, or both	Specifier can consult ACI Concrete Terminology (www.concrete.org) ACI 506.2 and ACI 506.5R for definition of terms normally used to describe shotcrete.
B. Dry-mix shotcrete: shotcrete in which most of mixing water is added at nozzle	
C. Ground wire: small-gauge high-strength steel wire used to establish line and grade for shotcrete work	
D. Laitance: layer of weak material derived from cementitious material and aggregate fines either carried by bleeding to surface or to internal cavities of freshly placed mixture; or separated from mixture and deposited on surface or internal cavities during mixture placement.	
E. Nozzle: attachment at end of delivery hose from which shotcrete is projected at high velocity	Note further description of “nozzle” in Chapter 4.
F. Rebound: shotcrete materials or wet shotcrete that bounces away from surface against which shotcrete is being projected	
G. Sagging: subsidence of shotcrete, generally due to excessive water in mixture	

H. Shotcrete: concrete conveyed through hose and pneumatically projected at high velocity onto surface

I. Wet-mix shotcrete: shotcrete in which ingredients, including water, are mixed before introduction into delivery hose

1.3 RELATED SECTIONS

Specifier should add sections as appropriate.

A. 013300—Submittal Procedures

B. 014300—Contractor's Quality Program

C. 033000—Cast-in-Place Concrete

D. 317111—Excavation by Mining Process

E. 317116—Drilling and Blasting

F. 317213—Rock Reinforcement

G. 317220—Lattice Girders

1.4 REFERENCE STANDARDS

A. Latest editions of the following reference standards are referred to in this specification and declared to be part of this specification.

Only standards written in mandatory language should be listed in this section. Guides, reports, and nonmandatory references should be listed in Section 1.4.B.

1. ASTM International (ASTM)

a. ASTM C31/C31M *Standard Practice for Making and Curing Concrete Test Specimens in the Field*

b. ASTM C39/C39M *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*

c. ASTM C94/C94M *Standard Specification for Ready-Mixed Concrete*

d. ASTM C150 *Standard Specification for Portland Cement*

e. ASTM C192/C192M *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory*

f. ASTM C266 *Standard Test Method for Time of Setting of Hydraulic Cement Paste by Gillmore Needles*

g. ASTM C1077 *Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation*

h. ASTM C1116/C1116M *Standard Specification for Fiber-Reinforced Concrete*

i. ASTM C1436 *Standard Specification for Materials for Shotcrete*

j. ASTM C1604/C1604M *Standard Test Method for Obtaining and Testing Drilled Cores of Shotcrete*

k. ASTM C1609/C1609M *Standard Test Method for Flexural Performance of Fiber-Reinforced*

This requirement is only relevant for macro-fiber-reinforced shotcrete.

Concrete (Using Beam With Third-Point Loading)

B. Latest edition of the following guides and reports are listed for reference but are not in mandatory language and are not part of this specification.

1. American Concrete Institute (ACI)

   a. ACI 506R *Guide to Shotcrete*

   b. ACI 506.1R *Guide to Fiber-Reinforced Shotcrete*

   c. ACI 506.5R *Guide for Specifying Underground Shotcrete*

   d. ACI CP-60 *Craftsman Workbook for ACI Certification of Shotcrete Nozzlemen*

C. Regulatory requirements

Specifier should add local as well as OSHA (Occupational Safety and Health Administration) requirements that apply to project.

1.5 QUALITY ASSURANCE

A. General: Shotcrete materials and operations may be tested and inspected by owner as work progresses. Failure to detect defective work or material will not prevent rejection if defect is discovered later nor shall it obligate engineer for final acceptance.

B. Testing agencies: Agencies that test shotcrete materials shall meet requirements of ASTM C1077. Testing agencies shall be accepted by engineer before performing any work. Field tests of shotcrete shall be made by ACI concrete field testing technician Grade I or equivalent.

C. Testing responsibilities of contractor: Unless otherwise specified in contract documents, contractor shall assume duties and responsibilities as listed in this section.

1. Qualifications of shotcrete applicators

   a. Ensure that shotcrete work is performed by personnel experienced in shotcrete application in underground applications.

Alternative could be that “record of proficiency and/or qualification should be available as a record of submittal” and required as a preconstruction submittal from contractor.

   b. Use only nozzlemen who have been qualified according to procedures in this specification and who possess minimum training and experience. In areas with lattice girders, ensure that nozzlemen are additionally qualified for shotcrete application in areas with lattice girders. Minimum experience for nozzlemen shall be 3 years applying plain and fiber-reinforced shotcrete. At least 1 year of experience shall include application of shotcrete onto rock.

Personnel with less than specified experience may be adequately trained when a proper and accepted program is installed and followed.

   c. Qualify each nozzleman by demonstration of acceptable proficiency with uniform application of shotcrete on vertical and overhead test panels and confirm by laboratory testing of cores taken from test panels.

d. Ensure that each qualified nozzleman immediately has available for inspection exact copy of signed test evaluation form.

e. Provide operators qualified to perform work conforming to requirements of this specification. Replace shotcrete crews that are not capable of placing shotcrete that meets requirements of these specifications.

f. Apply shotcrete under immediate supervision of foreman with at least three years of shotcrete operations experience. At least 1 year of experience shall include operations involving application of shotcrete onto rock.

g. Conform to regulatory requirements, including Code of Federal Regulations (CFR) 29-CFR 1910—OSHA, safety requirements for working platforms or lifting equipment, and for personal protective equipment.

2. Mixture proportioning and testing prior to production.

Compatibility testing should be done with materials and representative groundwater conditions expected underground. Acidic groundwater or other chemical exposure may require special cement or changes in mixture.

a. Develop shotcrete mixture by laboratory compatibility tests and field trials as specified herein at least 60 d before actual application of shotcrete.

b. To ascertain compatibility of ingredients and optimum proportions, develop shotcrete mixture with strength and characteristics necessary for actual application.

c. Perform compatibility tests to determine cement and admixtures to be used in field trial mixtures. Determine initial and final setting times for additive concentrations with varying percentages of cement content by mass that are contemplated for use in the work.

d. Make laboratory and field trial mixtures with ingredients identical to those proposed for use in work.

e. Determine initial and final setting times in accordance with ASTM C266 as modified by engineer to accommodate quick-set accelerators. Modify mixtures so that time of initial setting is 3 minutes maximum and final setting is 12 minutes maximum.

3. Standard concrete cylinder testing: Choose materials and proportions so that three 6 × 12 in. (15 × 30 cm) cast cylinders made with no accelerating additives or hydration control admixtures will achieve average minimum compressive strength at 28 d of 5,000 psi (34 MPa). Cast cylinders in accordance with ASTM C192/C192M and test cylinders in accordance with ASTM C39/C39M. Cast and test three cylinders minimum for each combination of materials proposed for use in work.

4. Preconstruction field trials—After completion and acceptance of laboratory tests, make field trials using proposed mixtures, including all additives in dosage proposed, to evaluate capability of equipment, workmanship, and materials under field conditions prior to actual application of shotcrete. Employ same equipment and personnel that will be used in work.

   a. Make field application of each mixture proposed for field trial on at least three overhead and three vertical test panels to simulate actual construction conditions. Shoot test panels into box measuring not less than 3 ft (91 cm) square and with depth of 6 in. (15 cm) or as required by engineer. Test panels shall be kept under in-situ conditions for 9 h after shooting, without disturbance, and covered by polyethylene sheet until time of coring.

Smaller panels may be used in accordance with ASTM C1140 during production. These large panels weigh more than 600 lb (272 kg) when filled.

5. Perform field trial work in presence of contractor's quality control engineer.

   a. Prepare and test nine cores from each test panel in accordance with ASTM C1604/C1604M, except as otherwise specified. Soaking of core specimens before testing is prohibited.

   b. Achieve average core compressive strength of six specimens—three from overhead test panels and three from vertical test panels—at 15-h age from completion of shooting test panel: 1,800 psi (12 MPa) minimum.

It may be impractical to core test panels prior to 20 h, depending on a variety of conditions. If short time early-strength data are required for specific ground conditions, alternative strength development curve testing procedures should be considered.

   c. Achieve average core compressive strength of six specimens—three from overhead test panels and three from vertical test panels—at 24-h age from completion of shooting test panel: 2,700 psi (19 MPa) minimum.

Specifier may want to require 7-d strengths rather than or in addition to 24-h strengths.

d. Achieve average core compressive strength of six specimens—three from overhead test panels and three from vertical test panels—at 28 d from completion of shooting of test panel: 5,000 psi (34 MPa).

e. Ensure that no individual core compressive strength test result at 28 d is below required 28-d strength by more than 500 psi (3 MPa).

f. Prepare and test one 4 × 4 × 14 in. (10 × 10 × 35.5 cm) beam specimens from each test panel shot in Section 1.5.C.4.a in accordance with ASTM C1609/C1609M. Beam specimens shall be sawed on all six sides and shall be kept continuously in lime water at 72°F (22°C) ±3 until testing.

g. Achieve flexural strength and residual strengths at 7 d as follows:

1) Achieve average ultimate flexural strength (modulus of rupture) at 7 d from completion of shooting test panel of 600 psi (4 MPa) minimum.

2) Achieve average residual strength at 0.02 in. (0.5 mm) of 280 psi (2 MPa) at 7 d from completion of shooting of test panel.

D. Uniformity of materials: In applied production, provide same materials in shotcrete mixtures as used in accepted preconstruction trials. Obtain each type or class of cementitious material of same brand from same manufacturer's plant, each aggregate from the same source, and each admixture and fiber from same manufacturer. Minor adjustments shall be permitted subject to prior review and acceptance by engineer. Maintain specified strengths prescribed in this section.

E. Use only calibrated equipment to measure quantity of ingredients during shotcrete production. Maintain exact copy of valid calibration certificate, which shall be attached or otherwise adjacent to equipment in area accessible to engineer for inspection with due consideration of safety.

1.6 SUBMITTALS

A. Submit data sheets for all ingredients proposed for use in shotcrete mixtures, which demonstrate compliance with these specifications.

B. Submit proportions by mass or volume for each shotcrete mixture proposed for use in work; aggregates source, grading and specific gravities; brand and source of portland cement; results of mix design and preconstruction testing.

C. Submit results of testing core specimens from work as specified herein. Test reports shall include

1. Sample identification, mixture number, location, and orientation (shooting vertical or overhead)
2. Date and time of coring
3. Curing conditions and specimen dimensions
4. Date and time of testing
5. Results including ultimate load, strength, correction factors if used, sketch of type of break, and unusual occurrences
6. Name and signature of those conducting the tests

D. Nozzleman qualification procedure:

1. Ensure that work is performed by personnel regularly engaged in shotcrete application in tunnels or trained, tested, and approved by experienced trainer/examiner complying with requirements of approved nozzleman training and qualification program that follows the prescribed curriculum and procedures of ACI- or industry-accepted qualifying procedure.
2. Employ or retain qualified shotcrete examiner/instructor to execute a nozzleman qualification program specific to region and requirements of project.
3. Submit program outline, curriculum, classroom and field evaluation procedures, and schedule and documentation procedure for confirming qualifications of shotcrete nozzlemen.
4. Submit qualifications of program supervisor and administrator.

Project personnel's qualification to apply shotcrete may consist of a combination of past experience, interview, and field evaluation. All combinations should be confirmed and documented by engineer.

Shotcrete for specific project may be hand applied or machine applied. A robotic nozzle operator will generally be more proficient after hand nozzle experience or training.

All nozzlemen should have classroom presentation or review of project conditions and requirements prior to commencing work.

Personnel (miners) employed on project with no previous shotcrete application experience, can be trained in the specific requirements of a project as a practical and acceptable alternative, in the interest of safety and mining practicality.

E. Submit test forms for nozzlemen additionally qualified for shotcrete application in areas with lattice girders. Additional test panels containing representative samples of lattice girders should be shot by nozzlemen who will apply shotcrete in areas that contain lattice girders. Cores of these test panels should be used to evaluate bond to lattice girders and to check for any shooting shadows around lattice girders. The name of tested nozzleman and test results should be clearly shown at top of form.

Panels containing lattice girder components may also be sawn in sections (preferable) to reveal compaction and encapsulation of girder members for assessment of nozzleman competence.

F. Submit qualifications of contractor's quality control engineer and shotcrete foreman.

G. Submit certification that independent testing agency and its personnel meet requirements of this specification.

H. Submit calibration certificates for each piece of equipment used to measure quantities of ingredients during shotcrete production.

1.7 PROJECT CONDITIONS

A. Ensure that nozzleman and shotcrete crew wear personal protective equipment as required by contractor's health and safety plan when applying shotcrete. The protective equipment shall be specifically designed to provide protection from materials included in shotcrete, particularly alkali hydroxides or other chemicals contained in admixtures and their application.

B. Use techniques that minimize level of dust when shooting.

C. Ensure proper delivery, handling, and storage of materials to prevent contamination, segregation, or damage. Store cement in dry place and use within 6 months of manufacture. Store admixtures and fibers in weather-tight enclosures to prevent against dampness, evaporation, freezing, contamination, or other damage.

PART 2. PRODUCTS

2.1 INGREDIENTS USED IN SHOTCRETE SHALL CONFORM TO ASTM C1436.

A. Cement shall be ASTM C150, Type I or Type II.

B. Combined aggregate shall conform to Grading No. 2.

C. Fibers shall be steel or macrosynthetic.

1. Steel fibers, if used, shall be carbon steel, deformed, and capable of meeting performance criteria of these specifications.
2. Macrosynthetic fibers, if used, shall be deformed and shall be capable of meeting performance criteria of these specifications.

ASTM C1436 covers all materials used in shotcrete and refers to underlying ASTM standard(s) covering each material such as C33, C150, C1141/C1141M, C1116/C1116M, and C1602/C1602M. C1436 allows C150, C595, or C1157 cement. If special properties are required, specifier must limit choice of cement. Likewise, if special properties of fibers are required, specifier should limit choice of fibers. Specifier must select type of cements to be used, grading and maximum size aggregate allowed, and fiber material to be used.

2.2 SHOTCRETE MIXTURES

A. Fiber-reinforced, wet-process shotcrete mixtures shall be supplied in accordance with ASTM C1116/C1116M, Type I, for steel fibers or C1116/C1116M, Type III, for macrosynthetic fibers, from plant accepted by engineer.

B. Wet-process shotcrete without fibers, if used, shall be supplied in accordance with ASTM C94/C94M from a plant accepted by engineer.

C. Wet-process shotcrete shall be used within 90 minutes after addition of water to mixture if hydration-control admixtures are not used.

PART 3. EXECUTION

3.1 EXAMINATION OF SUBSTRATE SURFACES

A. Examine ground, concrete, or other substrate surfaces and determine if substrate surfaces have been properly prepared as specified herein.

B. Inspect steel reinforcement and support members and determine if they are properly placed and secured with sufficient clearances provided and that they are free from oil, grease, loose rust, and other coatings that may impair bonding with shotcrete.

3.2 PREPARATION OF SUBSTRATE SURFACES

A. Using pressurized water, remove all loose ground, unacceptable shotcrete, or laitance on previously placed shotcrete and all other foreign matter from substrate surfaces, including previously placed shotcrete.

This example clause is for rock. Removal of loose ground by pressurized water is neither acceptable nor applicable to soft ground tunneling.

B. Take action to control water flowing from substrate and prevent it from adversely affecting shotcrete layer. Any water seepage that may cause deterioration of shotcrete or prevent adhesion shall be diverted by channels, chases, pipes, or other appropriate means.

C. Install thickness pins, ground wires, or other suitable means to establish layer thickness and surface plane. Install pins or wire at spacing accepted by engineer.

3.3 EQUIPMENT

A. All equipment shall be operated, tested, and maintained in accordance with manufacturer's instructions until completion of shotcrete lining work.

B. Shotcrete equipment shall be capable of delivering steady stream of uniformly mixed materials to nozzle and ejecting shotcrete from nozzle at velocities that allow adherence of materials to substrate surface with minimum rebound and maximum adhesion and compaction.

3.4 APPLICATION

A. Provide all labor, materials, testing, supervision, and equipment necessary to perform work as required by contractor's quality work plan and in accordance with these specifications and the contract documents.

B. Ensure that ambient and substrata temperatures are maintained at 40°F (4°C) or higher during shotcreting and for a minimum of 7 d following shotcreting.

C. Operate nozzle in manner such that stream of flowing shotcrete is applied as close as possible at right angles to surface to be covered.

Under normal spraying conditions, nozzle should be perpendicular and approximately 6–8 ft (2–2.5 m) away from substrate, but variables such as amount of reinforcement and access into tunnel heading affect these parameters.

These variables prevent the specifier from mandating spraying distance from substrate. However, unless filling lattice girders, where spraying at an angle to substrate is normal, working at right angles to substrate is desirable.

D. Place shotcrete in layers and maintain steady circular motion with nozzle while building up to thickness required by contract documents. Layer thickness shall not exceed 3 in. (8 cm) unless prior approval for thicker layers is obtained from engineer.

E. Apply shotcrete in manner that produces uniform consistency in order to maximize bonding, adhesion, and density of the shotcrete lining, and to minimize rebound, segregation, and sagging.

F. Ensure that rebound is not reused and is promptly removed from project site.

3.5 CURING

A. Prevent shotcrete from drying out, especially where affected by ventilation air.

B. When required, moisten shotcrete surface or apply curing compounds as specified.

C. Maintain temperature of shotcrete above 40°F (4°C) for 7 d after application.

3.6 FIELD QUALITY CONTROL

A. From in-situ shotcrete lining, obtain 3-in. (8-cm) diameter core test specimens with minimum length of 4 in. (10 cm), three specimens for each 25 yd^3 (19 m^3) of material used in shotcrete lining, at locations specified by engineer.

As described in Chapter 4, an alternative program of nondestructive testing that is calibrated with initial and periodic coring is considered an acceptable alternative and may be specified. This example has used a full coring and cylinder testing program.

As a nondestructive alternate, test panels may be shot in proximity to production work and cores recovered from same. In-situ cores may only be required if related test panel yields an unacceptable result. Clauses 3.6.E through H of this specification section may still apply.

B. Obtain additional core specimens from completed work on the date and at locations specified by engineer. Provide duplicate core specimens from locations specified by engineer upon request.

C. Prepare and test the core specimens in accordance with ASTM C1604/C1604M and as specified herein.

D. Ensure that standard cure core specimens are in accordance with ASTM C1604.

E. After application of first 250 yd^3 (191 m^3) of shotcrete and upon request of contractor, engineer may determine that lower frequency of test specimens is acceptable. Comply with frequency of core testing as established by engineer.

F. Provide additional core specimens upon failure of original specimens. If additional core specimens test results show acceptable strength, the work shall be accepted. If additional results show unacceptable strength, the work shall be rejected. Furnish additional core specimens as directed by engineer at no cost to owner.

G. Ensure that core locations are staggered in random pattern and not arranged in a continuous row. Minimum clearance between core specimens in the work shall be 1 ft (30 cm). Fill core voids caused by coring with nonshrink grout to ensure continuity of lining with respect to strength and appearance.

H. Ensure that shotcrete compressive strength requirements are determined by testing during construction as follows:

1. Average strength of three test results from one area: 2,000 psi (14 MPa) minimum when tested at 24 h after application.
2. Average strength of three test results from one area: 4,000 psi (28 MPa) minimum when tested at 7 d after application.
    a. Minimum strength of single core specimen at 7 d: 3,600 psi (25 MPa).
3. Average strength of three test results from one area: 5,000 psi (34 MPa) minimum at 28 d.
    a. Minimum strength of single core specimen at 28 d: 4,500 psi (31 MPa).

3.7 DEFECTIVE SHOTCRETE

A. Shotcrete that lacks uniformity, watertightness, or adequate bonding; exhibits segregation, honeycombing, lamination, cracking, or drumminess; fails to meet the strength criterion; or otherwise comply with contract documents shall be regarded as defective.

B. Defective shotcrete shall be documented, processed, and disposed of via contractor's nonconformance report according to contractor's quality program.

NOTE: For more information about standards and publications cited here, visit the American Concrete Institute Web site (www.concrete.org) and ASTM International Web site (www.astm.org).

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NOTE: *f.* indicates figure; *n.* indicates note; *t.* indicates table.

CONCRETE FOR UNDERGROUND STRUCTURES

GUIDELINES FOR DESIGN AND CONSTRUCTION

EDITED BY ROBERT J.F. GOODFELLOW

Concrete is a vital component of almost every underground construction project. Because it significantly impacts both the durability and cost of a project, owners, designers, and contractors are constantly challenged with designing and placing the concrete to meet their quality standards in the most cost-effective way.

Concrete for Underground Structures: Guidelines for Design and Construction can make that task a lot easier. Instead of searching pages of scattered reference materials when writing specifications, this book is a one-source guide to help you quickly find the answers you need.

The first resource of its kind, this practical nuts-and-bolts handbook provides an industry voice as well as recommendations for areas of concrete application. You'll get valuable insights into current best practices for all aspects of the design and construction of underground structural concrete.

Internationally respected authors examine three key applications: cast-in-place concrete, precast concrete segmental linings, and shotcrete. Each chapter addresses the differences between aboveground and underground use. The various types of concrete admixtures are also discussed, and sample specifications for each are included.

Concrete for Underground Structures is an indispensable resource for industry veterans as well as an educational tool for those who are new to the profession.

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