

Concrete in Transportation: Desired Performance and Specifications

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This lecture is about concrete—specifically, hydraulic-cement concrete. If one starts with the dry powder that is hydraulic cement—usually the particular class of hydraulic cement known as portland cement—and adds water, what results, depending on the amount of water added, is cement paste or grout. Grout can be poured like gravy. If fine aggregate is added, the result is mortar or sanded grout. If both fine aggregate and coarse aggregate are added, the result is concrete. As the Supreme Court of Pennsylvania once wrote in a decision dealing with cement-manufacturing plants, “cement is to concrete as flour is to fruitcake.” My first point is, to get proper concrete, get the terminology right. There is no such thing as a cement mixer. And sand is not a synonym for fine aggregate; sand is a class of fine aggregate produced by nature rather than by rock crushers and grinding mills.

Having dealt briefly with terminology, I will provide the procedure for obtaining concrete that has the desired performance. It is, simply, to include the relevant requirements in the contract documents and ensure through proper contractor quality control and proper owner quality assurance that the requirements of the specifications are followed. In fewer words, order the concrete that you need and make sure that you get it.

DEVELOPMENT OF HARDENED CONCRETE PROPERTIES

Robert Philleo, who in 1986, as chairman, signed the certificates making Katherine Mather and me honorary members of the Concrete Section of TRB and all of its committees, also in 1986 described the fundamental features of concrete. The following is paraphrased from his papers (1,2). For concrete to stiffen, harden, and develop strength, there must be a chemical reaction between the constituents of the cement and the mixing water. This reaction causes the anhydrous calcium silicates in the cement to be converted into calcium silicate hydrate (CSH), the cement gel or hydrated cement. The critical feature of this reaction is that if the ratio of the volume of water to the volume of cement is 1.2, then all the water and all the cement can combine and all the originally mixing water-filled space can be filled with hydration product. A water-cement ratio (w/c) of 1.2 by volume is 0.4 by mass. If the w/c is higher than 0.4, even if all the cement hydrates, there will always be some residual originally mixing water-filled space that can hold freezable water. If the w/c is lower than 0.4, some of the cement will always remain unhydrated but, in theory, all the originally mixing water-

filled space could be filled with CSH. There is a misconception, often stated, that it takes only the amount of water in a 0.2 w/c paste to hydrate all of the cement. This is based on the fact that only 0.2 unit of water by mass chemically combines with cement during hydration. However, for a given volume of cement to hydrate, there must be an amount of originally mixing water-filled space equal to 1.2 times the volume of the cement. This is because the hydration product has about 30 percent pore space that must be present, and water must be available to fill it. If the amount of originally mixing water-filled space is less than that provided at a w/c of 0.4, not all of the cement can hydrate, even though only half of that water will go into chemical combination.

A lot of modern “high-performance” concrete made at w/c’s well below 0.4 by mass will not, in fact, have all of the originally mixing water-filled space filled with hydration product. This is because the part of the mixing water that ends up in the gel pores undergoes a 10 percent reduction in volume because the pores are so small and the water is adsorbed. Philleo’s 1991 discussion goes into the implications of the fact that additional water may enter while test specimens of very low w/c concrete are curing in the laboratory, but such externally available water will not get very far into larger masses of field concrete even if it is made available. It is worth noting that even though I am a well-known advocate of membrane curing for field concrete, I put in the American Concrete Institute (ACI) standard for curing concrete (ACI 308-81) Section 3.2.2 that the use of liquid membrane-curing compounds should not be approved “when the concrete has a water-cement ratio of 0.4 or less.”

In these comments there may have been the implication that something is desirable about hydrating all the cement in a concrete mixture. If the w/c is higher than 0.4, then the more of the cement that hydrates, the larger the proportion of the originally water-filled space that gets filled, the higher the strength, and the lower the permeability. Therefore, if the concrete needs all the strength and reduced permeability that it can get, then the longer it is kept moist, the closer it will come to having all its cement hydrated and the greater its strength and impermeability will be. There are, however, at least two possibly undesirable consequences of a concrete’s having all of its cement hydrated and becoming as strong as it can become. First, if there is no remaining unhydrated cement, then there can be no autogenous healing of microfractures as water enters and is available to react with unhydrated cement to accomplish the healing. Second, if the concrete is as strong as it can possibly become, it will have a higher modulus of elasticity, be more brittle, and crack at a lower strain level. Reduced strain capacity—both elastic strain

and creep strain—is not a desirable property in a lot of concrete. As Philleo put it,

the desirable magnitude of creep is an issue on which practitioners have not agreed. Structural engineers find it a nuisance they could easily do without. . . . On the other hand, builders of unreinforced mass concrete structures find creep an indispensable property of concrete. . . . Creep redistributes stresses . . . permitting highly stressed regions to shed some of their stresses to low-stressed regions before cracking occurs. . . . [Mass concrete] structures could not survive if concrete behaved elastically. (1)

Those further interested in concrete should read Philleo (1,2).

CLASSES OF DETERIORATION

Very little concrete fails to provide the desired level of performance because its mechanical strength is intrinsically insufficient. Indeed, as noted earlier, one of the undesirable things that can happen to concrete is that, by getting too strong, it can also get too brittle and have an undesirably low strain capacity before fracturing. Consequently, the efforts to achieve the desired performance should be those that avoid the problems generally comprehended under the subject “durability.”

I have spent a great deal of time over a good many years explaining to people that concrete has no property called durability. Any concrete, no matter how unsuitable for use in many environments, will be completely durable in some environment. I have argued that any concrete strong enough to resist the loads to which it will be subjected in service will also be durable in that service, regardless of all other considerations, if it is allowed to get dry and to stay dry. “Dry” in this context means that the evaporable water is allowed to and does escape and the internal relative humidity drops below 80 percent, under which conditions there are no longer any chemical reactions that can take place and there is no water that can freeze. Earthquakes and possibly fire could damage such concrete, of course, but generally speaking, the resistance to earthquakes comes under the heading “if it is strong enough” and the damage due to fire is much lower in concrete that is dry than in concrete that is not.

We may thus explore durability as it relates to concrete in transportation with the assumption that the concrete that we are talking about is being used in an environment in which rarely, if ever, are the structures allowed to get dry and stay dry. However, remember that if we have any such structures, we are in a fortunate position with respect to the steps we need to take to achieve satisfactory performance.

I suggest classifying the causes of deterioration of concrete into two categories. The first category includes those causes of damage in which the cause acts on the concrete, causes the damage, and then ceases to act usually for a very long time or forever. These causes include earthquakes, tornadoes, hurricanes, fire, and lightning strikes. Once a gasoline truck has burned up on a pavement, the damage done to that concrete by that fire is the damage that can be discovered by simply examining the affected structure. Any concrete that has not been damaged by that influence is not going to be damaged by that event subsequently. Therefore, from the standpoint of maintenance and repair, if what has been damaged is re-

paired, that which is undamaged will never be damaged by whatever brought about the need for the repair.

The second, more difficult class of deteriorative influences includes all of the mechanisms that, when recognized as having caused damage to some concrete in service, are of such a nature that the prudent conclusion is that much more of the concrete may be expected to be damaged unless it can be protected from the processes that caused the initial damage. In this class are those concretes that are unable to resist freezing and thawing or chemical attack (especially sulfate attack); those concretes that have been produced of materials that contained the seeds of their own destruction such as aggregates of inadequate volume stability, unsound cement, and unfortunate combinations of alkali-reactive aggregate and high-alkali cement or high-alkali solutions from the environment; and those unprotected reinforcing steel with inadequate cover over the steel and an excess of substances that promote corrosion in the environment. All such damage may be expected to be progressive unless there is a way to stop it. This line of reasoning yields the conclusion that a very important purpose of the investigation of damage, distress, deterioration, and failure is to have a clear appreciation of the causes. This is essential to establishing whether the causes were permitted to work because of a defective specification or the failure to follow a proper one, which is relevant to issues of liability. It is also essential to planning a proper repair or replacement strategy. And, as related to the present topic, it is essential to one’s overall strategy in preparing proper specifications for future work.

The way to obtain satisfactory concrete is to know what to avoid and to take proper steps to avoid it. The art of knowing what to avoid means matching the intended environment of service with similar environments in which concrete has performed less well than desired and knowing what interrelationship of environmental stress and concrete deterioration produced that less-than-desired result. There is enough available knowledge, summarized perhaps best in the 1992 report of ACI Committee 201, that we need not to go back to square one (3). The ACI committee report takes up the problems of durability of concrete in chapters dealing with freezing and thawing; aggressive chemical exposure including sulfate attack, acid attack, and carbonation; abrasion; corrosion of embedded metals and other materials; and chemical reactions of aggregates. The report also includes two more chapters, which address the repair of concrete and the use of protective barrier systems to enhance concrete durability, respectively.

I will talk no further about repair practices, but I will say in passing that although protective barrier systems can function beneficially—especially if by some means one can cause some concrete to become dry and the barrier can be applied in such a manner as to cause the concrete to stay dry—then if the concrete is afflicted with a defect that would make it nondurable in service and it can be caused to become dry and stay dry, it may very well survive for a long time in that environment. However, most concretes that are vulnerable to damage in service have difficulty getting dry, and if they do dry, it is hard to keep them dry by sealing them because one cannot seal all six sides: top, bottom, front, back, left, and right. Some bridge decks appear to be the exception to this rule. In most other concrete structures in transportation, at least the bottom is in contact with moisture.

Let me now comment briefly on the several classes of phenomena potentially harmful to the durability of concrete as they are dealt with in the ACI 201 report.

Freezing and Thawing

I recently summarized in a paper what I thought I understood about the resistance of concrete to freezing and thawing (4). The content of this paper is not in conflict, I think, with what ACI Committee 201 says; however, I approach it from a slightly different point of view. I point out, rather obviously, that concrete will be immune to the effects of freezing for several reasons: (a) if it is not in an environment in which freezing and thawing take place so as to cause water in the concrete to freeze; (b) if, when freezing takes place, there are no pores in the concrete large enough to hold freezable water; (c) if any pores can hold freezable water, they are less than 91 percent full at the time of freezing; or (d) if pores that can hold freezable water are *more* than 91 percent full, the cement paste has a proper air-void system, sound aggregate, and moderate maturity. In the extensive work of Paul Klieger, moderate maturity was discovered to exist if the combination of w/c and cement hydration has proceeded so that the concrete has developed a compressive strength of about 30 MPa (4,000 psi) before it is allowed to freeze and thaw in a critically saturated state. Sound aggregate is the kind of aggregate that when used in concrete containing a proper air-void system in the paste, which is allowed to get moderately mature before freezing, gives frost-resistant concrete as can be measured in the laboratory using ASTM C666 Procedure A. A satisfactory air-void system in the paste is one that is characterized by having an air bubble not more than 0.2 mm (0.008 in.) from anywhere.

As far as I am aware, there is essentially no place in the world today where concrete is needed in transportation structures that is likely to be critically saturated when exposed to freezing and thawing for which sound aggregates cannot be obtained or the concrete be protected against freezing and thawing while critically saturated until it is moderately mature, and for which circumstances preclude using an appropriate amount of proper air-entraining admixture so as to produce concrete with a satisfactory air-void system. Some of us feared that we might be in a situation in which we could not produce concrete with a satisfactory air-void system using roller-compacted concrete, which is of considerable interest especially for paving in military reservations. However, recent work has made it clear that adequately frost-resistant concrete can be made even at the levels of harshness and dryness that characterize concrete suitable for roller compaction (5).

Aggressive Chemical Exposure

Sulfate Attack

The Corps of Engineers' requirements for taking account of sulfate in the environment say that if the level of water-soluble sulfate in soil or dissolved in fresh water expressed as SO_4 is less than 0.1 percent or 150 ppm, respectively, no precautions are necessary (6). If the amount of sulfate is between 0.1 and

0.2 percent in soil or 150 and 1,500 ppm in water, the attack is considered moderate and the appropriate precaution is to use Type II moderately sulfate-resisting cement in which the calculated C_3A content is 8 percent or less. If Type II cement is not economically available, an equivalent degree of sulfate resistance can usually be obtained by using either blended cement made with pozzolan or granulated blast-furnace slag or by adding a proper amount of ground slag or an effective pozzolan such as fly ash or silica fume. Finally, if the sulfate content is higher than the larger values just mentioned, the level of protection should be proportionately greater and will be achieved by using either Type V highly sulfate-resistant cement in which the C_3A content must be kept to 5 percent or less. If such cement is not economically available, a proper amount of an appropriate ground slag or pozzolan can be used.

It is interesting to note that much work has indicated that if the permeability of the concrete is reduced, either by adhering to a lower w/c or by adding ground slag, the amount of C_3A in the cement can be allowed to rise. The Corps of Engineers now permits up to 10 percent calculated C_3A if the w/c of the concrete is kept below 0.45 and the concrete is permanently submerged in seawater.

Another relevant recent development is a performance test that will properly evaluate the sulfate resistance of a cementitious material or blend of such materials. This is ASTM C1012, which involves fabricating mortar bars and storing them in water until they develop a specific degree of maturity and thereafter immersing them in a standard sulfate solution. The sulfate resistance of the cementitious material is regarded as adequate if the expansion of the mortar bars remains below 0.1 percent. This limit has been adopted in the new performance-based ASTM specification for blended hydraulic cements (ASTM C1157).

Acid Attack

Besides sulfate attack, other aggressive chemical exposures involve acid and CO_2 . Concrete in transportation structures is not likely to be exposed to acid very often or very severely. If I were making this talk 10 or 15 years ago, I would warn against the hazards to concrete bridge piers that might result from the establishment of an industry upstream that released strong acids into the water, but clean water laws make this much less likely today. There are cases in which acid from coal mine drainage can cause rivers and streams to have very low pH, in which case it would be better if no transportation structure made out of concrete came into contact with the acid. The Corps of Engineers was asked once to build a dam where the so-called water in the reservoir was said to have a pH as low as 2.0, except after a heavy rain when it went up to 2.4. Nobody suggested making concrete that would be intrinsically resistant to this exposure; instead, we worked on developing an organic coating that would keep the acid from coming into contact with the concrete.

Carbonation

The effects of exposure to concrete of CO_2 in the atmosphere and the resulting carbonation of the concrete appear to be

the current scare phenomenon in concrete technology. People have noticed for a century or more that, after some years of exposure in the first few millimeters below the surface, the cement paste in hardened concrete typically contains no longer calcium hydroxide but calcium carbonate. It has been assumed, correctly, that this represents the reaction of CO_2 from the air with calcium hydroxide in the cement paste to produce calcium carbonate. The carbonation of the cement paste in concrete lowers the amount of calcium hydroxide that is available to replenish the aqueous phase of that concrete with calcium hydroxide in solution to maintain saturation and, hence, a pH of about 12.6—which is, of course, the normal and desirable circumstance from the standpoint of preserving embedded steel from corrosion. Much nonsense has been perpetrated largely by people who oppose the use of ground slag and pozzolans in concrete; they have argued that such use consumes some of the calcium hydroxide that would otherwise be present and is needed to keep up the pH and prevent corrosion of steel. The other argument has been that one ought to use pozzolans for converting the otherwise useless and to some extent undesirable calcium hydroxide—or, as they would call it, free lime—to beneficial CSH with the concomitant reduction in the amount of soluble calcium hydroxide that could go into solution, be carried to the surface through cracks, exit the cracks, and produce ugly efflorescence.

In my judgment, both of these arguments are without practical merit and have little theoretical justification. It would take a great deal of activity to produce an aqueous phase in concrete of even modest quality that was not a saturated solution of calcium hydroxide considering the great excess of calcium in portland cement beyond that needed to make CSH of all the silica in the cement plus all of the available pozzolanic silica at the normal ratios of cement to pozzolan in high-quality structural concrete as should be used in transportation structures. To avoid efflorescence, one should provide appropriate drains so that water does not pass through cracks in walls and bridges and deposit calcium hydroxide that later carbonates.

Abrasion

The third chapter of the ACI 201 report deals with abrasion. This subject is a very important one in transportation because probably more concrete in the world is subject to abrasion in transportation than in any other class of use. When concrete is less resistant to abrasion than desired, sometimes unsafe surface textures develop that create severe hazards because the pavement is nonresistant to skidding.

ASTM has developed several tests to evaluate the degree to which concrete is worn away by rubbing and friction; however, it is important to separate the things that are relevant to one class of structure from those relevant to another. The wearing away of the top surface layer of a very smooth industrial floor under vehicular or other traffic and the consequent production of dust that can harm manufacturing operations is quite different from the wearing away of the texture of a pavement surface. Similarly, the issue of the relative resistance to wear of the mortar portion of the concrete and the coarse aggregate portion becomes very important to skid

resistance. It has been necessary and effective to use a special abrasion-resistant coarse aggregate so that if the surface mortar skin is removed by abrasion, the coarse aggregate will continue to protrude above the base level and provide a skid-resistant surface.

The ACI 201 recommendations suggest that almost all the good practices in concrete production benefit the improvement of abrasion resistance of the surface, especially practices that reduce segregation and the consequent development of a thicker-than-desired layer of mortar at the surface, avoiding bleeding that may cause the surface mortar to have a higher-than-intended w/c and avoiding finishing procedures that bring up more fines or take place at an undesirable time. Obviously, one cannot develop an abrasion-resistant concrete surface unless the concrete at that surface is allowed to develop its potential strength and, hence, abrasion resistance. Thus, a major factor in producing abrasion-resistant concrete is the quality of the curing.

Section 3.6 in the ACI 201 report deals specifically with wear on concrete resulting from tire chains and studded snow tires. It says that studded snow tires have caused widespread and serious damage even to high-quality concrete. It adds that “fortunately the use of studded snow tires has been declining for a number of years,” and it specifically calls attention to *NCHRP Synthesis of Highway Practice 32 (7)*.

Corrosion of Metal

It is my guess that more concrete that has failed to provide satisfactory service for its intended service life because of premature deterioration from interacting with its environment, especially over the last couple of decades, has done so because the precautions taken to prevent corrosion of the reinforcing steel turned out to be inadequate. I have gone on record several times to the effect that all that is needed to prevent corrosion of the steel is to have a cover of high-quality concrete 50 mm (2 in.) thick over the steel and appropriate measures to control concrete cracking. This approach usually works with massive constructions such as bridge piers, dry docks, wharf structures, and off-shore structures. However, this has not been the case with highway bridge decks and superstructures and parking structures. People who design bridge decks, or at least who did in the past, often failed to provide decks thick enough that the required reinforcing steel could be put in place and still have 50 mm (2 in.) of good concrete over it. Thin, strong bridge decks are preferred because they have less deadweight and are more aesthetically pleasing. We need to take extra precautions to protect steel from corrosion if we must live with thin reinforced concrete structures in environments of high potential for materials corrosion; these include highway bridge decks that are deiced with chloride-based deicing chemicals and parking garages in which slush caught under vehicle fenders melts and runs out onto the floor as water with a rather high chloride concentration. We should not, however, in the process give up the effort to make the concrete that we do have concrete of high quality, low w/c, and proper curing. What we do is one or both of two things; some recent experiences have suggested that where the need for protection is great, it is prudent to

do both. These two measures are as follows: (a) use properly prepared, thermally bonded epoxy coatings on the reinforcing steel, and (b) use corrosion-reducing chemical admixtures such as calcium nitrite. I believe that in nearly all cases of serious risk of corrosion, either of these solutions by itself will be completely satisfactory if the potential benefits are realized. However, it has become apparent that there are structures whose owners believed that they were using properly protective epoxy-coated steel when, in fact, the steel put into the structure corroded very quickly. I have not heard recently of an owner who believed that he or she was protected by a chemical admixture and was not, but I am sure I will someday. Then one should consider the use of lightweight concrete with which, for the same deadweight, one can have greater cover over the steel. Alternatively is the option of using up to 30 percent less concrete to achieve a 25 percent reduction in deadweight by using stronger concrete, 60-MPa (8,700-psi) concrete, rather than 35-MPa (5,000-psi) concrete and external prestressing.

There has been much debate on the issue of how much chloride should be in the concrete at the time it is produced. We are aware now, of course, that in addition to the intentional use of calcium chloride as an accelerator to assist in producing early strength in concrete, there is chloride to some degree in other admixtures used for other purposes. Chloride is present in some cementitious materials, and chloride can be present in sea-dredged aggregates, as are used in several places, notably England and Japan, and in certain limestones that are used as concrete aggregate in the United States far from the seacoast. There is also chloride in some mixing water. If one is using epoxy-coated steel or calcium nitrite, it should not matter how much chloride is in the concrete as made. If neither of these precautions is taken, it might matter a lot.

The question of whether it is ever prudent to assume that none of the measures that I have mentioned will be satisfactory and to install cathodic protection for the steel at the time the structure is built is, I think, still undecided. This may be wise in some cases, but I expect that they are few and far between.

Chemical Reactions of Aggregates

Tom Stanton, of the California Department of Transportation (although they didn't call it that then), discovered alkali-silica reaction in 1940 (8,p.54ff). I went to work for the Concrete Laboratory at the Corps of Engineers in August 1941. Word of his discovery had already reached that laboratory, and one of the first bits of work I did was looking at "siliceous magnesian limestone," which was described as the offending ingredient of concrete that had deteriorated. At that point the Corps of Engineers was quite concerned because, to us, magnesian limestone meant dolomite, and dolomite was very widely used in Corps of Engineers structures. Our position had been that *most* limestones were satisfactory but substantially *all* dolomites were. It turned out that the siliceous magnesian limestone that caused the trouble in California is an almost unique rock that contains a lot of opaline chert. As far as I know, no other dolomite in the world contains enough opaline chert to cause trouble.

The Ninth International Conference on Alkali-Aggregate Reaction on Concrete took place in July 1992 in London in the Queen Elizabeth II Conference Center across the street from Westminster Abbey; several hundred people attended from essentially everywhere in the world. Aggregates that were not previously regarded as reactive are now shown to be reactive when used with cements of higher alkali content than they had been used with. Cements in many parts of the world are now of higher alkali content than they used to be, partly because more of them are made using raw materials containing relatively large amounts of alkali (such as, for example, all of the cement in Iceland for which all of the calcareous raw material is seashells dredged from the ocean) and also partly because of the capture of kiln dust and its insertion into the product. With regard to this last point, I am told that some farmers now must buy substantial amounts of potash fertilizer because cement plants upwind are catching the kiln dust that used to go out their stacks and putting it back into the cement, to the detriment of both the cement and the farmer.

Alkali-silica reaction involves a series of events. A reactive aggregate is one that contains silica, SiO_2 , in a form that is capable of being dissolved by pH solutions substantially higher than the 12.6 pH that characterizes the saturated calcium hydroxide solution that is normal in concrete. Either the cement contains enough sodium or potassium so that the pH is raised when the cement hydrates and these ions go into solution, or the alkalies come in from the outside and raise the pH. In any event, the pore fluid in the concrete gets to be high enough in pH to dissolve the silica and produce an alkali-silica gel that has the property of taking up water, swelling, and expanding and disrupting the concrete.

If the circumstances are such that any problems that might be brought about by alkali from the environment can be ignored, then, in many cases, all that is necessary is to invoke the specification option for cement and require the use of low-alkali cement when reactive aggregate must be used. This requirement means that the cement will not be allowed to contain more than 0.60 percent alkalies calculated as the percentage Na_2O plus 0.658 times the percentage K_2O . If low-alkali cement is not available, equivalent protection can usually be provided by using an appropriate amount of an acceptable ground slag or pozzolan, the acceptability being based on tests by ASTM C441.

There is also an alkali-carbonate rock reaction that has been found in a number of highway structures, especially some in Illinois, Indiana, Iowa, Michigan, Missouri, New York, South Dakota, Tennessee, Virginia, and Wisconsin. Not all such reactions are deleterious. Sometimes the reaction simply takes place, and a small portion inward from the outer surface of an aggregate particle is altered so that when one slices through the concrete and acid-etches the sawed surface, the aggregate particle develops two levels of etching. Sometimes the interior is more acid-soluble than the rim, and sometimes it is less acid-soluble than the rim; in either case the fact that the rim is on crushed stone particle is evidence that a reaction took place in the concrete.

To avoid harmful alkali-carbonate-rock reactions, use non-reactive aggregate or figure out a combination with low-alkali cement. The ACI committee report recommends a value less than 0.60 percent Na_2O equivalent.

CONCLUSION

To obtain the performance that is desired of concrete in transportation, it is necessary to

1. Decide what sorts and levels of imperfections are tolerable and intolerable,
2. Understand what causes imperfections that are intolerable,
3. Evaluate the environment in which the concrete is to serve to recognize the presence of influences that must be resisted if the concrete is to perform as desired,
4. Prepare concrete specifications that require appropriate levels of relevant properties of concrete so that the concrete can resist the deteriorative influences it will encounter in service, and
5. Ascertain that both the contractor's quality control and the owner's quality assurance systems work to ensure that the concrete produced is as specified.

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