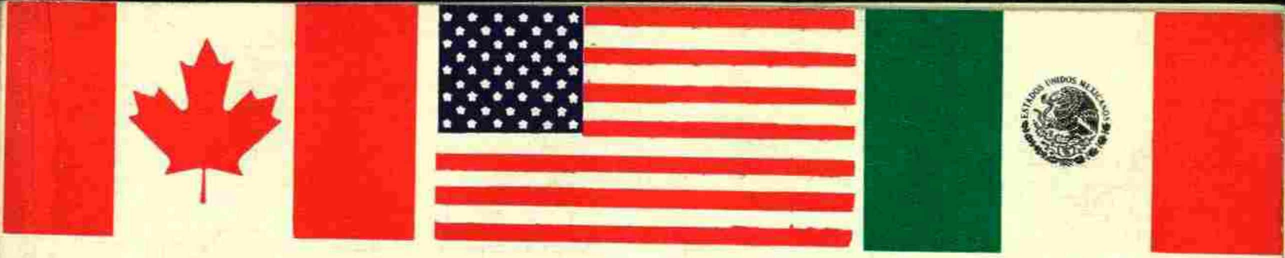

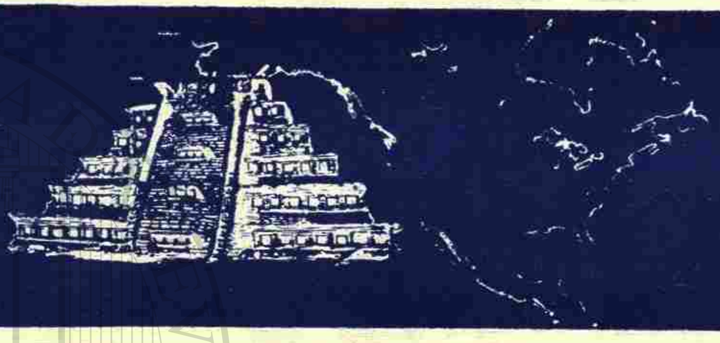

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DURABILIDAD DEL CONCRETO

CONCRETE DURABILITY

RAYMUNDO RIVERA VILLARREAL EDITOR

DURABILIDAD DEL CONCRETO

MEMORIA

Memorias del Seminario Internacional
sobre Tecnología del Concreto:
Durabilidad

Memoria del Seminario Internacional sobre Tecnología del Concreto: Durabilidad

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DURABILIDAD DEL CONCRETO

Memoria del Seminario Internacional sobre Tecnología
del Concreto organizado por la Universidad Autónoma
de Nuevo León.

EDITADO POR
RAYMUNDO RIVERA VILLARREAL

TRADUCCION
ILIANA M. GARZA GUTIERREZ

MONTERREY, N.L. MEXICO

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Facultad de Ingeniería Civil
Universidad Autónoma de Nuevo León
Apdo. Postal No. 17, San Nicolás de los Garza,
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CONACYT

PREFACIO

La construcción en todo el mundo enfrenta problemas de todo tipo, tales como la crisis energética, protección del medio ambiente, recursos naturales limitados y la necesidad de lograr un uso más eficiente y seguro de los materiales. En el conocimiento de los materiales para un uso eficiente y seguro, se contempla la modernización de la respuesta de los materiales para predecir su vida de servicio bajo diversas condiciones ambientales. La solución de estos problemas ha propiciado y estimulado la investigación científica y tecnológica a nivel internacional, produciéndose entre la comunidad científica mundial una transferencia de conocimientos y de técnicas que se ha agilizado con el avance de las ciencias computacionales aplicadas a la comunicación.

Una rápida transferencia internacional de tecnología se logra a través de conferencias y simposios sobre temas específicos y la publicación de las memorias respectivas. Aunque existen organizaciones internacionales que apoyan estos eventos como son la Réunion Internationale des Laboratoires d'Essais et de Recherches sur les Matériaux et les Constructions (RILEM) y el American Concrete Institute (ACI), cabe destacar la labor personal del Dr. Mohan Malhotra, Director del Programa sobre Tecnología Avanzada del Canada Center for Mineral and Energy Technology (CANMET), quien desde principios de la década de los 70s, ha organizado sistemáticamente, en diversos países, conferencias internacionales y publicado libros sobre el basto campo de la Tecnología del Concreto, incluyendo el tema de la Durabilidad. El Dr. Malhotra también ha apoyado eventos sobre tecnología del concreto efectuados en diversas ciudades de Latinoamérica y en especial ha apoyado a los simposios organizados por la Facultad de Ingeniería Civil de la Universidad Autónoma de Nuevo León (UANL), lo cual agradecemos en lo que vale. En este Seminario, la UANL ofreció un reconocimiento al Dr. Malhotra, por su gran empeño en estimular y diseminar la investigación enfocada a producir concretos durables, en Norteamérica y en todo el mundo.

La primer Conferencia Internacional sobre "Durabilidad de los Materiales de Construcción y sus Componentes" se celebró en la ciudad de Ottawa, Ontario, Canadá, del 21 al 23 de agosto de 1987, siendo patrocinada por el National Research Council (NRC) de Canadá y participando además la American Society for Testing and Materials (ASTM), RILEM y el National Bureau of Standards (NBS) de Estados Unidos. Las Memorias fueron publicadas por ASTM en su volumen STP 621.

La primer Conferencia Internacional orientada específicamente a la Durabilidad del Concreto, fue patrocinada por el U.S. Army Corps of Engineers y otras organizaciones de Canadá y Estados Unidos, reconocidas en el campo de los materiales de construcción. Se celebró en la ciudad de Atlanta, Georgia, del 27 de abril al 1 de mayo de 1987, actuando como presidente organizador el Sr. Scanlon. Esta conferencia llevó el nombre de KATHERINE AND BRYANT MATHER en homenaje a sus infatigables labores de muchos años en la investigación de los efectos en los materiales que forman el concreto, el concreto mismo y las técnicas constructivas en su habilidad para permanecer durables bajo las diversas condiciones de exposición agresivas del medio ambiente.

El Sr. Bryant Mather, como Director del Laboratorio de Estructuras en el Waterways Experiment Station del Cuerpo de Ingenieros de la Ejército de los Estados Unidos, en Vicksburg, MS., participó en el Seminario representando a esta organización. El Sr. Mather recibió un reconocimiento de la UANL por sus esfuerzos en promover el interés y la investigación sobre la Durabilidad del Concreto.

La segunda Conferencia Internacional sobre Durabilidad del Concreto se realizó en Montreal, Canadá del 4 al 19 de agosto de 1991 y fue organizada por CANMET y el ACI, con la participación de otras organizaciones internacionales de prestigio en el campo de la Tecnología del Concreto.

Con motivo de la celebración del 60° Aniversario de la fundación de la Universidad Autónoma de Nuevo león y de su Facultad de Ingeniería Civil, el Departamento de Investigación en Tecnología del Concreto del Instituto de Ingeniería Civil, dependientes de la misma Facultad y a la vez avizorando el próximo Tratado de Libre Comercio entre Canadá, Estados Unidos y México, decidió organizar este Seminario Internacional sobre Durabilidad del Concreto, considerándolo como prioritario.

La Tecnología del Concreto, en su avance vertiginoso ha entregado numerosas y diversas aportaciones a la construcción. En la actualidad es apremiante la consideración de la durabilidad, por los efectos destructivos de los diversos factores ambientales, con las consiguientes pérdidas económicas resultantes de una escasa durabilidad del concreto. La Tecnología del Concreto está desarrollándose en este campo eficazmente ante una complejidad de factores y complicaciones provenientes del uso de aditivos y suplementos cementantes.

El Seminario sirvió además de difundir conocimientos actuales, como un foro para discutir e identificar nuevos problemas y detectar qué mejoras requiere la práctica actual para alcanzar una mayor durabilidad de las construcciones de concreto. Esperando que haya influido en los diseñadores para que en sus especificaciones, las consideraciones sobre durabilidad serán aprobadas por aquellas personas que tienen el poder de decisión.

La Universidad Autónoma de Nuevo León agradece el copatrocinio del CANMET y de Cuerpo de Ingenieros de la Ejército de los Estados Unidos, sin los cuales no hubiera sid posible la organización de este Seminario.

En el Seminario se logró conjuntar a algunos de los especialistas más destacados de Norteamérica, para presentar los últimos avances en casi todos los temas involucrados co la durabilidad del concreto. La mayoría de los ponentes son investigadores que ha participado en conferencias internacionales sobre Durabilidad previas a este evento y quienes agradecemos cumplidamente su desinterés y apoyo para llevar a cabo est Seminario.

La Universidad Autónoma de Nuevo León agradece, además, la cooperación de las instituciones de procedencia de los conferencistas por el apoyo ofrecido a estos para participar en el evento.

Además la UANL desea expresar su reconocimiento al Consejo Nacional de Ciencia y Tecnología (CONACYT) por su valiosa aportación económica para la publicación de esta Memoria.

Ante todo deseo expresar mis gracias a la Lic. Iliana M. Garza Gutiérrez, por su esfuerzo en revisar la tipografía y traducciones necesarias de los trabajos presentados y el eficiente manejo de todos los aspectos administrativos relacionados con la organización del evento.

PREFACE

All over the world, construction faces any kind of problems, such as energetic crisis, environmental preservation, limited natural resources and the necessity to get an efficient and safety use of materials. In order to accomplish this objective, it is important to modernize the way to predicting materials expectancy of life under different environmental conditions. Solution of this problems has propiciate and stimulated scientific and technological research all over the world, producing among scientific community a transfer of knowledge and techniques which has been enable with the development of the computer sciences applied to communication.

International fast transfer of technology is achieved through conferences and symposia about specific subjects and the corresponding proceedings publication. Although, there exist international organizations which support that kind of events, as the Réunion Internationale des Laboratoires d'Essais et de Recherches sur les Matériaux et les Constructions (RILEM) and the American Concrete Institute (ACI), it is important to mention the individual effort of Dr. V. Mohan Malhotra, Program Principal-Advanced Concrete Technology Program, from The Canada Center for Mineral and Energy Technology (CANMET), who, since the 70's decade, has organized, systematically, in different countries, international conferences and published books about the vast field of Concrete Technology, including Durability subject. Dr. Malhotra also has supported other events about concrete technology carried out in different Latin American cities, and has specially supported some symposia organized by the Facultad de Ingeniería Civil of the Universidad Autónoma de Nuevo León (UANL), which we greatly thank him. In this Seminar the UANL had offered a recognition to Dr. Malhotra, because of his effort in stimulating and disseminating research focused in producing durable concrete, in North America and all over the world.

The First International Conference on "Durability of Construction Materials and its Components" was held in Ottawa, Ontario, Canada, from August 21 to 23, 1987, and was sponsored by the National Research Council (NRC) from Canada and there also participate the American Society for Testing and Materials (ASTM), RILEM and the National Bureau of Standards (NBS) from the United States. The Proceedings was published by ASTM in its Vol. STP 621.

The First International Conference focused on Concrete Durability, was sponsored by the U.S. Army Corps of Engineers and other recognized construction materials organizations from Canada and the United States. It was held in Atlanta, Georgia, from April 27 to May 1 1987, and its chairman was Mr. John Scanlon. This conference was named KATHERINE AND BRYANT MATHER honoring their tireless work researching on the effects of concrete materials, concrete itself and constructive techniques on its ability to remain durable under different environmental aggressive conditions.

Mr. Bryant Mather, as Structures Laboratory Director of Waterways Experiment Station of the U.S. Army Corps of Engineers, Vicksburg, MS, was representing this organization in the Seminar. Mr. Mather received an award from UANL, because of his efforts in promoting interest and research on concrete durability.

The Second International Seminar on Concrete Durability was held in Montreal, Canada, from August 4 to 19, 1991, and it was organized by CANMET, ACI as well as other very important institutions involved in concrete technology.

Due to the 60 th Anniversary of the Universidad Autónoma de Nuevo León as well as its Facultad de Ingeniería Civil, its Department of Concrete Research, taking into account the coming North America Free Trade Market, decided to organize this International Seminar on Concrete Durability, due to it is considered a very important topic.

Concrete technology, in its fast development has contributed with vast and different findings. Nowadays durability is of great importance, because of the different environmental destructive effects, with the well known economic loses due to a poor concrete durability. Concrete technology is being efficiently developed against a great variety of factors and complications arising from the use of admixtures and cementitious supplements.

The Seminar, served to diffuse knew knowledge as well as a place to discus and identify new problems and which current practices need to be improved, served to achieve an assured durability of concrete buildings.

The Universidad Autónoma de Nuevo León thanks CANMET and U.S. Corps of Engineers their invaluable cosponsorship, without which it could not be possible this event.

The Seminar joint some of the most important specialists in North America, in order to present the State-of-the Art in almost all the topics involved in concrete durability. Most of the speakers have participated on international conferences on Durability previous to this event, and we greatly thank their altruistic support to carry out this Seminar.

The UANL thanks the speakers' universities and/or organizations, due to their support offered to them in order to participate.

The UANL also wants to express its gratitude to Consejo Nacional de Ciencia y Tecnología (CONACYT) because of its valuable financial support for the publication of the Proceedings of this International Seminar on Concrete Durability.

Above all, I want to thank to Miss Iliana M. Garza Gutiérrez, because of her effort in formatting all the papers and necessary translations of them, as well as for her efficient managing of all administrative aspects related to the organization of this Seminar.

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COMO OBTENER CONCRETO DURABLE*

por

Bryant Mather

RESUMEN

El concreto se comportará tal como se desee en los edificios, puentes, compuerta diques, pavimentos para campos aéreas, carreteras, estructuras para estacionamiento embarcaderos, rompeolas, revestimientos, etc., siempre y cuando tenga los nivele apropiados de las propiedades relevantes. El concreto tendrá estos niveles si l especificaciones bajo las cuales es elaborado son las correctas y si se siguen al pie de letra. Esto podrá llevarse a cabo si el control de la calidad por parte del contratista y confirmación de la calidad por el propietario, funcionan como deben. Ellas serán l correctas si cubren los niveles necesarios de las propiedades de los materiales, l proporcionamientos requeridos para las mezclas y las prácticas de construcción empleada para llevar a cabo el trabajo. Los niveles de las propiedades de los materiales, l proporcionamientos de las mezclas y las prácticas de construcción serán correctament seleccionados y especificados si se posee el conocimiento adecuado y se usa pa relacionar el comportamiento del concreto con los factores relevantes del medio ambient de servicio, para la vida útil que se pretenda. Al seleccionar estas características se deb tomar en cuenta el cambio tolerable y el intolerable en el concreto en una estructu particular en un lugar de servicio dado. En resumen, si el concreto no se comporta com se desea, es porque las especificaciones no eran las correctas o porque no se siguiero adecuadamente: porque no se ordenó lo que se debía o porque lo que se ordenó no se llev a cabo.

Este trabajo es acerca del concreto, específicamente del concreto fabricado co cemento hidráulico. Si se empieza con el polvo seco, que es el cemento hidráulico; menudo, de la clase particular de cemento hidráulico conocido como Portland; y se añad agua, da por resultado, dependiendo de la cantidad de agua añadida, una pasta de cement o lechada; esta última puede ser usada como salsa. Si se añade agregado fino, resultado es un mortero o lechada arenosa. Si se añaden agregado fino y grueso resultado es concreto. Como escribió alguna vez la Suprema Corte de Pensilvania sob una decisión relacionada con las plantas manufactureras de cemento," el cemento es par el concreto como la harina al pastel de frutas". Mi primer propósito es, obtener el concret apropiado y la terminología correcta. No hay nada mejor como la mezcla de cement Además, arena no es un sinónimo de agregado fino, ésta es una clase de agregado fin producido por la naturaleza y no por trituradores o molinos de roca.

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* Para el Seminario Internacional, Monterrey, N.L., México. 5-8 Octubre 1993.

HOW TO OBTAIN DURABLE CONCRETE*

by

Bryant Mather

ABSTRACT

Concrete will perform as desired in buildings, bridges, locks, dams, airfield pavements, highways, parking structures, wharves, piers, revetments, and so on-if it has the appropriate levels of relevant properties. It will have such levels if the specifications under which it was produced are proper and if they are followed. They will be followed if the quality control by the contractor and the quality assurance by the owner function as they should. They will be proper if they cover the levels of the materials properties needed, the proportions required for the mixtures, and the construction practices used in carrying out the work. The levels of the materials properties, the mixture proportions, and the construction practices will all be properly selected-and specified-if adequate knowledge is available and is used to relate performance of concrete to the relevant factors in the environment of service for the intended service life. In making these selections, due account must be taken of what is tolerable and intolerable change in the concrete in a particular structure in a particular location of service. In short, if concrete fails to perform as desired, it is because the specifications either were wrong or were not followed: either what should have been ordered was not, or what was ordered was not delivered.

This paper 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 now provide the procedure for obtaining concrete that has the desired performance. It is, simply, to include the relevant

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* For International Seminar, Monterrey, N.L., Mexico. 5-8 October 1993.

requirements in the contract documents and ensure through proper contractor qualit control and proper owner quality assurance that the requirements of the specifications ar followed. In fewer words, order the concrete that you need and make sure that you ge it.

DEVELOPMENT OF HARDENED CONCRETE PROPERTIES

Bob Philleo, in 1986, described the fundamental features of concrete. The followin is paraphrased from his papers (1, 2). For concrete to stiffen, harden, and develo strength, there must be a chemical reaction between the constituents of the cement an the mixing water. This reaction causes the anhydrous calcium silicates in the cement t be converted into calcium silicate hydrate (CSH), the cement gel or hydrated cement. Th critical feature of this reaction is that if the ratio of the volume of water to the volume c cement is 1.2, then all the water and all the cement can combine and all the original mixing water-filled space can be filled with hydration product. A water-cement ratio (w/ of 1.2 by volume is 0.4 by mass. If the w/c is higher than 0.4, even if all the ceme hydrates, there will always be some residual originally mixing water-filled space that ca hold freezable water. If the w/c is lower than 0.4, then some of the cement will alwa remain unhydrated but, in theory, all the originally mixing water-filled space could be fill with hydration products. There is a misconception, often stated, that it only takes th amount of water in an 0.2 w/c paste to hydrate all of the cement. This is based on th fact that only 0.2 units of water by mass chemically combine with cement durir hydration. However, for a given volume of cement to hydrate there must be an amou of originally mixing water-filled space equal to 1.2 times the volume of the cement. Th is because the hydration product has about 30 percent pore space that must be prese and water must be available to fill it. If the amount of originally mixing water-filled spac is less than that provided at a w/c of 0.4, not all of the cement can hydrate, even thoug 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 ma will not, in fact, have all of the originally mixing water-filled space filled with hydrati product. This is because the part of the mixing water that ends up in the gel por undergoes a 10 percent reduction in volume because the pores are so small and the wat is adsorbed. Philleo's 1991 discussion goes into the implications of the fact that additio water may enter while test specimens of very low w/c concrete are curing in th laboratory but such externally available water will not get very far into larger masses field concrete even if it is made available. It is worth noting that even though I am a we known as an advocate of membrane curing for field concrete, I put in the Americ Concrete Institute (ACI) standard for curing concrete (ACI 308-81) Section 3.2.2 that th use of liquid membrane-curing compounds should not be approved "when the concrete h a water-cement ratio of 0.4 or less."

In these comments there may have been the implication that something is desirab about hydrating all the cement in a concrete mixture. If the w/c is higher than 0.4, th the more of the cement that hydrates, the larger the proportion of the originally water-fill

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 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)

I was tempted to paraphrase more of this story but I refrain. I suggest that those further interested in concrete 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 that is 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 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 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, we should 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 lightening 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 repaired, 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 concretes whose structures contain 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 in 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 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 up 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 sufficient available knowledge, summarized perhaps best in the 1992 report of ACI Committee 201 (3), that we need not go back to square one. 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 would 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 difficult 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 at least the bottom 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 what I thought I understood about the resistance of concrete to freezing and thawing in a paper entitled How to Make Concrete that Will be Immune to the Effects of 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 is not in an environment in which freezing and thawing takes 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 that can hold freezable water, they are less than 91 percent filled 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 C 666 Procedure A. A satisfactory air-void system in the paste is one that is characterized by having an air bubble located 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 that is likely to be critically saturated when exposed to freezing and thawing for which sound aggregates cannot be obtained and the concrete cannot be protected against freezing and thawing while critically saturated until it is moderately mature, and for which circumstance precludes 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 (5) 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.

AGGRESSIVE CHEMICAL EXPOSURE

Sulfate Attack

The Corps of Engineers' requirements (6) 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. If the amount of sulfate is between 0.1 percent 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, tricalcium aluminate, 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 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 sea water.

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 C 1012, 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 C 1157).

Acid Attack

Besides sulfate attack, other aggressive chemical exposures involve acid and CO_2. Concrete in most 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 there was no structure made out of concrete that came into contact with the acid. The Corps of Engineers was asked 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 millimetres below the surface, the cement paste in hardened concrete typically no longer contains 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. 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 uses 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 than 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 "fortunately the use of studded snow tires has been declining for a number of years," and it specifically calls attention to the NCHRP Synthesis 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 a 50-mm-thick (2-in.) cover of high-quality concrete 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 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 high-quality, low w/c, properly cured concrete. What we do is one or both of two things, and 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 structures do exist where their owners believed that they were using properly protected 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 his or her reinforced concrete 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 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) and concrete and external prestressing.

There has been much debate on the issue of how much chloride should be allowed to be present in the concrete at the time it is produced. We are now, of course, aware 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 shouldn't 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 54 ff). 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 a very widely used as aggregate 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 all of the cement in Iceland for which all of the calcareous raw material is sea shells dredged from the ocean) and 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 must buy substantial amounts of potash fertilizer because the cement plant upwind is catching the kiln dust that used to go out its stack 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 they 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, expanding, and disrupting the concrete.

If the circumstances are such that any problem 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 percent Na_2O plus 0.658 times the percent K_2O. If low-alkali cement is not available, then 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 C 441.

There is also an alkali-carbonate-rock reaction which has been found in a number of 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 the case the fact that the rim is on crushed stone particle is evidence that a reaction that took place in the concrete.

The way to avoid harmful alkali-carbonate-rock reaction is to use nonreactive aggregate or to figure out a combination with low-alkali cement. The ACI committee report recommends a value less than 0.60 percent Na_2O equivalent.

CONCLUSIONS

To obtain durable concrete, 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 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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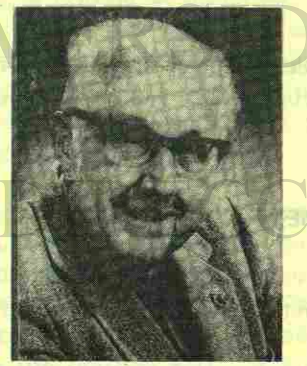

Bryant Mather, Director, Structures Laborato
Departament of the Army, Waterways Experime
Station, Corps of Engineers, Vicksburg, MS,
39180-6199, USA.

ALKALI-AGGREGATE REACTION EFFECT

by

Juan Luis Cottier Caviedes

Synopsis: In this paper it is intended to show the Alkali-Aggregate Reaction State-of-th Art which is a very important aspect in concrete structures durability. Now a days, th subject is becoming more and more important due to the increase of building structur affected by this reaction, as well as more aggregates are being identified as potentia reactive.

Mechanisms of reaction are explained in a general way, as well as factors whi influence its development; for example, alkalis present in concrete, and potentially reacti aggregates, etc. Main effects of this reaction in concrete are mentioned, as cracking, lo strength which affects structures safety and durability.

Keywords: alkali-aggregate reaction, aggregate, cement, expansion, damage, deleteriou alkalinity, elasticity modulus, reinforce, structure.

EFECTOS DE LA REACCIÓN ALCALI-AGREGADO EN EL CONCRETO

por

Juan Luis Cottier Caviedes

Sinópsis: Se intenta mediante el presente trabajo, mostrar el estado actual del conocimiento acerca de la reacción álcali-agregado, que constituye un aspecto muy particular de la durabilidad de las obras de concreto. El tema cobra interés día a día ya que van en aumento las obras identificadas como afectadas por la reacción, así como también se han identificado más agregados como potencialmente reactivos.

Se exponen en forma general los mecanismos de desarrollo de la reacción, los factores que influyen su creación como son: la presencia de álcalis en el concreto y los agregados potencialemnte reactivos, entre otros. Se expone de igual forma cuales son los principales efectos que la reacción causa en al concreto, como son: agrietamiento y disminución de la resistencia, lo que afecta a las estructuras en su seguridad y durabilidad.

Palabras clave: Reacción álcali-agregado, agregado, cemento, concreto, expansión, deterioro, deletéreo, alcalinidad, resistencia, módulo de elasticidad, refuerzo, estructura.

Juan Luis Cottier Caviedes. Ingeniero Civil egresado de la Facultad de Ingrenieria de Universidad Nacional Autónoma de México. Estudios de posgrado en la Ecole Nationale d Ponts et Chaussés en París, Francia y en el Instituto de la Construcción y del Cemen "Eduardo Torroja" en Madrid, España. Ha sido jefe del laboratorio de materiales de Facultad de Ingeniería de la UNAM y ha dirigido proyectos de diseño y reparación de obr en la iniciativa privada. Actualmente es gerente del departamento de Asesorías Técnic del Instituto Mexicano del Cemento y del Concreto.

LA REACCION ALCALI AGREGADO EN EL CONCRETO

Antecedentes

El concreto ha sido -desde su descubrimiento- reconocido como un material d construcción durable, entendiendo como durabilidad la capacidad de resistencia d material, a lo largo del tiempo, en relación con las condiciones del medio ambiente a como también con las condiciones de servicio de las estructuras.

Material compuesto de la mezcla de cemento, agua y agregados, los cuales reaccionar y endurecer forman una piedra artificial, con un comportamiento que se acept como homogéneo.

Las características propias del concreto como son: resistencia, manejabilidad, form de producción, facilidad de tomar la forma del molde donde se deposita, durabilidad, etc le han permitido ser el material de construcción más usado a través del tiempo.

En los inicios del siglo XIX se aceptaba la hipótesis de que los agregados (arena grava) eran cuerpos inertes cuya función era principalmente como relleno sin activida alguna, además se aceptaban solo dos agentes que causaban daños al concreto: congelamiento y el agua de mar, haciendo a un lado cualquier otro tipo de reacciones qu se relacionaran con la pasta y los agregados las cuales, por lo general, tienen efecto perjudiciales que normalmente dan origen a expansiones nocivas para la integridad d concreto.

Durante los años 20's y 30's en el estado de California, en Estados Unidos, s observaron una serie de agrietamientos en estructuras que cumplían ampliamente con la especificaciones marcadas en los códigos de diseño, construcción y calidad de lo materiales; motivados por encontrar una respuesta real al fenómeno de agrietamiento, u amplio grupo de técnicos se dieron a la tarea de estudiar el comportamiento inexplicab

En el año de 1940 el investigador Thomas Stanton pudo demostrar la existencia de la reacción llamada álcali-agregado, como un proceso intrínseco de degradación del concreto, dejando en claro que las condiciones del medio ambiente en que se encuentran las estructuras son tan importantes como lo es el cemento y los agregados, reafirmando el verdadero papel que los agregados desempeñan como componentes activos, utilizados en la dosificación del concreto.

Estudios posteriores a los realizados por Thomas Stanton demostraron que éstos agrietamientos y expansiones en el concreto tenían su origen en una combinación de cemento con alto contenido de álcalis y agregados opalinos usados en su dosificación.

Durante las décadas siguientes este fenómeno se ha estudiado en diferentes laboratorios y en diversos países tales como Australia, Canadá, Francia, Nueva Zelanda, China, Sudáfrica, etc.; investigaciones que han experimentado un rápido progreso en distintas direcciones, logrando identificar los diferentes tipos de agregados que son susceptibles a intervenir en dicha reacción, mecanismos de desarrollo y métodos de diagnóstico. Muchos han sido los aportes al tema, sin embargo se pueden identificar cuatro como de particular importancia: Thomas Stanton de Estados Unidos quien explicó por primera vez el fenómeno de la reacción, Swensoon de Canadá, quién identificó la reacción Alcali-Carbonato, Idarm de Dinamarca quién por primera vez investigó un concreto dañado con la reacción álcali-agregado en Europa y Vivian de Australia quien explicó los mecanismos de la reacción.

Las investigaciones de T. Stanton marcaron el camino para innumerables investigaciones en el área de la reacción álcali-agregado que han enfocado sus objetivos, primordialmente, en métodos que permitan seleccionar el agregado a utilizar para evitar la reacción, dejando en un plano secundario la identificación del fenómeno en el concreto endurecido, sistemas de monitoreo del desarrollo de la reacción, así como métodos y materiales de reparación de estructuras afectadas por la reacción.

Principios de la Reacción Alcali-Agregado

La reacción álcali-agregado se identifica como un proceso físicoquímico en el cual intervienen principalmente los minerales que constituyen la roca utilizada como agregado, según sea su naturaleza cristalina o amorfa y los hidróxidos alcalinos del concreto que pueden ser aportados, bien por el cemento, por los mismos agregados o por algún agente externo.

Gran parte de los agregados utilizados en la dosificación del concreto con cemento portland son químicamente estables y sin interacción deletérea con otros ingredientes del concreto, sin embargo este no es el caso de los que contienen ciertos minerales que reaccionan con los álcalis solubles en el concreto.

Esta reacción que se genera es denominada en forma general como álcali-agregado identificándose tres diferentes tipos de reacción como son:

- Alcali-carbonato
- Alcali-silicato
- Alcali-sílice

Varios tipos de interacciones pueden ocurrir en cada clase y no todas so necesariamente expansivas o deletéreas.

De las reacciones presentadas, la primera se considera diferente de las otras pues s lleva a cabo entre los álcalis aportados por el cemento en la fase líquida del concreto y la rocas carbonato; este es un caso poco frecuente. Se identifica como un proceso químic de dedolomitización esto es una descomposición de la dolomita ($CaMgCO_3$) en presenc del hidróxido de calcio $Ca(OH)_2$ propiciando la formación de calcita $CaCO_3$ y de brucit $Mg(OH)_2$, minerales estables e insolubles.

En la realidad se conoce poco de este tipo de reacción por lo poco frecuente que s presenta. Existe un solo tipo que se produce en presencia de agregado fino o aren dolomítica, la cual contiene calcita y arcilla intersticial y produce expansione significativas.

Las reacciones álcali-silicato ocurren en concretos ricos en álcalis los cuales contiene argilita y rocas del tipo grauvaca en el agregado. La reacción de este género de rocas y lo álcalis es por lo general lenta y no está completamente comprendida. Los constituyente silíceos en los agregados pueden expandirse causando la ruptura del concreto. Por l expansión de partículas individuales, se sugiere la absorción de agua sobre la superficie alúmino-silicosas previamente secas localizadas en las porciones microcristalinas de la mismas. Se deduce que puede existir una relación directa entre la cantidad de materi microcristalino, la porosidad y la expansión del concreto que contiene estos agregados

La reacción más frecuente donde intervienen los hidróxidos álcali y el material silíceo d los agregados del concreto es identificada como álcali-sílice, fenómeno que e particularmente expansivo ya que tiene la capacidad de desarrollar suficiente presión d dilatación para fisurar y romper el concreto. Generalmente la reacción progresa lentament permitiendo que las expansiones sean previstas algunos años antes de que el daño de l estructura sea de gravedad.

Esta reacción tiene la particularidad de producir un gel álcali-silicoso el cual e higroscópico y es el resultado de la interacción de los álcalis solubles en el cemento y lo elementos integrantes de las partículas de agregados, que por su característica de s hidrofílico absorbe humedad, incrementando su volumen. De esta manera, gene presiones suficientes para fracturar la estructura del concreto.

De acuerdo a diferentes investigadores, la reacción se considera que progresa en función de las siguientes ecuaciones idealizadas:

$$4SiO_2 + 2NaOH = Na_2Si_4O_9 + H_2O$$
$$SiO_2 + 2NaOH = Na_2SiO_3 + H_2O$$

En términos generales, la reacción puede ser en dos etapas. La primera es la hidrólisis de la sílice reactiva por OH- formando un gel, y en una segunda fase comienza la absorción de agua por lo cual el gel aumentará de volumen induciendo a la generación de presiones que formarán microgrietas cercanas a los lugares de la reacción, permitiendo su propagación y aglutinamiento, provocando así, agrietamientos dentro de la estructura del concreto y expansiones generalizadas del elemento afectado, como se observa en la foto 1.

Se observa que al aparecer las primeras grietas, estas permiten el acceso de agentes degradantes al interior del concreto, ocasionando que aparezcan otros mecanismos destructivos. El fenómeno de corrosión del acero de refuerzo, no se desarrolla en forma convencional, debido al pH del gel que es altamente alcalino. El fenómeno de lixiviación del carbonato de calcio es común. El carbonato de calcio se deposita sobre las superficies externas del concreto, y deja insterticios mayores en el interior del mismo. Se ha encontrado etringita en las grietas, mismas en las que se ha observado el gel álcali-silicoso, observándose las variedades de etringita cristalina y amorfa, así como desarrollos normales de portlandita en la matriz cementante.

Es importante hacer notar, que en la mayoría de los casos, la etringita encontrada es la que normalmente se forma entre el aluminato tricálcico y el sulfato de calcio en la hidratación del cemento, lo que sugiere que ninguna fuente externa de ataque de sulfatos ha causado el desarrollo de la etringita, sino que por el contrario el sulfato necesario se deriva de la matriz cementante.

El remplazo de gel por etringita también sugiere que la reacción avanza antes de la formación total de la misma, la cual se estima que se desarrolla de manera principal en el gel, cuya composición es muy variable como se puede observar en la tabla 1.

El transporte de iones sulfato conjuntamente con el agua para la hidratación del gel álcali-sílice, quizá sea el mecanismo por medio del cual los cristales de etringita se desarrollen y crezcan. Su crecimiento en microgrietas y poros en la pasta de cemento pueden ejercer suficiente presión dentro de la estructura del concreto para contribuir a las expansiones observadas en elementos estructurales.

Necesidades de Alcalis en la Reacción Alcali-Sílice

En forma generalizada las estructuras de concreto que se han identificado como dañadas por la reacción fueron construidas con un concreto con cemento portland ordinario, el cual normalmente contiene una pequeña proporción de sodio (Na) y potasio (K) presentes como sulfatos y sulfatos dobles $(Na, K)SO_4$, los cuales tienden a cubrir a otros minerales clínker y también como constituyentes menores en los otros minerales del cemento.

Al parecer los álcalis tienen su origen en la materia prima utilizada en la fabricación cemento, usualmente la fracción arcillosa y el carbón (si éste es utilizado co combustible del horno). Si el material arcilloso utilizado como materia prima contiene m o arcilla ilítica, entonces el clíncker producido estará enriquecido en potasio, mientras si está presente el feldespato degradado, el clincker puede contener más sodio o potas o ambos, dependiendo de la composición del feldespato en la materia prima.

Las cantidades finales de los álcalis presentes en un clincker dependerán de proporciones de mica, ilita o feldespato en la alimentación del horno.

Las fases álcali tienden a ser una fracción volátil en el ambiente del horno. Alrede de un % se volatiliza durante el proceso de quemado. Una gran cantidad del álcali redepositado en la cadena de sección del horno y en los precalentadores, precipitado de polvo y filtros. A fin de reducir los consumos de combustible y emitir gases m limpios, la recirculación de polvos es práctica común en las plantas modernas pero e procedimiento tiene un efecto adverso en la composición del clincker del cemento.

Con el objeto de contabilizar la cantidad de álcalis presentes en el cemento o concre se ha generalizado la práctica de expresar el contenido de álcalis en términos de so equivalente, correlacionando los óxidos de sodio y potasio en términos de proporcion moleculares. El cálculo a seguir es bajo la relación matemática siguiente:

Sodio equivalente = $Na_2O + 0.653\ K_2O$ relación en peso.

Cuando el valor del sodio equivalente se encuentra por debajo de 0.6% en peso, reacción álcali-sílice no puede llevarse a cabo; este valor es recomendado por much autores como máximo permitido a fin de minimizar el riesgo de daño. Del mismo modo, masa de álcalis provenientes de otras fuentes no debe ser mayor de 3 Kg/m3 en concreto. Es importante señalar que en estructuras donde se ha identificado la reacci estos valores han sido encontrados en niveles menores. Esto puede ser quizá po resultado de álcalis que han sido lixiviados de la estructura con el tiempo.

Una vez iniciada la reacción, es capaz de generar suficiente energía libre, que le perm continuar a pesar de los bajos niveles de álcalis, o concentraciones altas de álcalis m localizadas dentro del concreto, siendo capaces de mantener la reacción en esos siti

Se tienen evidencias en base a microscopía electrónica de que, a pesar de que exist bajos niveles de álcali en la pasta de cemento, las partículas reactivas de agregado en mismo concreto pueden inducir niveles altos de reacción dentro del concreto, en conjunto.

Componente Sílice-Reactivo en el Concreto.

Para que la reacción se lleve a cabo, es necesaria la presencia de una determinada forma de "sílice reactivo". El volumen para producir efectos deletéreos necesita ser solo muy pequeño. En estructuras donde se han observado daños calificados como severos, se han reportado componentes reactivos del 2%. Existen varios tipos de rocas que son utilizados en la fabricación del concreto y solo rocas puras tales como las calizas, se pueden excluir de la posibilidad de contener una pequeña proporción de una forma de sílice reactiva, ya sea como constituyente original, primario o secundario.

El considerar únicamente el tipo de roca como criterio para evaluar su potencial de reactividad, nos lleva a situaciones erróneas, por lo cual diversos autores han puesto especial atención en los constituyentes minerales de la roca misma.

Aunque por lo general, la mayor parte de las rocas son capaces de contener formas reactivas de sílice, el número de tipos de sílice que exhiben reactividad es pequeño.

Probablemente los requisitos dominantes para que un material sea reactivo son, entre otros, que deberá ser una forma de sílice que es pobremente cristalina o contiene muchos defectos de arreglo, o alternativamente debe ser amorfa o vítrea en carácter. Un ejemplo de los minerales naturales que cumplen estos criterios se presenta en la tabla 2.

Algunos granitos, gneises graníticos, hornblendas y grauvacas, se ha encontrado que son reactivos cuando se utilizan en el concreto. Se ha notado que, aunque el mineral reactivo preciso constituyente dentro de estas rocas no puede ser identificado, los granos de cristal de cuarzo que ellas contienen muestra que son amorfos cuando son examinadas utilizando un microscopio polarizado. Una observación ulterior (ref. 6) mostró que la reactividad puede ser correlacionada en forma general con la severidad del esfuerzo.

Además de las características de la sílice reactiva, existen otros factores que influyen en la forma como se desarrolla la reacción y la intensidad de sus efectos.

Entre los factores más importantes se encuentran la temperatura, la humedad y la granulometría de los agregados.

La temperatura es un factor que incrementa la velocidad con que se produce la reacción e interviene en las dos etapas del desarrollo de la reacción: creación del gel y proceso de expansión.

Existe una prueba la cual permite demostrar la influencia de la temperatura en la primera etapa. Esta prueba es identificada como "trozo de gel" y fue desarrollada por Jones y Tarleton (ref. 2), en la cual las partículas de agregado reactivo están expuestas a soluciones concentradas de álcali sobre la superficie de una tableta de cemento, a temperatura ambiente. Los materiales altamente reactivos, tales como el ópalo, desarrollan gel sobre sus superficies, dentro de unos pocos días de almacenados. Si la temperatura de almacenamiento alcanza unos °C, el desarrollo de gel sobre la superficie de tales partículas ocurrirá dentro de 24 horas, mientras que en algunos materiales menos reactiv tales como algunos pedernales, comenzarán a mostrar desarrollo de gel, aunque temperaturas normales no se presenten signos de tal reacción.

En la segunda etapa de la reacción, cuando la temperatura es alta, las expansion producto de la absorción de agua se generan en forma más rápida y su inicio es m temprano. Sin embargo a medida que la reacción continúa, tanto el rango de reacc como el rango de expansión disminuyen. Cuando los concretos reactivos son almacenad a bajas temperaturas, reaccionan en forma más lenta, sin embargo, eventualmente, expansión alcanza el mismo nivel y puede exceder las expansiones alcanzadas a alt temperaturas.

La influencia de la variación de la temperatura y su ciclicidad sobre la reacción y expansión, es aún un tema en discusión, pues los efectos resultantes de una variac rápida en la temperatura estarán presentes en las capas externas y expuestas dado q los cambios de temperatura dentro de una estructura de concreto en circunstanci normales serán lentos y limitados.

Existen numerosos reportes donde se indica que las partes de una estructura que est expuestas a los elementos del medio ambiente están más severamente dañados por l efectos de la reacción, que otras partes que se encuentran protegidas del intemperism Existen casos donde hay una diferencia notable entre la superficie expuesta a la humed y los lados protegidos en una misma estructura.

El agua tiene una función dual. Primeramente, es esencial como portador de l cationes álcali e iones oxhidrilo, y en segundo lugar es absorbida por el gel hidroscópic el cual se expande, desarrollando presiones suficientes para agrietar al concreto.

Es importante recordar que el concreto aún en condiciones secas, tendrá la capacida de absorber agua, así que, con la excepción de una capa exterior de poco más de 10 m de espesor la humedad relativa dentro del concreto permanecerá entre un 80 a 90%.

Investigaciones y experimentos han demostrado que los efectos de la reacción tal como la expansión, varían directamente con el porcentaje de humedad relativa d concreto. El tipo de relación se ilustra en la figura 1, donde puede verse que abajo del 70 de la humedad relativa, la expansión y la reacción expansiva no son significativas, pe arriba del 80% de humedad relativa, los efectos de expansión se ve que se increment dramáticamente.

Se ha observado así mismo, que la reacción puede ser capaz de formar geles de ba contenido de humedad, en un inicio, los cuales se expandirán y ejercerán presione conducentes a la expansión inmediatamente que llega a haber disponibilidad de agu también hay evidencias de que el gel parcialmente deshidratado puede ser rehidratad generándose así una expansión cuando se añade agua adicional al especimen, debido a qu el gel seco que se ha vuelto blanco y carbonatado, puede ser reconstituido y no se fácilmente soluble en agua.

Relación "Pésima".

Se han mencionado los diferentes factores que gobiernan el desarrollo de la reacción álcali-agregado. Sin embargo, es interesante indicar que las peores condiciones que pueden presentarse en un concreto no son aquéllas donde los factores de reacción se encuentran en sus máximas concentraciones. Este fenómeno fue ampliamente estudiado por Vivian (ref. 3), denominando a dicho ámbito como "proporción pésima", en donde demostró que, para cada agregado reactivo, tiene que estar presente un determinado contenido de álcali, para que se produzca la máxima expansión. De dichos estudios se encontraron curvas que, esquemáticamente, pueden ilustrarse de acuerdo a la figura 2.

Las curvas obtenidas varían en función de la cantidad y distribución granulométrica del agregado reactivo, es decir, de la superficie expuesta a la reacción y de la relación sodio o potasio en que se presentan los álcalis liberados.

Así, resulta que cementos con muy distintos contenidos de álcalis, pueden dar la misma expansión, con un agregado determinado, variando la proporción de éste último.

Efectos de la Reacción Alcali-Agregado

Como se ha comentado, la reacción álcali-agregado es un fenómeno de carácter expansivo, el cual produce agrietamientos, generando esfuerzos en el seno del concreto con la aparición de fisuras. La reacción tiene lugar en la unión de la pasta y el agregado reactivo, así como también en pequeños poros y microfisuras. En ocasiones se genera un anillo en el contorno del agregado reactivo según se observa en la fotografía 2, y en ciertos casos se presenta una exudación en la superficie del concreto que permite que sea más visible el agrietamiento superficial del concreto (ver foto 3).

La expansión del concreto tiene una influencia negativa en las propiedades mecánicas del mismo. El investigador Swamy (ref. 5) en uno de sus trabajos, investigó el comportamiento de los concretos dosificados con agregados considerados como altamente reactivo, uno, y otro como de reactividad moderada según se muestra en la tabla 2; en ella, se puede ver la pérdida en la resistencia a compresión simple, la cual puede llegar a ser de un 40 a un 60% menor a la resistencia especificada de proyecto. De igual manera se tiene registro de una disminución de resistencia a tensión de alrededor de un 65 a 80%. La pérdida del módulo de elasticidad se ha estimado entre un 60 y 80%. La pérdida de resistencia y del módulo de elasticidad, intervienen en la rigidez de los elementos y, consecuentemente, influyen en el comportamiento estructural y la durabilidad de las edificaciones.

La presencia de acero en el concreto es un elemento que interviene en el aspecto del agrietamiento, ya que impone restricciones a la fisuración. Por la naturaleza expansiva del fenómeno y los esfuerzos creados en la masa del concreto, el aspecto de las fisuras creadas por el agrietamiento en elementos sin refuerzo, será de forma casual y muy irregular, ocurriendo en todas direcciones, con un aspecto como el de la piel de un cocodrilo, también conocido como "mapeo" (fotos 4 y 5). En estructuras reforzadas, panorama es diferente, ya que las fisuras se presentan en forma paralela al acero princ y en dirección de los esfuerzos predominantes. La expansión creada en el concreto impo esfuerzos de tensión al acero de refuerzo, que aunados a los esfuerzos de compresión concreto contiguo al refuerzo le imponen una restricción para deformarse, lo cual da lu al nacimiento de fisuras paralelas a la posición del refuerzo, mismas que pueden llega tener un espesor de 15 mm y una profundidad de 30 cm, sobrepasando por mucho capa de recubrimiento de los elementos (foto 6).

Debido a que la humedad y temperatura favorecen el desarrollo de la reacción, el da causado podrá variar en una estructura, ya que los elementos que se encuentran contacto con el medio ambiente estarán mayormente afectados que aquéllos que encuentran protegidos, aún cuando todos ellos estén construidos con el mismo concre

Los métodos de auscultación y diagnóstico de un concreto dañado, así como evaluación del grado de reactividad de los agregados quedan fuera de los objetivos presente trabajo, sin embargo es importante comentar que existen grandes adelantos las investigaciones que permiten contar con métodos confiables para poder determina inclusión o no, de un agregado en el concreto y qué tipo de práctica es recomenda ejecutar a fin de minimizar los efectos nocivos de la reacción álcali-agregado.

CONCLUSIONES

La reacción álcali-agregado es un fenómeno de carácter expansivo, el cual tiene origen en la interacción química entre los álcalis liberados en la hidratación del cemen portland y los minerales reactivos, como la sílice amorfa, que presentan ciertos agregad usados en la dosificación del concreto.

El desarrollo de la reacción depende de factores como: presencia de agregad reactivos, presencia de álcalis en el concreto y, de la humedad y la temperatura. Por naturaleza de la reacción, se puede definir su desarrollo en dos etapas principales que s la primera: formación de gel, y la segunda: absorción de agua y expansión del produc alcalino provocando daños al concreto. Las dos etapas, aún en estudio, se generarán una estructura en medio ambiente normal.

Los efectos causados por la reacción van desde la aparición de fisuras en la masa d concreto hasta llegar a la disminución en la resistencia a compresión simple del concre y variación en el módulo de elasticidad, efectos irreversibles que merman la seguridad durabilidad de las estructuras afectadas.

El conocimiento del principio de la reacción y los factores que la generan y desarrol ha permitido la creación de una serie de ensayes y metodologías orientadas a evitar siendo aún motivo de estudio el seguimiento que se debe dar a una estructura afectada principalmente, que tipo de soluciones son aplicables para reparar elementos dañados p la reacción.

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Ing. Juan Luis Cottier Caviedes,
Gerente de Asesorías Técnicas,
Instituto Mexicano del Cemento
y del Concreto, (IMCYC).
México, D.F.

Tabla 1. Análisis Químico del Gel Alcali-Sílice Desarrollado por Diferentes Investigadores

Na_2O	K_2O	SiO_2	CaO	MgO	DIFERENCIA EN 100%	INVESTIGADOR
12.9	-	53.9	2.9	0.6	29.8	Stanton (1942)
12.9	-	53.4	2.6	0.8	30.2	
14.9	5.2	61.7	0.6	-	17.6	
13.4	5.1	65.5	0.5	0.2	15.3	Idorn (1961)
12.4	4.9	69.9	0.3	0.5	12.0	
17.9	8.2	73.7	1.1	0.1	0.0	
9.4	4.1	72.8	1.3	0.2	11.2	
14.6	6.2	61.9	-	0.1	17.2	
16.2	5.7	58.8	-	-	21.3	
8.2	4.1	56.1	17.4	0.2	14.0	
8.3	5.0	28.5	22.5	0.2	35.6	
1.2	0.4	51.4	29.9	10.0	17.1	
7.4	0.7	53.0	22.1	10.0	16.7	
1.5	13.9	38.9	27.3	-	17.4	Hobbs (1980)
0.4	4.7	51.1	21.5	-	22.3	Regourd (1983)
	0.6	27.9	35.2	-	36.2	
1.0	6.9	61.5	9.2	-	21.3	Oberholster(1983)
1.0	6.2	53.8	8.2	-	30.7	
1.8	5.5	49.9	12.8	-	29.9	
1.0	5.2	.4	12.0	-	30.7	
1.4	9.0	62.9	12.5	-	13.8	
0.8	7.4	53.2	10.0	-	28.4	
1.2	4.1	66.5	6.5	-	21.7	Baronio (1983)
3.7	12.9	43.3	21.5	0.8	14.0	Mullick (1986)
3.9	11.4	49.4	15.9	0.5	16.7	

Tabla 2. Efectos de la Expansión provocada por la Reacción Alcali-Sílice sobre las Propiedades del Concreto

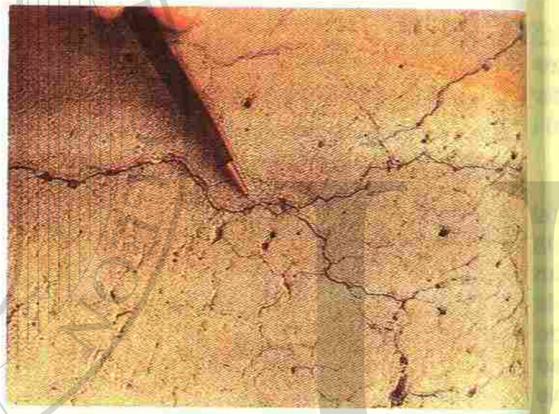
PROPIEDAD	MEZCLA	EDAD (DIAS)			
		1	7	28	36
EXPANSION (%)	A. Control	0.000	0.000	0.003	0.02
	b. 4 1/2 ópalo	0.000	0.071	0.316	1.64
	c. 15% sílice amorfa	0.000	0.000	0.023	0.62
RESISTENCIA A COMPRESION	A. Control	26.700	48.600	60.100	73
MPa	b. 4 1/2 ópalo	31.100	44.400	44.0	27
	c. 15% sílice amorfa	-	-	52.0	44
RESISTENCIA A TENSION	A. Control	2.610	3.580	3.900	4.29
MPa	c. 15% sílice amorfa	-	-	3.290	1.8
MODULO DE RUPTURA	A. Control	3.520	4.880	52.2	5.58
MPa	c. 15% sílice amorfa	-	-	4.580	1.30
MODULO DINAMICO DE	A. Control	35.600	41.000	42.0	45.40
ELASTICIDAD (GPa)	b. 4 1/2 ópalo	33.900	32.700	20.800	10.40
	c. 15% sílice amorfa	-	39.0	40.800	18.90

1. La guarnición muestra daños en forma de cuña, presentando desplazamientos en sentido vertical respecto a su posición original.

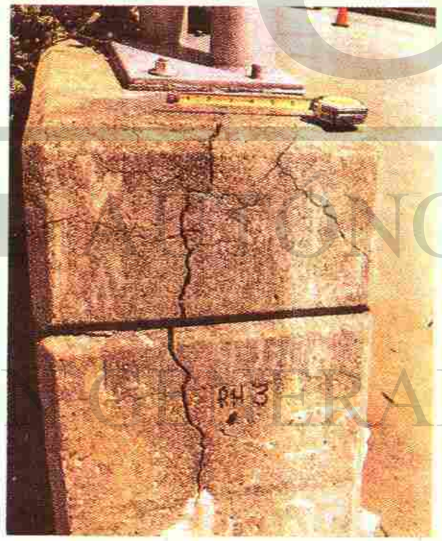
2. Se observa en la muestra de concreto los anillos de reacción que circundan al agregado reactivo.

3. Agrietamientos y movimientos diferenciales en las secciones colindantes de un puente carretero, la exudación resalta la configuración de las grietas.

4. Aspectos del agrietamiento identificado como "mapeo" defecto presente en un pavimento de concreto.



5. Acercamiento del agrietamiento identificado como "mapeo".



6. Grietas cuyo espesor es mayor de 10 mm y profundidad variable que supera en algunos puntos los 1 mm.

DURABILIDAD DE LOS AGREGADOS Y DEL CONCRETO CONTROLADOS POR ADSORCION DE AGUA Y DE CATIONES Y POR OSMOSIS

por

Peter P. Hudec

Sinopsis: La superficie interna de los sólidos porosos hacen que las paredes del poro los agregados y del concreto jueguen un importante papel en la durabilidad de est materiales bajo condiciones cíclicas de humedecimiento-secado y congelamien descongelamiento. Esto es especialmente verdadero si los poros son pequeños.

Este trabajo presenta tanto los argumentos teóricos como la evidencia experimen relacionando las características de la superficie del poro y el tamaño de este con expansión y falla de los agregados y del concreto. La magnitud de las fuerzas (cargas la superficie son determinadas por el tipo de mineral que integra la superficie del poro estabilidad cristalina de la matriz y si la superficie es una superficie de cristal, c hendiduras o fracturada. La magnitud en las cargas en la superficie determinan la cantid de agua que es adsorbida de la superficie. Los iones en solución son preferiblemen adsorbidos a la superficie y aumentan la cantidad de agua retenida en ésta. El ag adsorbida tiene niveles menores de presión de vapor y si el poro es lo suficientemen pequeño para poder llenarse completamente con el agua adsorbida, la baja presión vapor de agua actúa como un fluido osmótico, estableciendo fuerzas osmóticas y expansión. Los iones presentes en la superficie y en el agua en los poros acelera proceso.

Mientras mayor sea el área total de la superficie interna para un tamaño dado sólido, menores serán los poros y mayor será el potencial osmótico. El área de superficie interna está directamente relacionada con el daño por congelamiento descongelamiento. Se demuestra que el contenido de agua adsorbida está relacionada c la expansión isotérmica, con los ciclos de humedad-secado y por el daño por ciclos congelamiento y descongelamiento. Además se demuestra que existe una relación ent la expansión por la reactividad alcalina y la expansión isotérmica al humedecer el concre que no ha reaccionado.

Palabras Claves: Superficie interna, poros, adsorción, expansión isotérmic humedecimiento y secado, congelamiento y descongelamiento, durabilidad, reactivid alcalina.

AGGREGATE AND CONCRETE DURABILITY AS CONTROLLED BY WATER AND CATION ADSORPTION AND OSMOSIS

by

Peter P Hudec

Synopsis: The internal surfaces of porous solids that make up the pore walls in aggregate and concrete play a significant role in the durability of those materials under cyclic conditions of wetting and drying and freezing and thawing. This is especially true if the pores are small.

The paper presents both theoretical arguments and experimental evidence linking the characteristics of the pore surface and pore size to expansion and failure of aggregates and concrete. The magnitude of surface forces (charges)are determined by the type of mineral making up the pore surface, its crystal matrix stability, and whether the surface is crystal, cleavage, or fracture surface. The magnitude of the surface charge determines the amount of water that is adsorbed to the surface Ions in solutions are preferably adsorbed to the surface, and increase the amount of surface-held water. The adsorbed water has lower vapour pressure, and if the pore is small enough so that it is completely filled with adsorbed water, the low vapour pressure water acts as an osmotic fluid, setting up osmotic forces and expansion. Ions present on the surface and in the pore water exacerbate the process.

The larger the total internal surface area for a given size solid, the smaller the pores, and the larger the osmotic potential. Internal surface area is directly related to freeze-thaw damage. Adsorbed water content is shown to be related to isothermal expansion, wet-dry cycling, and freeze-thaw cycling damage. A relationship between alkali reactivity expansion and the isothermal expansion on wetting of the un-reacted concrete is also demonstrated.

Keywords: Internal surfaces, pores, adsorption, isothermal expansion, wetting and drying, freezing and thawing, durability, alkali reactivity.

Peter P Hudec is a professor of Engineering Geology, specializing in durability of build materials, specifically aggregates and concrete. He has been working in this field for last 30 years, as researcher, teacher, and consultant.

INTRODUCTION

The durability of porous inorganic construction materials (stone, aggregate, concre tile, brick, etc) can be defined as their resistance to <u>natural</u> deterioration due to weather and <u>induced</u> deterioration due to impact, wear, temperature extremes, and the use deicing salts and other chemicals.

If the material remains in a dry state, the amount of deterioration is likely to be minir (excluding dry thermal expansion or contraction, which in some environments can severe). Likewise, if the interior of the material is kept dry because it lacks pores because the pores are not connected, very little damage is likely to occur Interconnec porosity (effective porosity) which provides access to the material's interior and moistu are necessary for deterioration to occur.

Moisture in the form of water or ionic solution and a porous, permeable solid a required before any deterioration takes place. The total porosity, effective porosity, a pore size distribution, along with the degree of saturation by moisture are among the m factors in the durability of porous solids Pore size is especially important For giv porosity, the material with the smaller pore size is more likely to fail.

INTERNAL PORE SURFACES, ADSORPTION, AND PORE SIZE

Nature of Surfaces

Surface is a zone of discontinuity between the solid and either air or solution. surfaces posses a residual charge due to partially satisfied bonding. The magnitude a polarity of the charge is a function of the nature of the surface. Four types of internal po surfaces can be recognized in minerals and materials of a given building material:

1. Surface on amorphous solid
2. Crystal face surface
3. Cleavage surface
4. Fracture surface

All pore walls are lined with one of the types of surfaces listed above. Diagrar illustrating the nature of the surfaces are given in Figures 1 and 2. The strength of t surface force is indicated by the thickness of the band in both figures.

Amorphous solids are minerals that have not crystallized, that is, that have no or incomplete arrangement of internal elements (atoms). Examples of these are chert, opal or flint, and quickly chilled melt, such as glass, slag, and volcanic rock. Surfaces on these solids are very active, since few of the elements (atoms or ions) in the solid are bonded to other surrounding elements, and the unsatisfied bonds are reflected in high surface charges. This is reflected in the random orientation of the elements as shown in Figure 1.

Crystal face surfaces are the least charged, since the atoms making up the solid are bound to other internal atoms, usually by ionic or covalent bonding, and occasionally by Van der Wall bonds. Because the atoms bond by sharing electrons, only residual surface charges remain at the surface of the crystal. An illustration is given in the left upper diagram of Figure 2.

Cleavage surfaces of minerals in rocks and cement paste represent a relative plane of weakness along which the mineral preferentially fails when subjected to stress. The cleavage is a plane along which the bonds are relatively weak. When cleaved, the bond that held the two sides of the cleaved plane together now becomes a surface charge. The more difficult it is to cleave the surface, the higher the surface charge. The best example of this type of surface is a clay mineral, which cleaves easily along its basal plane; pure clay with a stable lattice such as kaolinite has a relatively weak residual surface charge. Calcite and feldspar respectively have progressively higher surface charges along their cleavage planes.

Fracture surfaces have the largest surface charges, even exceeding those of an amorphous solid. The break along a fracture surface takes place in a random direction across the solid, because the solid does not have a weak bond plane or direction. An example of a fracture surface would be that of quartz or garnet.

The charge on most crystal and cleavage surfaces is negative, but the charge at fracture surfaces can be positive. The surface charge in natural and artificial (hydrated cement) minerals is affected by the lattice imperfections in the crystal due to atomic substitution by elements similar to the principal element comprising the lattice - such as the substitution of Mg for Ca in calcite. The substitution may strain the lattice and significantly increase the surface charge of crystal, cleavage, or fracture surfaces.

Leaching of the lattice due to weathering will likewise increase surface charges. Leaching partially and selectively removes certain cations without changing the crystal lattice. The lattice remains that of the original mineral until a sufficient number of cations (and anions) are removed to make it totally unstable, at which point it recrystallizes into a new form stable under the given conditions. Thus, in the first instance, fresh chert changes to its weathered equivalent which is more active, and in the second instance, the feldspar of igneous rocks recrystallizes into clay, which may be more or less surface active, depending on the degree of 'perfectness' of its lattice.

Surface Activities of Hydrated Cement Paste and Aggregate

Hydrated cement -- Hydration of cement in the presence of water is basic crystallization of di- and tri-calcium silicates, tricalcium aluminate, and tetracalci alumino-ferrite (1). The result is a micro-crystalline mass of the above 'minerals'. T porosity and permeability of the hardened paste is governed largely by the water-cem ratio, all other conditions being equal. The surface activity of a paste pore is governed the cement composition and any additives that may have been used. It is therefore hig controllable and relatively constant. Since the paste crystallizes under ambient clima conditions, it is inherently more stable than minerals of aggregates that usually form un significantly different conditions of temperature, pressure, and moisture. The minerals the paste are hydrates, and as such in equilibrium with the ambient moisture conditio Adequate curing allows the minerals to develop stable lattices with relatively low surfa charges.

Aggregates -- Coarse and fine aggregates together make up approximately 75% of t finished concrete. Thus, the pore surface characteristics of the aggregate in so far as th affect its durability will have a major impact on the durability of concrete. The surfa activity of aggregate pore walls is a function of the minerals making up the aggregate, a the nature of the porosity. In most fresh igneous rock aggregate, the pore walls cons of crystal faces (Figure 2). Pure chemical precipitates (limestone and dolostone) a commonly also crystalline and micro-crystalline in texture, and pore walls behave simila to igneous rocks. However, limestone and dolostone are seldom pure, and may cont clay as impurities. Clay minerals, depending on type, have very active external a internal surfaces, and even in small amounts exert undue influence on the pore propert of the rock. Shale, consisting mostly of clay minerals, is of course undesirable aggregate. Sandstone and siltstone, depending on the grain type, size, type of impuriti and the cement type, present a variety of pore surfaces, ranging from relatively inact to very active.

The degree of weathering of aggregate greatly affects its properties. Dur weathering, the minerals undergo a variety of chemical alterations. The weather process results in a gradual change of the original minerals through lattice substitution a recrystallization into minerals most stable at the earth's surface - clay, iron hydroxi calcite, gypsum, etc. In general, there is a decrease in density and an increase in porosi The character of the pore walls change as the mineral composition changes. In the in stages, leaching of and substitution of different elements in the mineral lattice strains increasing its surface energy. The end product minerals are often smaller in size than t originals, decreasing the pore size. Also, the end product minerals are often more surfa active than their parents.

EFFECT OF GRAIN SIZE ON PORE SIZE AND ON THE TOTAL INTERNAL SURFACE AREA OF AGGREGATES

The chapters in most introductory texts dealing with the physical weathering of rocks give an example relating the importance of an increase in surface area to the decrease of particle size for the same weight of material. Thus, a one centimetre cube of solid has a surface area of 6 cm². If this 1 cm³ cube were divided into 1000 1mm³ cubes, the surface area would increase to 6000 cm². The relationship of grain size to surface area is given in Table 1. A rock containing smaller particle size is likely to have a significantly larger internal surface area than an equivalent coarser grained rock, all other conditions being equal. The relationship of internal surface area to freeze-thaw durability on a variety of building materials - rock, brick, and concrete is given in Fig 3 (after Blaine et al, (2)). The internal surface area determined by the BET nitrogen method gives an excellent correlation with durability - the larger the surface area, the lower the durability, as expressed in freeze-thaw cycles to failure (cracking).

The internal surface area bears a relationship to pore size. Pore size is in turn determined by the grain size of aggregate. Consider spheres of 1cm diameter packed into a 10cm per side cube. Assuming cubic packing, the pore size would be:

$$\text{pore diameter} = 1\ \text{cm}^3 - (4/3\ \Pi\ 05^3)$$

or 0476 cm³. For 001 cm³ diameter spheres, the maximum pore size would be 4764^{7} cm³. The relationship is shown in Table 2. The relationship between durability of carbonate aggregates (as expressed by. freezing and thawing tests) and their estimated grain size is shown in Figure 4 (3). The grain size of a variety of aggregate rock types has been estimated by assigning the small grain size a category one, intermediate category 2, and the large grain size a category 3. Although the relationship is a general one, a trend is indicated: the smaller the grain size, the lower the resistance to freezing and thawing. The wide range of freeze-thaw failure in the small grain size may be due to the fact that some larger pores may be present which are not related to grain size.

WATER AND ION ADSORPTION ON INTERNAL PORE SURFACES

Water Adsorption

Water is a di-polar fluid, and as such is attracted to the charged internal surfaces of the paste and aggregate. The polarity of water is due to the uneven distribution of hydrogen ions in the molecule. The shape of the molecule can be thought of as a 'Mickey Mouse Head' - a round face (oxygen) with two big ears (hydrogens) - Figure 5.

Because the internal surfaces are statically charged, water dipole molecules ar attracted to the surface and may cover it to a depth of several layers. These are terme 'adsorbed' water layers. The strength of the charge on the surface determines the amoun of water that will be adsorbed, the thickness of the adsorbed layer, and the 'rigidity' of th adsorbed water. The latter refers to the opposition to water flow within the adsorbe layers. The thickness of the double layer at different surfaces is proportional to th mineral's surface charge - the larger the charge, the thicker the adsorbed layer. In Figure 1 and 2, the thickness of the adsorbed water layer can by substituted for the sectio labelled 'surface force'.

Pore sizes in rock and concrete can be classified according to the dominant water typ that they contain - whether adsorbed, capillary, or normal (bulk) water. If a pore is sma enough, all of it will be occupied by adsorbed water. The pore size classification is show in a diagrammatic form in Figure 6 (4). The **critical pore** size in terms of durability can b considered that which is filled with adsorbed water at near 100% relative humidity. Th pore size contains both adsorbed and capillary water, both of which have lower vapo pressure than normal water.

Ion Adsorption

Any dissolved ions in the water, especially cations, have a profound effect on th adsorbed layer water thickness. The cations, being more strongly charged than the wat dipole, are preferentially adsorbed to the internal surfaces. The stronger charge of th cation attracts more water molecules, and the thickness of the layer increases. This illustrated in Figure 7. The affinity of water for the cation is a function of its charge/si ratio, and is given in the series below, where the first mentioned ion has the great affinity for water than the following:

$$Ca^{2+} > Mg^{2+} > Li^{+} > Na^{+} > K^{+} \qquad (5)$$

Divalent ions such as calcium have a greater hydration potential than monovalent ion Calcium is an abundant ion in cement paste, and thus has a dominant influence on 'critic pore size'. As will be seen later, the proportion of pores in the critical pore size rang determine the material's durability.

The effect of ions on the critical pore size is shown in Figure 8. Comparing Figure to Figure 8b, the net effect of the presence of ions is to fill a greater proportion of po with adsorbed water and thus to increase the critical pore size - that is, place more of th pores in the material in the critical category.

A. Vapour pressure in pores.
Vapour pressure in pores is a function of the surface tension of the solution, th temperature, and the pore radius and is expressed by Kelvin's equation:

$$\ln (p/p_o) = -2\,\sigma M / \rho R T r \qquad \text{(Eq 1)}$$

where: p/p_o = relative vapour pressure
p = pressure over concave surface
p_o = pressure over plane surface
σ = surface tension of solution
M = molecular weight of the solution
ρ = density of the solution
R = the Gas Constant
T = absolute temperature
r = radius of the pore

Because large changes (in orders of magnitude) of radius are possible, the radius is the main factor in determining the relative (and absolute) vapour pressure existing over the meniscus of the fluid in the pore. As the radius of the pore decreases, the vapour pressure over its meniscus also decreases. The vapour pressure of the fluid in the small pore can be significantly lower than the vapour pressure of fluid in a larger pore, or the vapour pressure outside the porous material (ie, if the porous material finds itself in a higher humidity environment or submerged in water).

Vapour pressure is also affected by the dissolved ion content, but to a lesser degree than that due to the pore radius. It is well known that the vapour pressure of a solution is a function of its molality - that is, the number of moles of the solute dissolved in one liter of the solvent, which in this case is water. As molality increases, vapour pressure of the solution decreases. The solute in the pore solutions of concrete paste is derived from the hydration reaction, and is mostly calcium hydroxide. The pore fluid in aggregates can contain a variety of soluble ions, the most common being the alkalies Na, K, and Ca.

OSMOTIC PRESSURE AND EXPANSION

Whenever two fluids of different concentration are brought in proximity, they seek to establish equilibrium, ie, common concentration through exchange of their dissolved ions The equilibrium can be established by transfer of water molecules through the vapour phase, through a semi-permeable membrane, or directly by diffusion. The driving force to establish equilibrium is osmosis (greek for *push*). Osmosis is defined as the spontaneous flow of solvent into a solution, or a flow from a more dilute to a more concentrated solution.

Nature of osmotic forces

In porous, permeable solids, if the pore is full of high concentration solution, the flow of the lower concentration solution from other pores is resisted. This resistance is known as osmotic pressure. The semi-permeable membrane in paste and aggregate are the small pores that contain tightly held adsorbed water. The tightly held water allows only wate molecules to diffuse through the pore, barring the larger hydrated cations. Th micropores, in effect, act as ionic filters. Thus, an osmotic potential develops if two large pores containing a solution of different concentrations are joined by a micropore as show in Figure 9 The magnitude of the pressure is expressed by the equation:

$$P_o V_1 = RT \ln (p / p_o) \qquad \text{(Eq 2)} \quad (6)$$

Where:
P_o = Osmotic pressure (dynes cm^2)
V_1 = Volume occupied by 1 mole of solvent, ie, water, (ml)
R = Gas constant
T = Absolute temperature, K
p/p_o = relative vapour pressure

Relative vapour pressure is used in the above equation, since it is directly proportiona to the concentration of the solute. The effect of relative vapour pressure on the osmoti pressure is given in Figure 10.

Adsorbed water as osmotic fluid

At elevated humidities common in moist concrete, capillaries having a diameter of micrometers or less may be totally filled with adsorbed water. The adsorbed water ha a lower vapour pressure, and can, according to Equation 2, set up significant osmoti pressures. This is illustrated in Figure 11. The internal pore pressure acts against th tensile strength of the paste or rock, expanding it. The tensile strength is, on the average 5 to 10 times lower than the compressive strength of the material. The rock aggregate that consist mostly of pores and capillaries in the critical diameter size will undergo interna stresses upon wetting and drying. The relationship of the adsorbed water to the expansio upon wetting of some typical Paleozoic carbonate aggregates from South West Ontario given in Figure 12 (7). A typical expansion curve with time of a 'sound' versus 'unsound carbonate aggregate from Figure 12 is given in Figure 13. Although the more expansiv carbonates did contain more clay (illite), the illite was non-expansive (ie, did not sho inter-lattice expansion), and this type of expansion was not a factor in the expansio observed. The clay in the pores was instrumental principally in producing the very fi porosity responsible for the expansion.

Salt solutions as osmotic fluid

Salt solution is normally considered an osmotic fluid. The concentration differenc between the solution in the pores and outside the concrete or aggregate is. considere as the driving force for osmotic expansion. However, adjacent pores within the solid ma contain solutions of different concentration that may act as osmotic centres. Activ surfaces attract ions, and the concentration of ions per volume of fluid in a small pore likely to be greater than in larger pores because of a greater ion to pore volume ratio in th

smaller pore. This is illustrated in Figure 8, part B. If we assume an ion concentration per unit of internal pore surface area to be constant, and the ratio of surface area to volume varies as given in Table 1, then it follows that the smaller pores will have a greater ionic concentration than larger pores, and therefore greater osmotic potential.

DURABILITY RELATED TO OSMOTIC EXPANSION

Wetting-drying deterioration

Wetting a susceptible fine grained material causes it to expand (Figs 13 and 14). Normal vapour pressure water will flow under osmosis into the smaller pores containing lower vapour pressure water (Fig 11) until the internal pressure within the pore balances the osmotic pressure. The expansion of the rock is in proportion to the internal pore pressure generated, and achieves equilibrium as shown in Fig 13. Upon drying, the higher vapour pressure water in larger pores is evaporated first, leaving a lower vapour pressure meniscus in the smaller, critical pores. The tension within the capillary, P_c, is proportional to the curvature of the meniscus (and the minimum radius of the capillary) according to the relationship:

$$P_c = hdg = 2 \cos \boldsymbol{\theta} \; \boldsymbol{\rho} / r \qquad \text{(Eq 3)}$$

where:
h = height of rise in the capillary (tension)
d = density
g = gravitational constant
$\boldsymbol{\rho}$ = surface tension
r = smallest capillary radius
$\boldsymbol{\theta}$ = contact angle

Since only 'r' in the above relationship is variable, the tension varies as the inverse square of the capillary radius. The tension within the capillaries is acting against the tensional strength of the pore walls. The effect is sufficient to account for the negative volume change of the solid when first wetted, and when subsequently dried to the equivalent relative vapour pressure of the remaining capillary water of 045. Beyond this vapour pressure water cannot exist as a fluid. The contraction of the porous solid is seen in Fig 13. It is greater for those solids that contain finer pore sizes and which are also known to be prone to rapid deterioration. Fine grained shaly carbonates, fine siltstones, and shales exhibit this behaviour - rocks that are known to be deleterious when used in concrete.

Freezing-thawing deterioration

Although freezing and thawing is not a problem in the lower latitudes, it is in Canada and the northern states in the USA. It is interesting to note that the rocks containing most of their water in the adsorbed state also are the most prone to failure due to frost. Figur 14 shows the relationship between freezing and thawing deterioration of a variety aggregates (carbonates, granites, volcanics, etc) and the adsorbed water content (perce by weight of sample). Adsorbed water does not freeze readily in the normal freezing rang of the middle latitudes or in the test.. Freezing is a drying process, thus a similar mech nism may be operating as outlined in the previous section.. The connection betwe adsorbed water, isothermal expansion, and freezing-thawing damage is shown in Fig 1 The aggregate with high isothermal expansion on wetting also has high freeze-thaw los

Alkali silica reactivity

The causes of alkali silica reactivity are chemical - the reaction of the alkalis in th concrete with susceptible aggregate.. The alkali-silica reactive aggregates are fine graine silica-rich aggregates with very active surfaces. Chert and opal, the most comm reactive aggregates, both have imperfect silica lattices and are amorphous in nature Some crystalline silicates, such as reactive quartz, have strained crystal lattices (as show by their wavy extinction under petrographic microscope), and are also highly surface acti The alkali-silica reaction is initiated because of the high surface activities of the minera involved. The product of the reaction is silica gel. The gel is also very surface activ imbibes water by osmosis, and swells. It is the swelling that causes concrete distres In concrete containing alkali-reactive aggregate, the relationship between isotherm expansion on wetting and alkali reaction - induced expansion is shown in Fig 16 (8) Th relationship is linear, and the amount of expansion developed under the two conditions also similar.

CONCLUSIONS

The degree of activity of internal pore surfaces, and the pore size determine response of the porous permeable solid to water and to ions in the water. The more act the pore surface, the larger is the size of pore affected by it. Water is adsorbed, a strongly held by the surface. If the lower vapour pressure, adsorbed water fills the sm pore completely, and the pore is exposed to a source of higher vapour pressure water a larger pore, an osmotic flow is initiated from the regions of higher vapour pressure regions of lower vapour pressure. An osmotic equilibrium is established when the pressu inside the pore builds up to sufficient magnitude to resist the flow. The pressure build- acts against the weaker tensile strength of the material, and the material initially expan elastically. Sufficient pore pressure may initiate fracture formation and propagation. drying, the material contracts. Wet-dry cycling can ultimately destroy the material

The dissolved ions (primarily cations, since the surface is negatively charge decreases the vapour pressure of the pore water further, filling larger pores with lo vapour pressure water, and increase the number of pores involved in the osmotic flow

Porous materials in which fine pores predominate are subject to expansion and contraction on wetting and drying, respectively. Repeated cycling may 'fatigue' the solid, causing fracture development

Adsorbed water content, ion content, and the resultant osmotic expansion control the durability of aggregates and concrete under conditions of cyclic wetting and drying and freezing and thawing

Upon closer examination, the activity of the internal pore surface plays a major part in the behaviour of the pore in the presence of water and ions in solution under a variety of climatic, environmental, and chemical environments. The pore behaviour affects the integrity of the solid, and as such, deserves closer scrutiny

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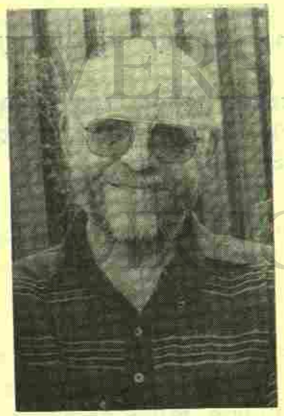
Table 1. Relationship of cube size to surface area.

Edge Cube size cm	Individual Cube Volume cm³	No of smaller cubes in 1 cm volume	Per cubic Surface Area cm²	Total Area of all cubes cm²	Per cubic ratio of area:volume
1	1	1	6	6	6
0.1	0.001	1000	0.06	60	6
0.01	1E-06	1000000	0.0006	600	60
0.001	1E-09	1000000000	6E-06	6000	600
0.0001	1E-12	1E+12	6E-08	60000	6000

Table 2. Relationship of spherical particle size to pore size (cubic packing).

Edge Cube size cm	Individual Cube Volume cm³	Sphere Volume, cm³ d = cube size	Pore Volume cm³	Maximum Pore Radious cm
1	1	5.236E-01	4.764E-01	0.4142
0.1	0.001	5.236E-04	4.764E-04	0.0414
0.01	1E-06	5.236E-07	4.764E-07	0.0041
0.001	1E-09	5.236E-10	4.764E-10	0.0004
0.0001	1E-12	5.236E-13	4.764E-13	0.0000



Peter P. Hudec, Geology Department
University of Windsor, Windsor, Ont
Canada, N9B3P4

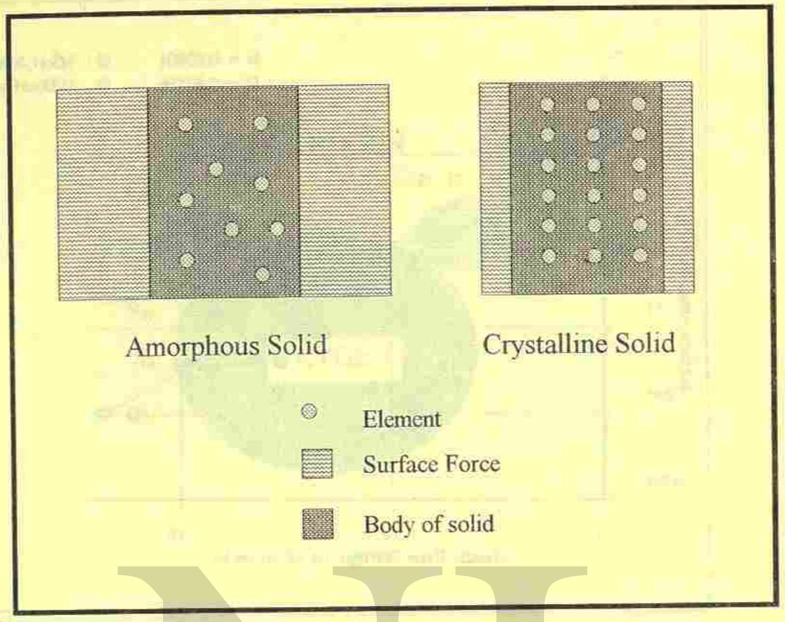


Figure 1 Illustration of surface forces active on amorphous and crystalline solids Surface force is indicated by the width of band adjacent to the surface.

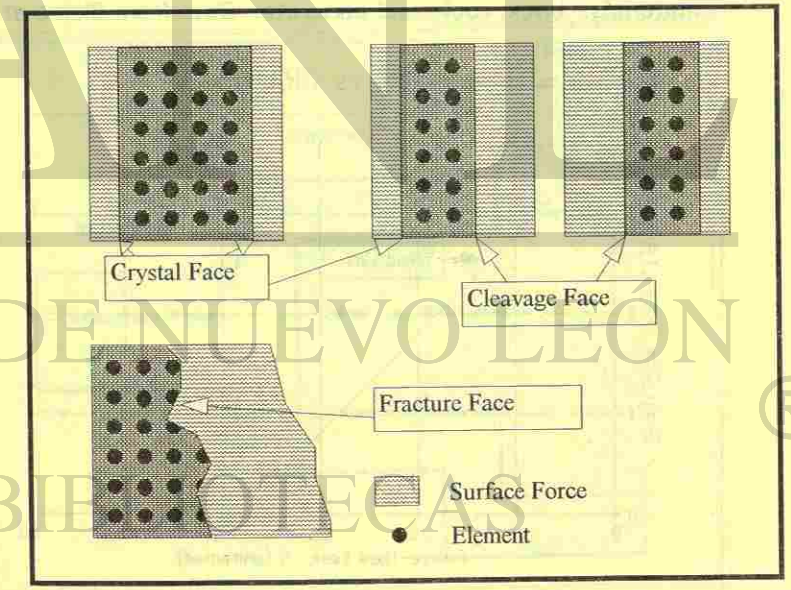


Figure 2 Illustration of surface forces active on crystal faces, cleavage faces, and fracture faces of crystals. Surface force is indicated by the width of band adjacent to the surface.

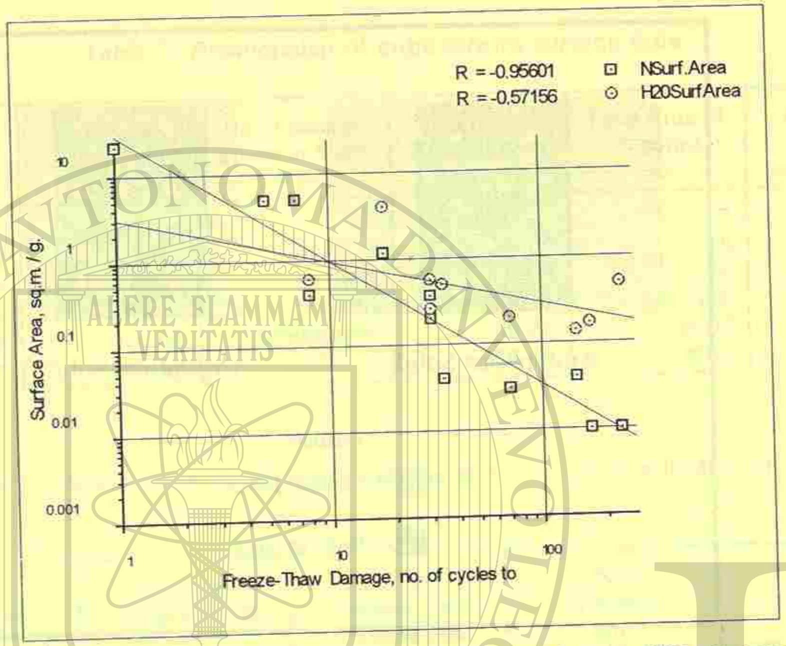


Figure 3 Relationship of surface area as calculated by the BET nitrogen method a water adsorption method and the freeze-thaw damage to various build materials: brick, rock, and concrete. Data from Blaine et al (2).

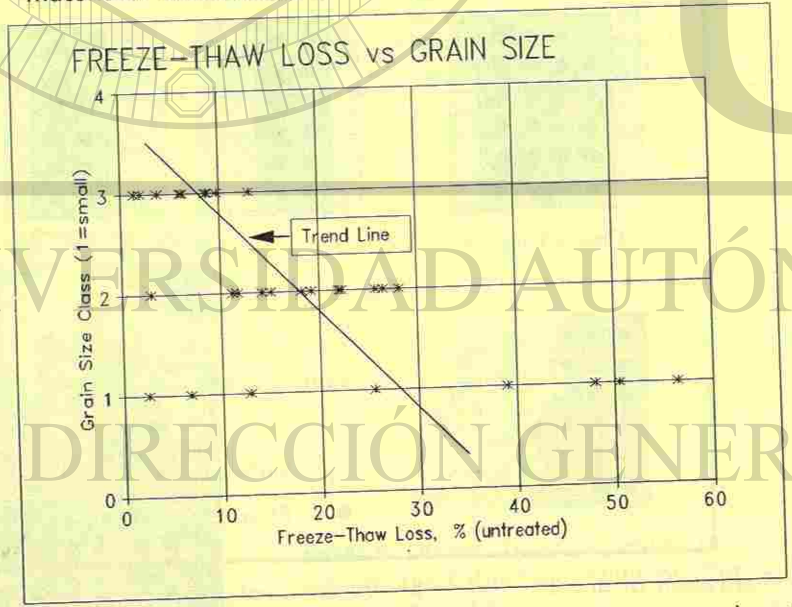


Figure 4 Relationship of grain size estimates (1 = fine, 3 = coarse) and freeze-th durability of aggregate.

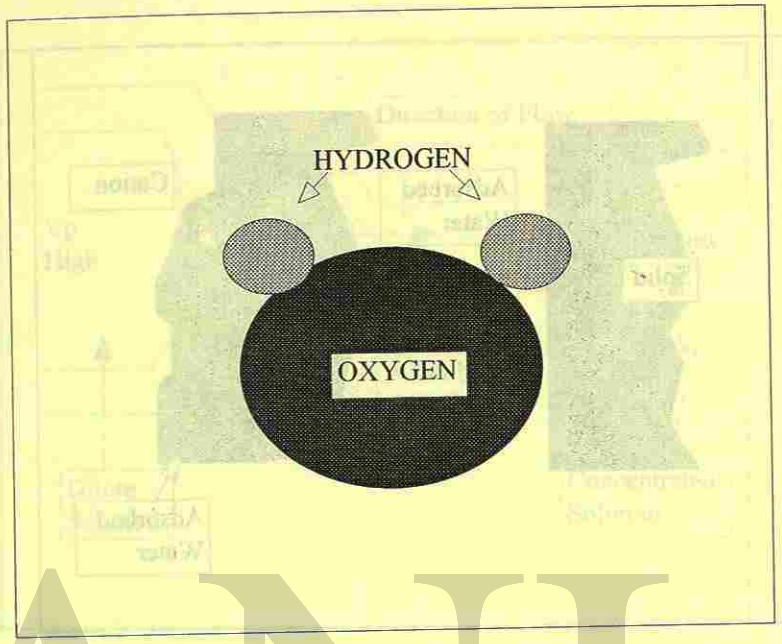


Figure 5 Diagram illustrating the polar nature of water.

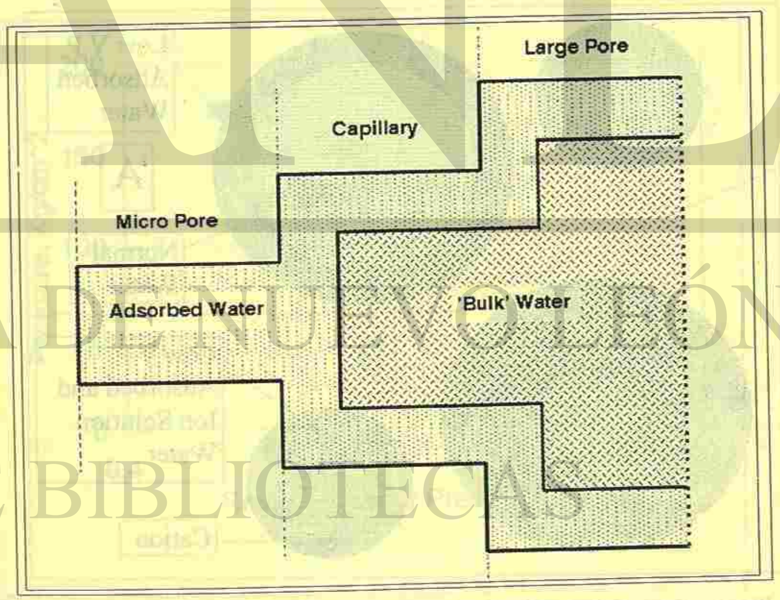


Figure 6 Classification of pore (capillary) size based on proportion of water adsorbed to normal water present in the pore

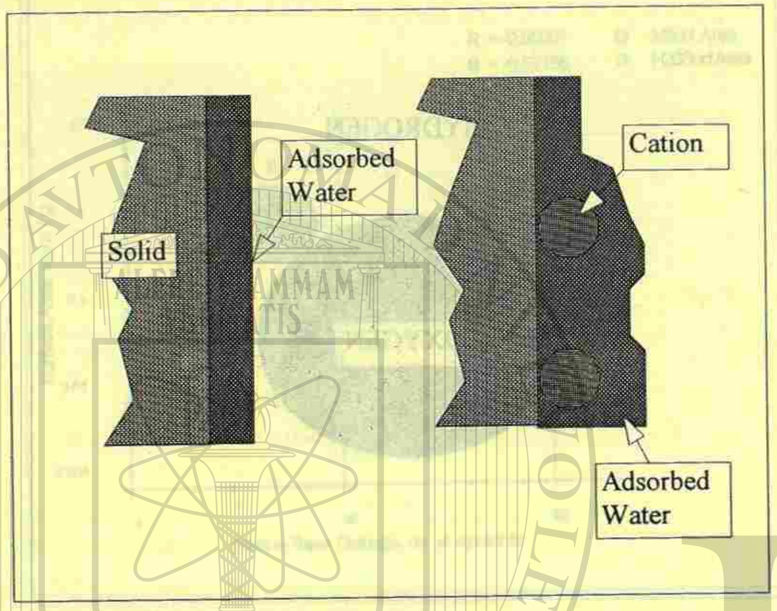


Figure 7 Illustration of the amount of adsorbed water present on surface with an without cations.

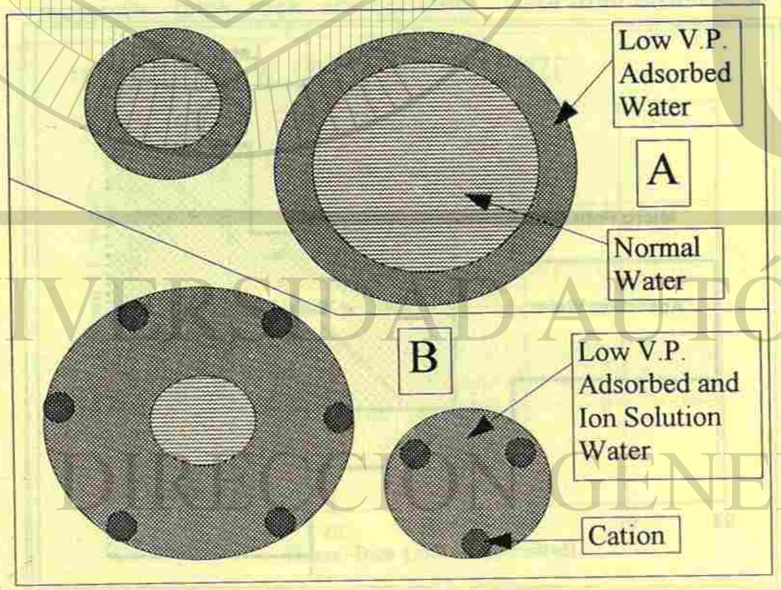


Figure 8 Illustration of the effect of cations on the critical size of pore. Presence o ions in the pore increase the number of pores in the 'critical' range by fillin larger pores with adsorbed water

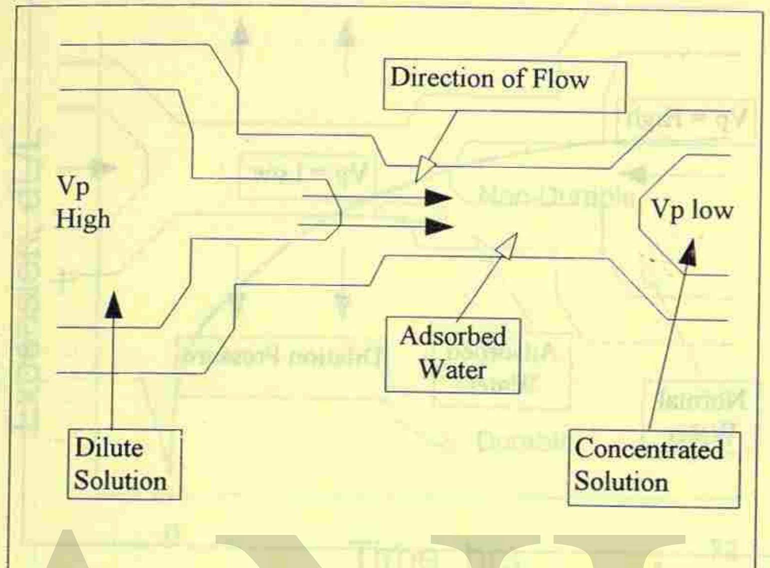


Figure 9 Diagram illustrating the osmotic phenomenon in porous materials. Small pores containing only adsorbed water act as semi-permeable membranes

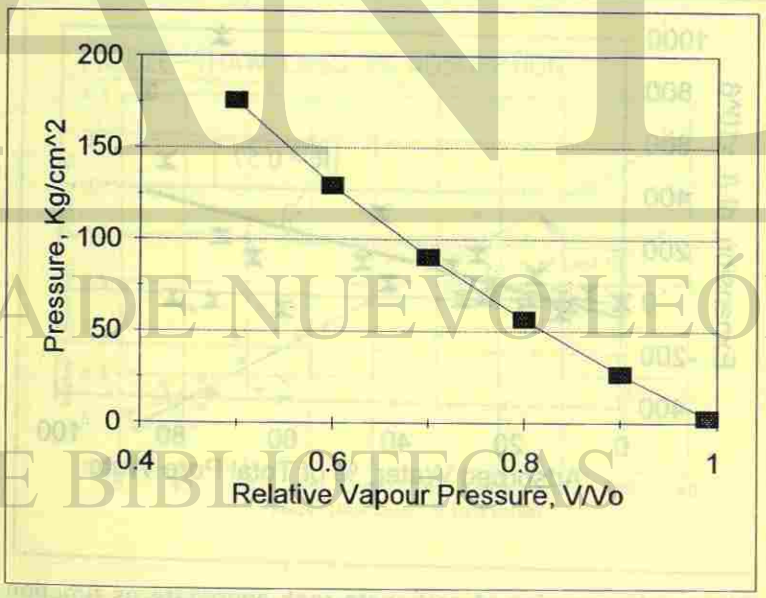


Figure 10 Magnitude of osmotic pressure as function of relative vapour pressure in small pores or ionic solutions

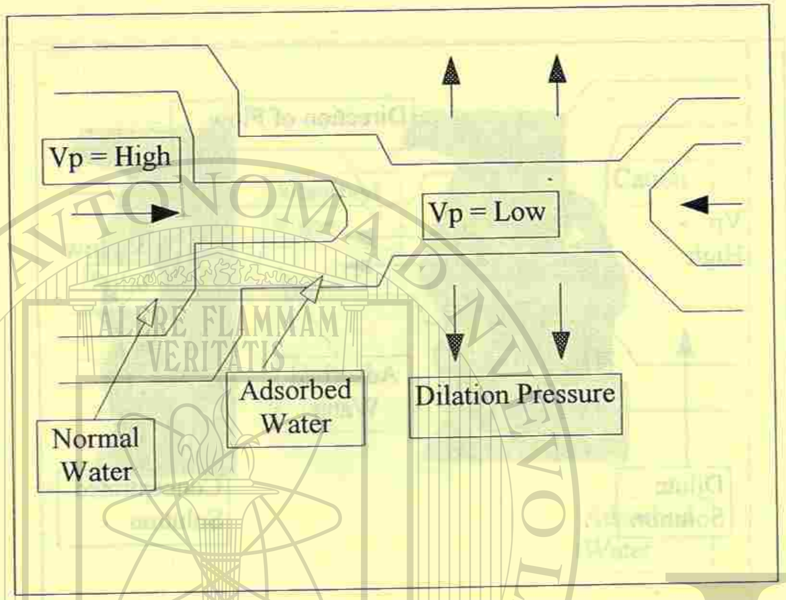


Figure 11 Diagram illustrating the role of adsorbed, low vapour pressure water in s pores as osmotic fluid, and the development of expansive forces withi pore

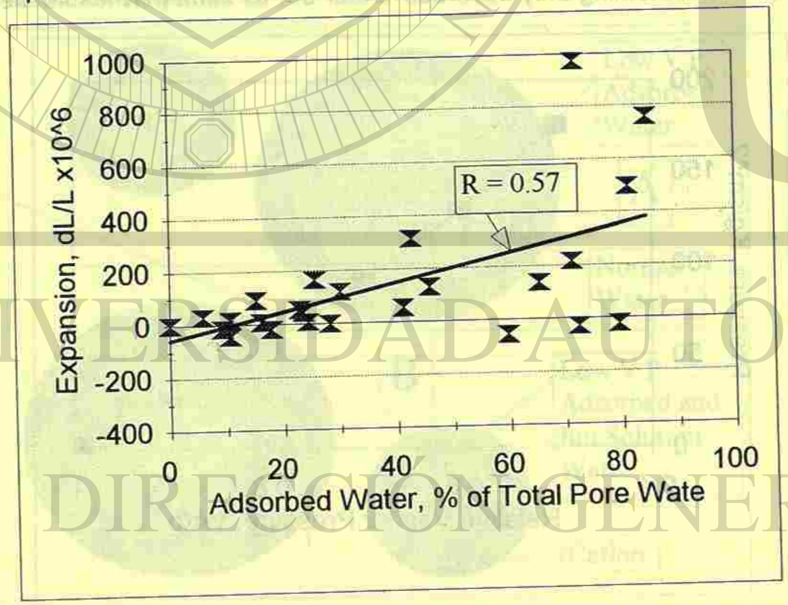


Figure 12 Observed expansion of carbonate rock aggregate as function of adso water content. Adsorbed water is expressed as percent of total water in saturated rock

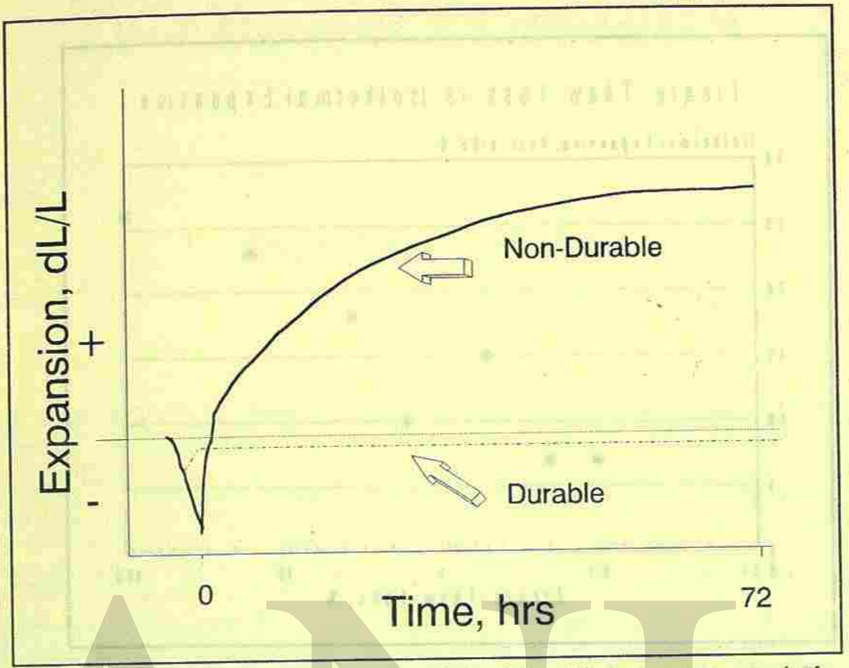


Figure 13 Typical expansion curve on wetting of durable aggregate, and of non-durable aggregate

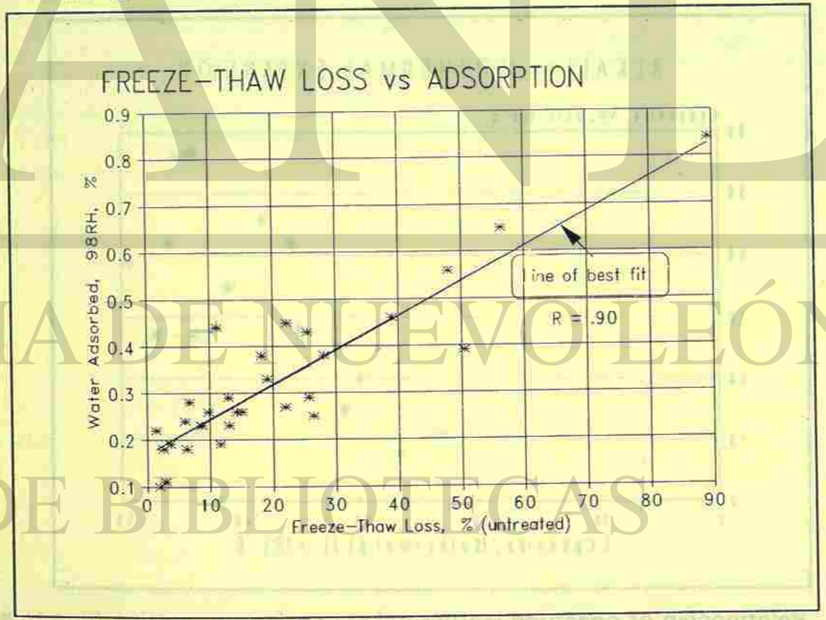


Figure 14 Relationship of adsorbed water content at 98% relative humidity, 20°C (expressed as weight percent) and the freeze-thaw durability of coarse aggregate

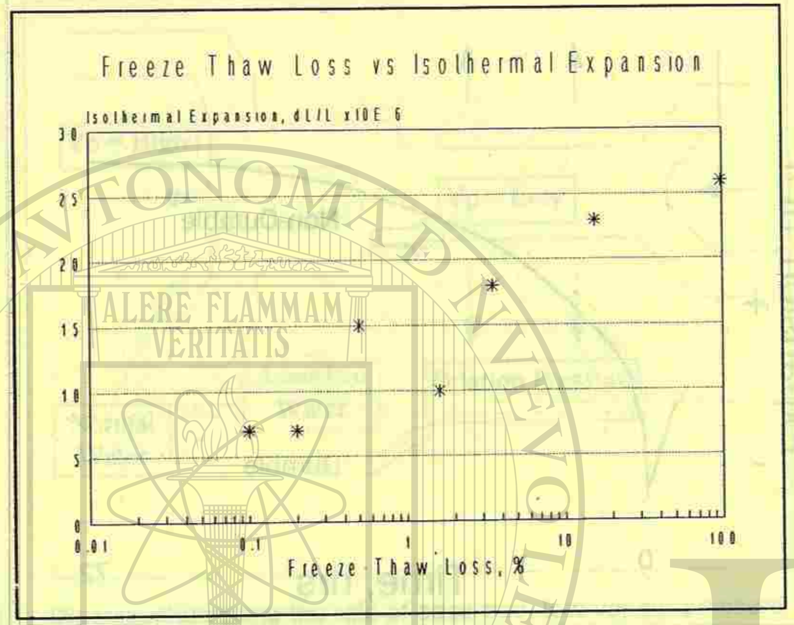


Figure 15 Relationship of observed isothermal expansion on wetting (due to adsorbe water) and the freeze-thaw loss of coarse aggregate

Figure 16 Relationship of observed isothermal expansion on wetting (due to adsorbe water) of coarse aggregate in concrete before testing, and the alkali reactiv expansion observed after AR testing.

ENSAYES PARA DETERMINAR LA REACCION ALCALI-AGREGADO EN EL CONCRETO

por

M.A. Bérubé y B. Fournier

Sinopsis: La información más realista sobre la reacción potencial alcalina de los agregad para concreto es aportada por su comportamiento en las estructuras existentes. S embargo, si dicha información no está disponible o se juzga poco confiable por divers razones, los agregados deben ser ensayados en el laboratorio. Este trabajo revisa y discu los principales métodos de ensaye de la reacción álcali-agregado usados en Norteaméric En el actual estado del conocimiento, sólo algunos métodos son considerad estadísticamente confiables para ser utilizados en la mayor parte de los tipos de agrega para concreto. El examen petrográfico ASTM C-259 es siempre el primer paso a segu El otro método rápido de ensaye recomendado, es el Método Acelerado para Barras Mortero (ASTM C 9 - P 214 ó la propuesta CSA). Esta ensaye no puede ser usado pa rechazar materiales, ya que es severo para un gran número de agregados inocuos, pe representa una poderosa herramienta de selección, ya que a sólo algunos agregad deletéreos no pueden ser detectados. El método de ensaye más realista es el Método d Prisma de Concreto CAN/CSA A23.2-14A. Por consiguiente, se propone un diagrama d decisión para la evaluación de agregados, que está, sin embargo, basado principalmen en la experiencia Canadiense del AAR (reacción álcali-agregado) y puede no s necesariamente aplicable a todos los agregados encontrados en otros países.

Palabras Clave: Métodos de ensaye, método acelerado de ensaye, reacción álca agregado, alcalinos, concreto, durabilidad, expansión, mortero (material).

TESTING FOR ALKALI-AGGREGATE REACTIVITY IN CONCRETE

by

M.A. Bérubé and B. Fournier

Synopsis: The most realistic information on the potential alkali-reactivity of concrete aggregates is provided by their field performance in existing structures. However, if such information is not available or judged not reliable for a number of reasons, the aggregates have to be tested in the laboratory. This paper reviews and discusses the principal testing methods for alkali-aggregate reactivity that are used in North America. At the present state of knowledge, only a few methods are considered statistically dependable enough to be applicable to most types of concrete aggregates. The Petrographic Examination ASTM C 295 is always the first step to do. The only other rapid test method that is recommended is the Accelerated Mortar Bar Method (ASTM C 9 - P 214 or CSA proposal). This test cannot be used for rejecting materials, because it is severe for numerous innocuous aggregates, but remains a powerful screening tool since only a few deleterious aggregates cannot be detected. The most realistic testing method is the Concrete Prism Method CAN/CSA A23.2-14A. Accordingly, a decision chart, which is, however, mainly based on Canadian experience of AAR and could not necessarily apply to all aggregates found in other countries, is proposed for aggregate evaluation.

Keywords: Testing methods; accelerated testing methods; alkali-aggregate reaction; alkalies; cements; concrete; durability; expansion; mortar (material)

M.A. Bérubé is Professor at the Geology and Engineering Geology Department of Lav University (Sainte-Foy, Quebec, Canada, G1K 7P4). He is a member of the CS Sub-Committee A5 on Alkali-Aggregate Reaction.

B. Fournier is research scientist at the Resource Utilization Laboratory of CANMET (4 Booth Street, Ottawa, Ontario, Canada, K1A 0G1). He is a member of the CS Sub-Committee A5 on Alkali-Aggregate Reaction.

INTRODUCTION

Basic concepts -- For many years, aggregates were believed to be essentially inert a chemically inactive in concrete mixtures. It is now well established that the properties aggregates greatly affect the strength, the durability and the structural performance concrete. In concrete, the aggregates are subjected to a highly basic and alkali environment where some mineral phases, generally stable in normal environmen conditions, can produce significant deteriorations as a result of deleterious chemic reactions commonly called alkali-aggregate reactions.

Two types of alkali-aggregate reactions (AAR) are described in the CAN/CS A23.1-M90 (Appendix B) which differ fundamentally in the type of mineral phases and t mechanisms involved (1) (see Table 1): A), alkali-carbonate reaction (ACR), and alkali-silica reaction (ASR). The former involves fine-grained argillaceous dolomi limestones that are mainly found in Ontario (Canada), and in a few states in the USA. T alkali-silica reaction may be subdivided into two categories according to the type reactive silica involved: A1), ASR which occurs with poorly crystalline or metastable sil minerals, and volcanic or artificial glasses, and A2), ASR which occurs with quartz-bear rocks. The latter is presented separately because of the delayed onset of expansion a cracking that can be observed either on concrete prisms tested in the laboratory or concrete structures (which can be up to 20 years).

Three conditions must prevail to initiate and maintain alkali-aggregate reactions concrete: 1), the aggregates must be reactive; 2), the alkalies must remain abundant in t concrete pore solution; usually, these are mostly supplied by the cement, but some m also be provided by chemical or mineral admixtures, by some mineral phases prese within the aggregate particles such as altered feldspar, micas and zeolites, or by seconda sources such as sea-water and deicing salt, and 3), the concrete must be exposed to hi humidity, over 80 to 85% R.H. according to many authors.

Predicting the potential alkali-reactivity of concrete aggregates -- A critical questi arises when planning the construction of a concrete structure to be subjected to conditio that might promote the development of AAR: *Are the proposed aggregates alkali-reacti in concrete?* To answer this question, the most realistic information is provided by the fi

performance of these aggregates in existing structures. This information is only available for aggregate sources frequently exploited in the past, for many years. Indeed, problems related to alkali-aggregate reaction usually arise years after construction. The field survey may also be inconclusive for other reasons, such as: 1), difficulty in identifying a number of sufficiently old and/or severely exposed structures built with the aggregates from the source under study, which is often due to insufficient information in the construction files about the aggregate sources used; 2), lack of information on parameters that affect AAR and the durability of concrete, such as the alkali content of the cement, the cement content, the other concrete constituents and the curing methods used; 3), variations in exposure conditions from one structure to another, e.g., availability of moisture, freezing/thawing, wetting/ drying, sea-water and deicing salts, and 4), variations in the composition of aggregates produced by the source under study between the construction period and the survey of structures, such as changes in the exploitation levels or zones, and modifications in the methods of exploitation or preparation. For instance, a structure that contains very reactive aggregates might not have suffered from AAR if a low alkali cement or effective supplementary cementing materials in sufficient amounts had been used in concrete.

Because of the reasons mentioned above, the actual determination of the potential alkali-reactivity of aggregates is often only possible through laboratory testing programs. An engineering judgement is then necessary to predict the risk for deleterious expansion with the aggregate investigated, based on the laboratory results, the known limitations of the testing methods used, the past field performance of concrete aggregates that are quite similar to the one under study, and the conditions to which the particular structure to built will be subjected.

The daily reality in the construction industry is also that, on many occasions, the aggregates must be evaluated within a very short period of time; this calls for testing methods that are rapid, reliable, simple, and reproducible. This paper reviews the principal testing methods for AAR, standardized or not, that are commonly being used in North America. These methods are listed in Table 2 with the types of AAR for which they can be used (ACR and/or ASR). Many other methods used in other countries are reviewed and discussed in (2).

First, it must be pointed that all these methods are accelerated tests, even the CSA Concrete Prism Method, which requires one year, and that they cannot exactly reproduce field exposure conditions. Indeed, they all try to predict in less than one year what may happen in the field after five, ten, twenty years or even more. To achieve this goal, one or many of the following test conditions are generally used: 1), increase the alkali concentration to which the aggregate is subjected using high-alkali mixtures or immersing the samples in alkaline solutions; 2), store the test samples at high temperature, for example 38°C, 80°C, and even more in autoclave treatments; 3), subject the samples to high pressure as in the autoclave; 4), subject the samples to high humidity environments such as 100% R.H. or immersing them in aqueous solutions, and/or 5), increase the specific area in reducing the aggregate samples to powder or sand size.

PETROGRAPHIC EXAMINATION ASTM C 295

Potentially reactive mineral phases and corresponding host rock types are listed Table 1. The petrographic examination of aggregates in thin sections under the opti microscope generally allows recognition of these potentially mineral phases or roc Techniques such as X-Ray diffraction, measurement of the undulatory extinction angle quartz grains, and scanning electron microscopy might be also useful (ASTM C 295). L us mention that the effectiveness of the undulatory extinction angle method is presen questioned (3). When conducted by a petrographer with experience of AAR, t petrographic examination can sometimes be sufficient for accepting or rejecting aggrega for use in concrete, in accordance with the past field performance of petrographica similar aggregates. If there is doubt, the petrographic examination will help selecting t additional tests to be performed, considering the nature of the aggregates under stud thus preventing poor choices and reducing the amount of work. Indeed, some test methods are not capable of detecting some deleterious aggregates, while being too seve for innocuous ones. In fact, the petrographic examination must always be performed pri to or in parallel to any other quality control test.

CHEMICAL METHODS

Chemical Method CSA Proposal A23.2-26A

This method, which is in the process of being adopted in the Canadian standards f AAR (1), covers the evaluation of the potential alkali-carbonate reactivity of quarr carbonate rocks. In this test method, a representative sample of the aggregate to be test is reduced using a small jaw crusher and a disk pulverizer such as to pass a 160 μm siev The material is then carefully homogenized and sent for chemical analysis for CaO, Mg and Al_2O_3. The results thus obtained are plot on a graph showing zones associated w "aggregates considered non-expansive" and "aggregates considered potentially expansiv (Fig. 1). Aggregates for which the results in this test are falling in the "consider potentially expansive" zone should be considered as such until their innocuous charact is demonstrated by either satisfactory service record or concrete prism test results.

Chemical Method ASTM C 289

The Chemical Method ASTM C 289 is certainly one of the most widely used tests f evaluating the potential reactivity of silica-bearing aggregates. Its popularity is mai because it requires only small quantities of material and results can be obtained within few days. In this test, 25 g sub-samples of crushed aggregate particles, 150-300 μm size, are immersed in 25 mL 1N NaOH solution at 80°C for 24 hours. The solution is the filtered and analyzed for dissolved silica (Sc) and reduction in alkalinity (Rc). The resu are plotted on a standard chart showing three fields corresponding to innocuou deleterious and potentially deleterious aggregates (Fig. 2).

This test is applicable to alkali-silica reactivity only. However, a significant number of known alkali-silica reactive aggregates from all over the world pass the test, while other aggregates with good field performance fail (2). There are several reasons for this:

Poor representativity of the aggregates used to design the test -- The original ASTM decision chart was based on expansion of mortar bars, field performance or petrographic examination, in some cases, of 71 rocks from USA containing very reactive siliceous mineral phases such as opal, calcedony and volcanic glass. However, the increased application of the chemical test to aggregates from various countries led to the conclusion that the original chart is not universally applicable (4).

Mineral interference -- Mineral phases such as calcium, magnesium and iron carbonates, hydrated magnesium silicates, gypsum, zeolites, clay minerals, organic matter and iron oxides have proved to create interferences which result in: 1), underestimated Sc values due to precipitation of silica or interference during chemical analysis, or 2), overestimated Rc values due to reactions with Na^+ and OH ions (5). In particular, calcium carbonate leads to precipitation of some of the silica dissolved as CSH, thus causing the acceptance of several reactive aggregates (6) (Fig. 2). A modified version of the Chemical Method ASTM C 289, which consists in performing the test procedure on the dried insoluble residue, 0 to 300 μm in size, of the aggregate to be investigated, has recently been proposed to overcome carbonate interference (6,7) This method, despite producing interesting information on the basic chemical stability of the insoluble residue of the investigated aggregates in an alkaline solution, showed only limited success in differentiating potentially reactive from non-reactive aggregates.

Crushing and sieving effects -- During the processes of crushing and sieving of the aggregate sample to obtain the material required for the test, 150 - 300 μm in size, much of the reactive phases may be taken out from the test specimens when sieving to discard the < 150 μm fraction, for instance the reactive quartzitic cement around the non-reactive quartz grains in some Potsdam sandstones (2).

Conclusion -- The past experience has shown that the Chemical Method ASTM C 289 is severe for a number of innocuous aggregates, while being not severe enough for many deleterious ones. Moreover, modified procedures or limit criteria based on regional geological considerations were assessed with either limited or good success. In Canada, this test is not used anymore by many agencies and has been discarded of the new proposed version of the CSA Standards for AAR (1).

MORTAR BAR METHODS

Mortar Bar Method ASTM C 227

Mortar bars are made with the aggregate meeting specific grading requirements and a cement with the highest alkali content representative of the general use intended, or available in the laboratory making the tests. The ratio of cement to graded aggregate 2.25. The amount of mixing water is adjusted such as to get a specified flow. The ba 25 x 25 x 285 mm in size, are kept in their moulds for 24 hours at 23°C and then stor at 38°C and 100% R.H., in sealed specified containers. Length change measurements a made at frequent intervals. The expansion limits are 0.10% at 6 months, or 0.05% at months. According to Grattan-Bellew (8), these limits should be reported to 12 and months, respectively. Such limits are already being used by many agencies.

The test does not apply to alkali-carbonate reactive aggregates, and also proved to not capable of detecting many slow-late expanding alkali-silica/silicate reactive aggregat in particular greywackes and argillites (9), as well as all the reactive aggregates in UK (1 A number of parameters have shown to greatly affect the results:

Effect of container and wicking -- The test is largely affected by the presence or n of wicks inside the storage container (11) (Fig. 3). With containers with wicks, for instan the proposed reference ASTM container, a large number of reactive aggregates satisfy t test requirements due to excessive leaching of alkalies from the mortar bars, thus leadi to lower expansions (11). As a result, numerous tests performed in the past and fr which it was concluded that the aggregates were innocuous are doubtful if the bars ha been stored in containers with wicks inside.

Effect of alkali content -- The variations permitted in the alkali content of the ceme for the test may also explain some of the experimental variations observed in the past (F 4). A current practice in many Canadian laboratories consists in adding NaOH to t mixture water so as to increase the alkali content to 1.25% (Na_2O eq.) by mass of ceme

Effect of water/cement -- No water/cement is specified in the mortar bar meth ASTM C 227 as water is added to reach a specified flow. However, it has been observ that variations in the w/c may significantly affect the expansion results (Fig. 5). T behaviour might be attributed to the smaller quantity of "free" pore water in lower w mixtures, which results in higher alkali concentrations in the pore solution. The low porosity of low w/c mixtures also offers less room for stocking the deleterious reacti products, while such mixtures are possibly less susceptible to alkali leaching because a relatively lower permeability.

Conclusion -- In Canada, the CSA Concrete Prism Method, which does not take mu longer, while considered much more reliable, is preferred to the Mortar Bar Method AS C 227. However, when this mortar test has to be performed, it is highly recommende 1), to use a container without wicking; 2), to increase the alkali content to 1.25% (Na eq.) of the cement mass by adding NaOH to the mixture water; 3), to control t water/cement to 0.50 (0.44 for uncrushed natural sands), and 4), to test in paralle well-known (or reference) reactive aggregate. The above recommendations concerning t storage conditions and the alkali content have been taken into account in the rece development of the French equivalent Mortar Bar Method AFNOR P 18-585.

Accelerated Mortar Bar Method (ASTM C 9 - P 214 & CSA Proposals)

Mortar bars for this test are prepared in accordance with ASTM C 227. After 24 hours of initial moist curing in the molds, the bars are placed in a sealed plastic container filled with water at 23°C and the containers immediately placed in an oven stove at 80°C. The next day, the zero reading is taken and the bars transferred to a 1N NaOH solution at 80°C for two weeks (12 days in the original NBRI proposal (12)), and measured hot each working day. In this test, expansion of mortar bars generally increases when increasing the w/c (2,8,13). The ASTM C 9 - P 214 (14) and CSA (1) proposals specify using a fixed w/c of 0.50 for coarse aggregates and manufactured sands, and 0.44 for natural sands.

Expansion limit criteria and test performance -- There is a general agreement that aggregates which expand less than 0.10% after 14 (or 12) days be considered innocuous (Table 3). Indeed, up to now, only a few deleteriously reactive aggregates were found to satisfy this quite severe criterion, including alkali-carbonate reactive rocks from the Kingston area (Ontario, Canada) (15), some Potsdam sandstones from the Montreal area (Quebec, Canada) (2,16) (Fig. 6), some granite and gneisses of Grenville age from Maryland and Virginia (USA) (17), and a particular phyllite from Australia (18). As shown in Table 3, less or more severe criteria are also used for aggregate acceptance by other workers or agencies. In some cases, different limit criteria are proposed according to the type of aggregate tested.

On the other hand, there is also a general agreement that all above criteria are severe for numerous aggregates with good field performance (2), and that materials that exceed the proposed limits should require further testing. This is clearly evidenced by the results obtained at Laval University on a number of quarried silicate and carbonate aggregates from Quebec (2,16) (see Figs. 6 and 7).

Conclusion -- The Accelerated Mortar Bar Method should be used with care for rejecting aggregates. Indeed, many innocuous aggregates that perform well in the field and/or in other laboratory tests on mortar or concrete specimens, have been reported to have failed this accelerated test when using limits of 0.10% or 0.15% expansion at 14 days. Then further testing is required for aggregates exceeding the proposed limits. Nevertheless, this test remains a useful screening tool as it is capable of recognizing within two weeks most deleterious aggregates, while correctly recognizing a high proportion of innocuous aggregates. This test method has been applied to a large number of aggregates in several countries. In addition, its precision has also been assessed by several workers (8,13,19,20), with very satisfactory results.

CONCRETE PRISM METHODS

Concrete Prism Method CAN/CSA-A23.2-14A (Current Procedure)

Three concrete prisms, not less than 75 x 75 x 300 mm and not more than 120 x1 x 450 mm in size, are made with the coarse aggregate under study, a non-reactive sa and a normal Portland cement containing between 0.8% and 1.2% alkalies (Na_2O eq.). T specified cement content is 310 kg/m^3 and the amount of mixture water is adjusted s as to give a slump of 80 ± 10 mm. Since 1986, the test method requires the total al content being raised to 1.25% (Na_2O eq.) by mass of cement, by adding NaOH to mixture water. This corresponds to a total alkali content of 3.88 kg/m^3 of concrete. T prisms are stored either in a moist curing room at 23°C or above water in sea containers at 38°C. Length change measurements are made periodically. The contain stored at 38°C shall be taken out of the high temperature storage condition 16 ± 4 ho before measurement. The test prism are then immersed for 30 minutes in a water bat 23°C before measurement. The 23°C storage condition was first proposed for detect the alkali-carbonate reactivity, while the so-called "accelerated" version at 38°C currently being used for detecting the alkali-silica reactivity. The expansion at one y should not exceed 0.025% at 23°C or 0.04% at 38°C.

Influence of mixture proportionings (w/c, coarse aggregate/sand) -- According to standard, water is added to the concrete mixture to give the specified flow, whatever water/cement. However, similarly to that observed in the Mortar Bar Method ASTM C2 (Fig. 5), variations in the w/c may have a significant effect on the expansion process For instance, a lower ratio normally leads to a higher strength, a lower permeability a lower porosity, but in turn to a higher alkali concentration in the "free" pore water, to less space for stocking the expansive reaction products. A number of parameters affect the water demand in plain concrete mixtures are not sufficiently well controlle the current procedure, such as the particle shape, which may be greatly influenced by type of crusher used to prepare the aggregates, and the ratio between coarse and f aggregates, which can vary between 50 : 50 and 65 : 35, thus causing also variati in the amount of reactive aggregate particles. In practice, most laboratories which perfo the CSA concrete prism test on a routine basis use to fix the coarse/fine aggregate and w/c ratio.

Influence of storage conditions -- According to Rogers & Hooton (11,21), significa amounts of alkali are progressively leached from the test concrete prisms, with the amo of expansion obtained in the long term being related to the amount of alkalies remain in the prisms. For instance, after 130 weeks of testing, specimens made with alkali-carbonate reactive aggregate from Ontario (Canada) suffered 63% alkali leaching 23°C (in moist curing room), and 42% at 38°C (in plastic pails with wicks). A numbe experiments have shown that the one-year expansion of concrete prisms stored abo water at 38°C, in sealed containers, was similar or even lower when the wicks removed from the containers (8,22), while lowered by about 15 to 20% when concrete specimens are immersed in water 30 minutes before each measurement

specified in the current CSA standard), compared with no immersion (22), as a result of alkali dilution in water. Leaching of alkalies from the concrete prisms has been mentioned as one of the possible factor to explain why the expansion curves are flattening out after a few months of testing.

Effect of cement content -- The current CSA concrete prism method can detect a wide variety of reactive aggregates, with the exception of a number of slowly-reactive quartz-bearing aggregates such as some greywackes, argillites, quartzwackes, quartzites, phyllites, arkoses, sandstones, and granites that are found in gravel deposits or exploited in a number of quarries in Eastern Canada (2). According to Rogers (21,23), in order to recognize these slowly-reactive aggregates, the test should be modified to prevent excessive alkali-leaching, for instance by storing the prisms in sealed plastic bags, or to compensate for this phenomenon by adding more alkalies and/or using more cement in the concrete mixture.

Experimental variations (reproducibility) -- A multilaboratory study using an alkali-carbonate reactive aggregate from Ontario and which involved twenty different laboratories suggested that the coefficient of variation for the CSA Concrete Prism Test conducted at 38°C is about 23% (21), which is quite high.

Conclusion on the current test method -- Despite of all the above limitations, the current CSA concrete prism test is still considered in Canada as the most realistic method used for evaluating the alkali-reactivity potential of concrete aggregates in the laboratory. Indeed, the test is capable of recognizing most reactive aggregates except a number of slow-late expanding ones for which a higher cement content around 410 kg/m^3 content seems to be required (15).

However, in our opinion, testing together the coarse aggregate with the sand used in AAR affected concrete structures incorporating such slowly reactive aggregates might have allowed detection of many of them, since the sands used often presented composition and potential reactivity that are quite similar to those of the corresponding coarse aggregates. For instance, the 1-year expansion obtained with the current CSA procedure for a coarse greywacke that is very similar to the one used in the AAR affected Mactaquac dam, in New Brunswick (Canada), was 0.034% when using a well-known non reactive sand, but increased to 0.066% in presence of a sand that was very similar to the one used in the dam (24).

New Proposed Concrete Prism Method CAN/CSA-A23.2-14A

In response to the various problems that have been progressively identified with the current CSA Concrete Prism Test, a CSA Task Group recently proposed a number of modifications to the current test method (1). The procedure proposed, which is in the final steps of the process for being introduced in the Canadian Standards, includes the following modifications:

1) The cement content is increased to 420 kg/m³, still adjusting the alkali content 1.25% (Na_2O eq.) by mass of cement by adding NaOH to the mixture water, to g a concrete alkali content of 5.25 kg/m³.
2) The cement used for the test is a normal Portland cement with an alkali content of ± 0.1% (Na_2O eq.), compared to ± 0.2% in the current standard.
3) The effective water/cement is controlled between 0.42 to 0.45.
4) The proportion between coarse and fine aggregates is fixed to 60/40 (with so exceptions for high- and low-density aggregates).
5) The cross section of the concrete prisms is fixed at 75 x 75 mm with the possib for their length to vary between 275 and 405 mm.
6) The test is always performed at 38°C (even when testing for alkali-carbon reactivity).
7) The 30-minute immersion in water at 23°C before measurement is taken out.
8) The storage container is a 22 to 25-litre plastic pail with air tight lid; a wick absorbent material is still placed around the inside wall of the container. It is poss to use an alternative storage container at certain conditions.

The test can be used for evaluating the potential alkali-reactivity of coarse or f aggregates, using a non-reactive fine or coarse aggregate, respectively, that expands le than 0.1% in the Accelerated Mortar Bar Method ASTM C 9 - Proposal - P 214, or le than 0.015% in the CSA Concrete Prism Method (new procedure). A 1-year, 0.04 expansion limit is still used for acceptance. In our opinion, the test could be also used testing coarse-fine aggregate combinations.

__Conclusion on the new proposed test procedure__ -- The new procedure proposed sho allow detection of most slowly-expanding reactive aggregates which the current t procedure failed in recognizing. However, based on Quebec experience and at least concretes made with cement contents of less than 375 to 385 kg/m³, the method mi be severe for many innocuous or marginally reactive aggregates, in particular for natu and quarried carbonate aggregates. Indeed, half of all representative gravels from Queb that have been tested at Laval University, using a cement content of 410 kg/m³, expand more than 0.04% after 1 year, while just a few of them are considered to be deleteriou reactive in the field (25). In another study on representative quarried carbonate aggrega from Quebec, in which the concrete prisms were made with a significantly lower cem content (e.g. 350 kg/m³) than the one proposed, it was necessary to adopt a less sev criterion of 0.06% after 1 year to distinguish between reactive and non-react aggregates, based on known field performance of the aggregates tested (see Figs. 7 a 9).

__Accelerated Concrete Prism Methods in 1N NaOH solution__

The CSA Concrete Prism Method requires one year to conclude, and is therefore satisfactory in many situations. A number of accelerated concrete methods have been th proposed to accelerate the process of aggregate characterization, using autoclaving immersion in alkaline solution at high temperature for most of them. These methods are discussed elsewhere (2).

__Immersion in 1N NaOH at 38°C__ -- In a study sponsored by the Canadian Electrical Association (CEA) on mass concrete structures affected by AAR (26), a number of concrete test procedures were investigated, among which: 1), the current CSA Concrete Prism Method; 2), a procedure that is very similar to the new proposed CSA concrete prism method described before, and 3), a procedure consisting in immersing concrete prisms made in accordance with the current CSA procedure, but tested in 1N NaOH at 38°C. The latter procedure looked very promising for several reasons:

1) The current CSA Concrete Prism Method failed in detecting 4 of the 9 reactive aggregates tested in that particular study (Fig. 8), in addition to the slowly expanding aggregates mentioned before.
2) The new proposed CSA Concrete Prism Method was not able to detect one of the reactive aggregates tested (a natural lithic gravel from Alberta) (Fig. 8), while being severe for many other natural gravels and quarried carbonate aggregates from Quebec, as mentioned before.
3) In the meantime, testing CSA concrete in 1N NaOH at 38°C properly classified all aggregates tested in this study, 9 reactive and 2 non-reactive, according to a 6-month, 0.04% expansion limit criterion (Fig. 8).
4) The latter method allowed the best differentiation between aggregates.
5) It showed the lowest variability between companion prisms tested together.
6) The specimens are not subjected to alkali leaching during the tests.
7) The results can be obtained in 6 months, using the usual 0.04% expansion limit, compared to 1 year for the two other procedures.

__Immersion in 1N NaOH at 80°C__ -- In Quebec, concrete test prisms are also made in accordance with the CSA Concrete Prism Method and tested in 1N NaOH solution at 80°C (2,26) with the expectation to obtain more realistic results, because testing concrete specimens, than with the corresponding accelerated method on mortar specimens (ASTM C 9 - Proposal - P 214). A 24-day, 0.04% expansion limit criterion is suggested for aggregate acceptance in this so-called "Accelerated Concrete Prism Method". Up to now, the test has been performed on a large number of aggregates of various types and compositions proving to be severe for numerous innocuous or presumably innocuous aggregates (2), particularly for a number of quarried carbonate aggregates (Fig. 9), natural gravels (25) and other aggregates with a granitic composition (26).

CONCLUSION

In Canada, a number of testing methods proved to be not reliable enough to be adopted as routine tests, such as the Chemical Method ASTM C 289, the Mortar Bar Method ASTM C 227 and the Accelerated Concrete Prism Method performed at 80°C in 1N NaOH, while others are very promising, for instance the Concrete Prism Method performed at

38°C in 1N NaOH. At the present state of knowledge, only three methods are consider applicable to most concrete aggregates: 1), the Petrographic Examination ASTM C 29 2), the Accelerated Mortar Bar Method (ASTM C 9 - P 214 or CSA proposal), and 3), t Concrete Prism Method CSA A23.2-14A. Accordingly, the decision chart shown on F 10, which is, however, mainly based on Canadian experience of AAR and could n necessarily apply to all aggregates found in other countries, has been included in the ne proposed version of the CSA Standards for AAR (1).

However, as shown on this figure, the most realistic information on the potent alkali-reactivity of concrete aggregates is provided by their field performance in exist structures. If such information is not available or judged insufficient for a number reasons, the aggregates then have to be tested in the laboratory. In such situations, must be highly emphasized that the Petrographic Examination is always the first step do. It can be used to accept or even to reject the aggregate under study, or at least select the most appropriate test methods to run, in order to prevent poor choices a reduce the amount of work. Indeed, some testing methods are not capable of detect some deleterious aggregates, while being too severe for innocuous ones. The only oth rapid testing method that is statistically dependable enough and more highly recommend as a routine test is the Accelerated Mortar Bar Method. This method cannot be used rejecting materials because it is severe for numerous innocuous aggregates. However remains a very powerful screening tool since only a few deleterious aggregates cannot detected, which can be readily recognized in the Petrographic Examination. The curr CSA Concrete Prism Method is considered as the most realistic testing method determining the potential reactivity of concrete aggregates except for a number of slow expanding aggregates which, however, should be easily detected using the new propos CSA Concrete Prism Method.

ACKNOWLEDGEMENTS

The National Science and Engineering Research Council of Canada (NSERC), the Fon pour la Formation des Chercheurs et l'Aide à la Recherche of Quebec (FCAR), and t Ministry of Transportation of Quebec are gratefully acknowledged for financial suppo

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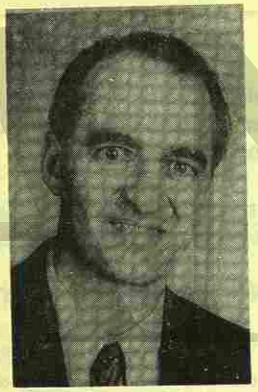

M.A. Bérubé, Professor at the Geology and Engineering Department of Laval University Sainte-Foy, Quebec, Canada, G1K 7P4.

Table 1 -- Mineral Phases and Corresponding Rocks Susceptible to Deleterious Alkali-Aggregate Reactions in Concrete.

A. ALKALI-REACTIVE SILICA MINERALS AND ROCKS

A_1. Alkali-Reactive Poorly Crystalline or Metastable Silica Minerals, and Volcanic or Artifi Glasses (Classical Alkali-Silica Reaction).

Reactants: Opal, tridymite, cristobalite; acid, intermediate, and basic volcanic glasses; artifi glasses, beekite.

Rocks: Rock-types containing opal such as shales, sandstones, silicified carbonate roc some cherts, flints, and diatomite.

Vitrophyric volcanic rocks: acid, intermediate and basic, such as rhyolites, dacit latites, andesites and their tuffs, perlites, obsidians; all varieties with a gla groundmass; some basalts.

A_2. Alkali-Reactive Quartz-Bearing Rocks.

Reactants: Chalcedony; cryptocrystalline to microcrystalline quartz; quartz with deformed crys lattice, rich in inclusions, intensively fractured or granulated; poorly crystalline qua at grains boundaries; quartz cement overgrowths (in sandstones).

Rocks: Cherts, flints, quartz veins, quartzites, quartz-arenites, quartzitic sandstones wh contain microcrystalline to cryptocrystalline quartz and/or chalcedony.

Volcanic rocks such as A_1 but with devitrified, cryptocrystalline to microcrystal groundmass.

Micro-granular to macro-granular silicate rocks of various origins which cont microcrystalline to cryptocrystalline quartz:

- Metamorphic rocks: Gneisses, quartz-mica schists, quartzites, hornfels phyllites, argillites, slates.
- Igneous rocks: Granites, granodiorites, charnockites.
- Sedimentary rocks: Sandstones, greywackes, siltstones, shales, silice limestones, arenites, arkoses.

Sedimentary rocks (sandstones) with epitaxic quartz cement overgrowths.

B. ALKALI-REACTIVE CARBONATE ROCKS

Reactants: Dolomite (dedolomitization process) and active clay minerals (illite) exposed dedolomitization process.

Rocks: Argillaceous dolomitic limestones, argillaceous calcitic dolostones, quartz-bear argillaceous rocks, calcitic dolostones.

Table 2 -- Testing Methods Commonly Used in North America for AAR.

TESTING METHOD	Used ASR	For ACR
Petrographic Examination (ASTM C 295) (≥ 1 d)	X	X
Chemical Methods		
● Chemical Method CSA Proposal A23.2-26A (1-2 d)	X	
● Chemical Method ASTM C 289 (2-3 d)	X	
Mortar Bar Methods		
● Mortar Bar method ASTM C 227 (6 m)	X	
● Proposed Accelerated Method ASTM C 9 - P 214 or CSA (2 w)	X	
Concrete Prism Methods		
● Concrete Prism Method CAN/CSA A23.2-14A (1 y)	X	X
● Proposed method in 1N NaOH at 38°C (used in Canada) (6 m)	X	?
● Accelerated Method in 1N NaOH at 80°C (used in Canada) (1 m)	X	?

ASR: Alkali-silica reactivity. ACR: alkali-carbonate reactivity.

Table 3 -- Expansion Limit Criteria Proposed for Aggregate Acceptance with the Accelerated Mortar Bar Method.

Reference	Aggregate type	Criterion for acceptance	Remarks
Davies & Oberholster (12) (South Africa))	All types	<0.10% - 12 d	0.10-0.25%: sl. exp.; >0.25%: rapidly exp.
Shayan et al. (18) (Australia)	All types	<0.22% - 22 d	<0.10%/10d & >0.10%/22d: sl. exp.; >0.10%/10d: rap. exp.
Grattan-Bellew (15) (Canada)	Siliceous limestones Greywackes, argilites Others	<0.10% - 14 d <0.20% - 14 d <0.15% - 14 d	
Bérubé et al. (16) (Canada)	Quarried aggregates Natural sands & gravels	<0.10% - 14 d <0.20% - 14 d	
Hooton & Rogers (17) (Canada)	All types	<0.15% - 14 d or <0.33% - 28 d	
ASTM C 9 - Proposal -P 214 (14) (U.S.A.)	All types	<0.10% - 14 d	0.10-0.2%: sl. exp.; >0.20%: rapidly exp.
CSA Proposal (1) (Canada)	All types	<0.15% - 14 d	Indications in the standard regarding this limit which might be too severe for some aggregates while not enough for others

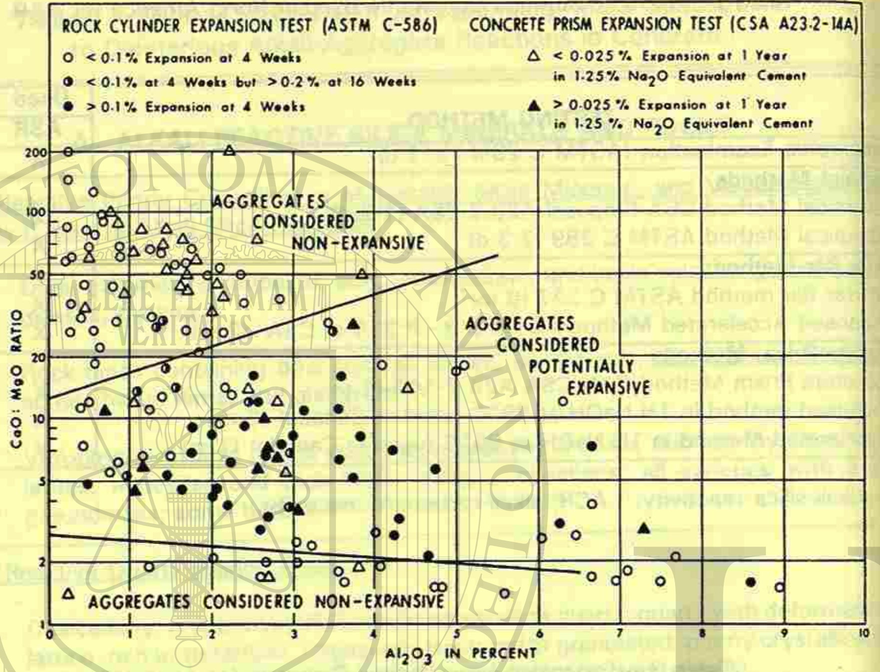

Fig. 1 -- Illustration of the division between non-expansive and potentially expan alkali-carbonate reactive rock on the basis of chemical composition. (Proposed met CSA A23.2-26A for detecting alkali-carbonate reactivity (1)).

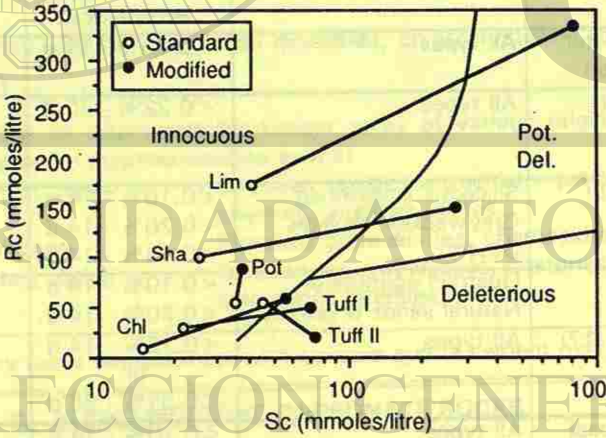

Fig. 2 - Examples of alkalisilica reactive aggregates from Quebec (Canada) that are detected using the standard Chemical Method ASTM C 289. However, most of th appear deleterious or potentially deleterious when the test is performed on the insolu residue in accordance with the so-called "Modified Chemical Method" (6). (Lim = silice limestone; Pot = Potsdam sandstone; Chl = chloritic schist; Sha = siliceous shale; T I & II = rhyolitic tuff I & II).

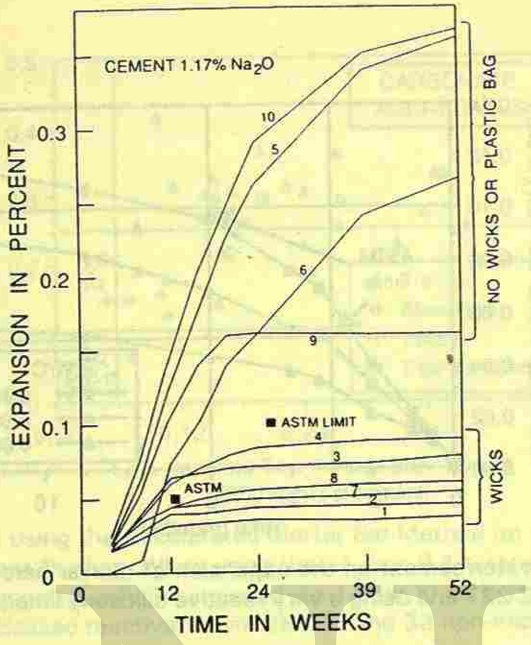

Fig. 3 - Expansion of mortar bars made and tested in accordance with ASTM C 227 with a very alkali-silica reactive limestone from Ottawa (Ontario, Canada), and stored in various types of containers. The presence or absence of wicks inside the containers is very critical. Container #1 is the container proposed in the ASTM standard. From reference (11).

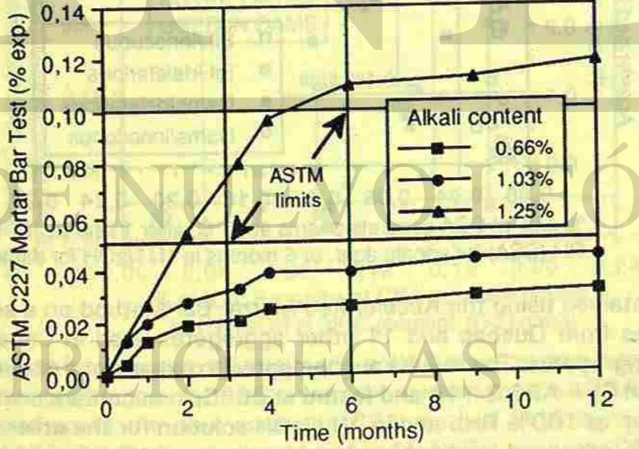

Fig. 4 - Effect of the alkali content (%Na_2O eq. by mass of cement) on the expansion of mortar bars made and tested in accordance with ASTM C 227 and using a very reactive siliceous limestone from Ottawa (Ontario, Canada).

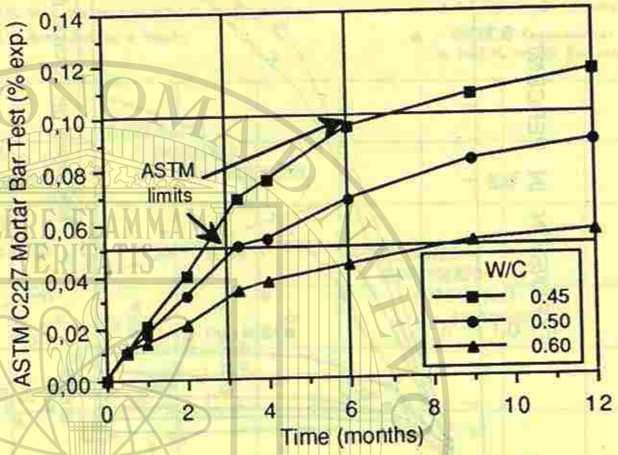

Fig. 5 - Effect of the water/cement on the expansion of mortar bars made and tested in cordance with ASTM C 227 and using a very reactive siliceous limestone from Trois-Rivi (Quebec, Canada).

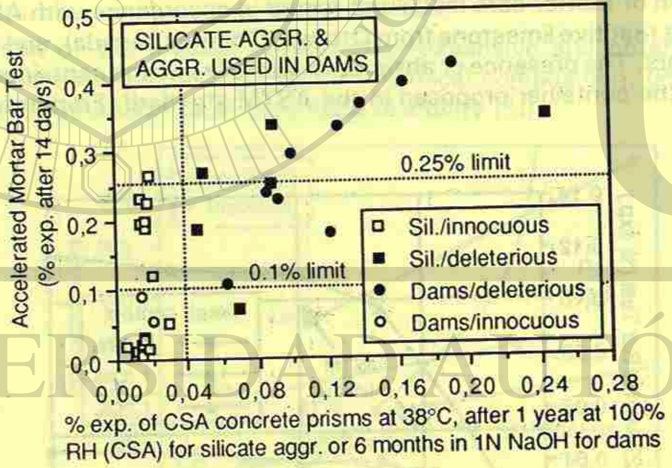

Fig. 6 - Results obtained using the Accelerated Mortar Bar Method on a series of 22 qua silicate aggregates from Quebec and 11 other aggregates used in Canadian dams, mo quarried silicate aggregates. The results are compared to expansions of concrete prisms m in accordance with CSA A23.2-14A and tested at 38°C, in accordance with this standa the first series (e.g. at 100% RH), and in 1N NaOH solution for the other series. When u the 14-day, 0.10% proposed mortar bar expansion limit, all 15 deleterious aggregates in field, which induced concrete prism expansion >0.04% after one year (CSA test) or 6 mo (immersion test), respectively, are classed reactive (all except the Potsdam sandstone), w only 11 of the 18 innocuous or presumably innocuous aggregates were evaluated corre

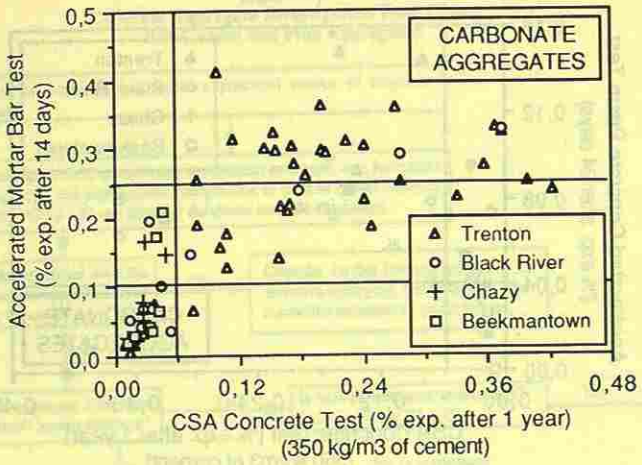

Fig. 7 - Results obtained using the Accelerated Mortar Bar Method on a series of 71 quarried carbonate aggregates from Quebec. When using the 14-day, 0.1% mortar expansion limit, 37 of the 38 expansive aggregates, which exceeded the 1-year, 0.06% concrete expansion limit used in that study, are classed reactive, while 85% of the 33 non-expansive aggregates are evaluated correctly.

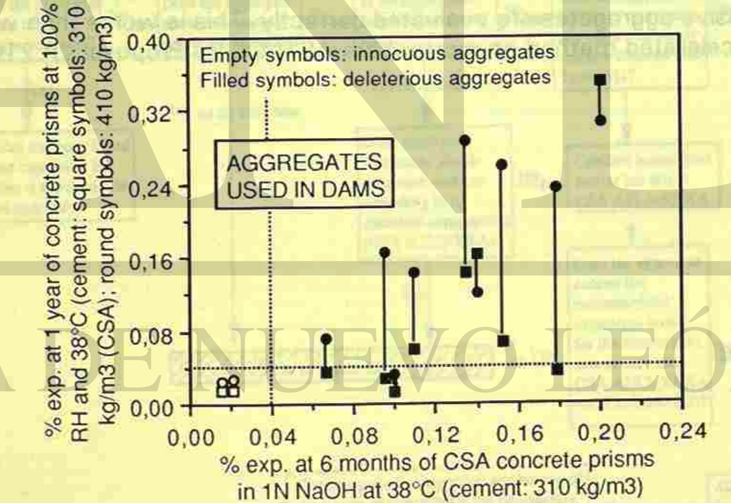

Fig. 8 - Results obtained for concrete prisms made with 9 reactive and 2 non-reactive aggregates used in Canadian dams, in accordance with the current method CSA A23.2-14A (310 kg/m³ of cement), and about the new proposed CSA method (410 kg/m³ of cement), and tested at 38°C, at 100% RH (both concretes) and in 1N NaOH solution (CSA concrete). The latter procedure was the only one able to properly classify all aggregates tested, using a 6-month, 0.04% expansion limit criterion. The current and the new proposed CSA methods failed in recognizing 4 and 1 of the reactive aggregates tested, respectively.

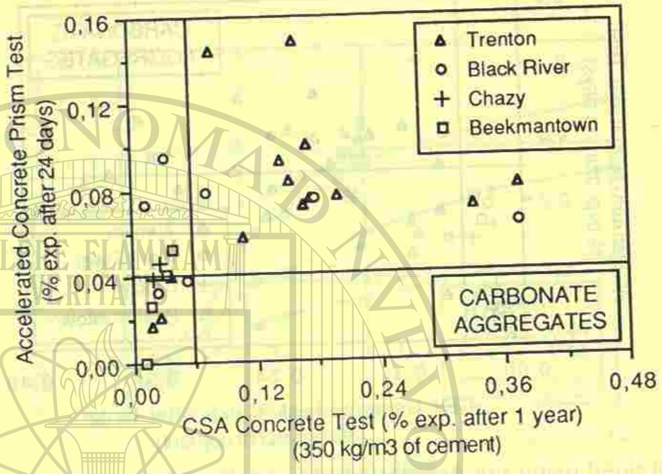

Fig. 9 - Results obtained using the Accelerated Concrete Prism Method on a series o quarried carbonate aggregates from Quebec. When using the 24-day, 0.04% prop expansion limit, all the 14 expansive aggregates, which exceeded the 1-year, 0.0 concrete expansion limit used in that study, are classed reactive, while only 62% of 13 non-expansive aggregates are evaluated correctly. This is worse than when using ASTM C 9 - Proposal - P 214 (see Fig equivalent accelerated method on mortar bars

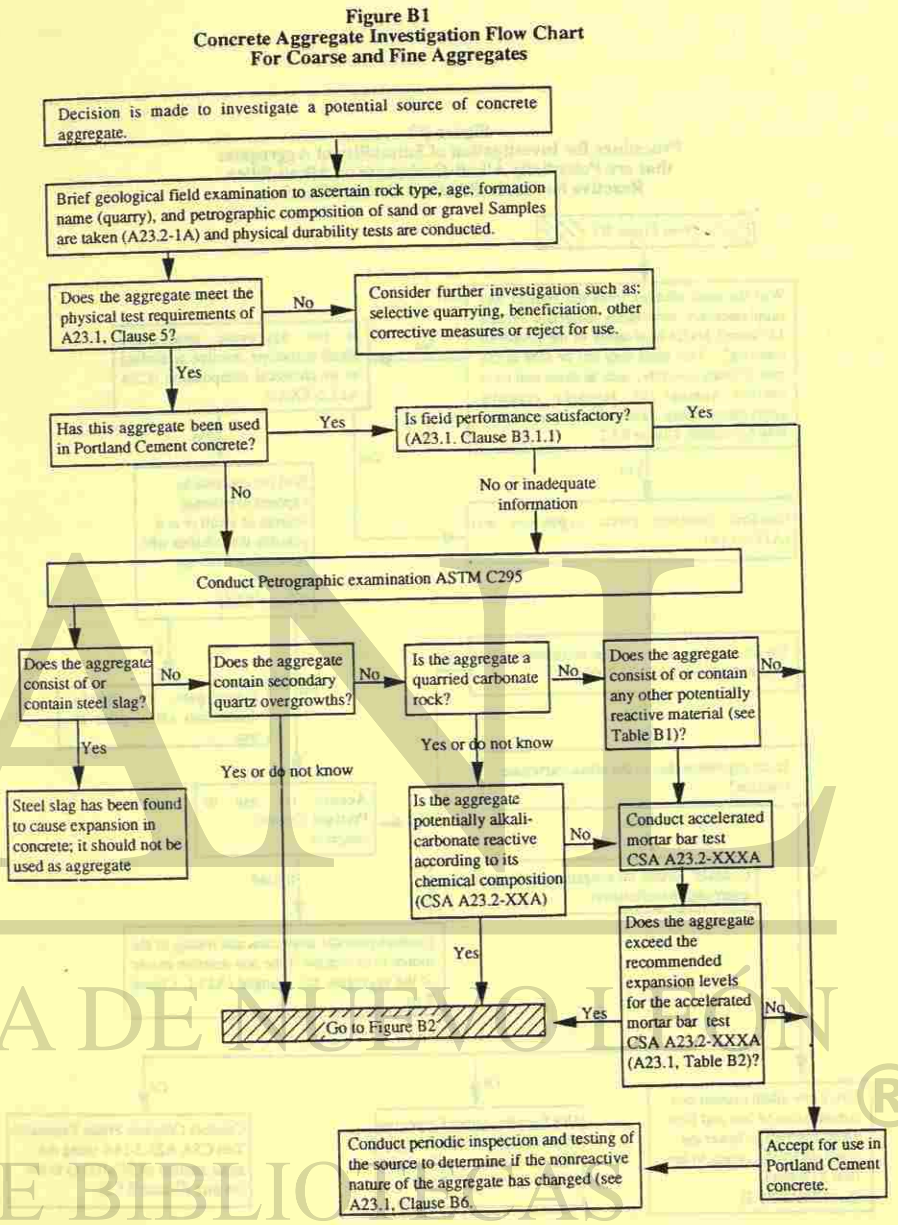

Fig. 10 - Decision chart proposed in Appendix B of the revised version of the CSA A23.1 standard for determining the potential alkali-reactivity of concrete aggregates (1). This chart is based on Canadian experience.

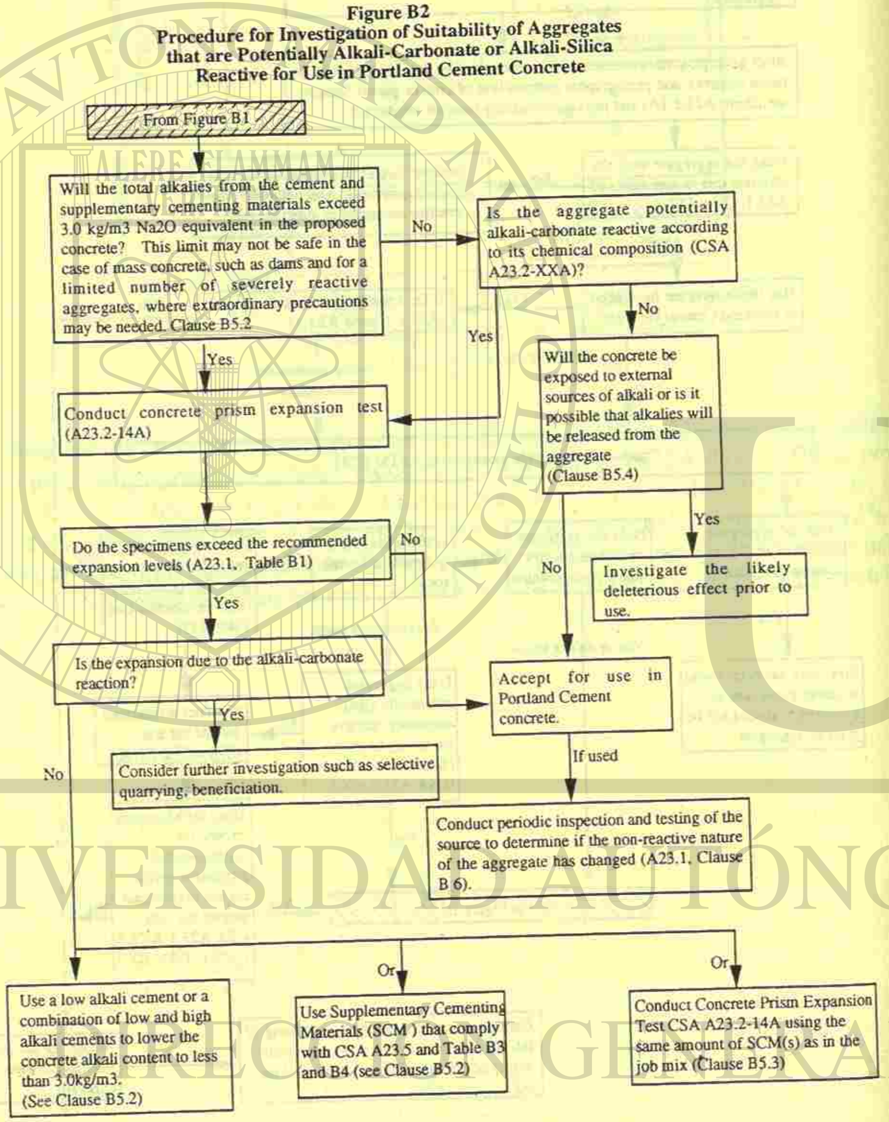

Fig. 10 (cont'd) - Decision chart proposed in Appendix B of the revised version of the C
A23.1 standard for determining the potential alkali-reactivity of concrete aggregates
This chart is based on Canadian experience.

AVOIDING CRACKING ENHANCES DURABILITY

by

Carlos J. Mendoza Escobedo

Symopsis: The principal causes of cracking in concrete and recommended crack co
procedures are presented. The current state of knowledge in microcracking is discus
The control of cracking due to drying shrinkage is covered in detail. Informati
provided to assist the engineer and the constructor in developing practical and effe
crack control programs for concrete structures.

Keywords: Steel for reinforcement, admixtures, aggregates, cracking, cement, expa
cements, dry shrinkage, cold concrete, humidity content, volumetric change, te
stress, thermic expansion, contraction joints, microcracking, modulus of elast
pozzolans.

EVITANDO AGRIETAMIENTOS SE MEJORA LA DURABILIDAD

por

Carlos Javier Mendoza Escobedo

Sinópsis: Se presentan las principales causas de agrietamiento del concreto y los procedimientos recomendados para el control de las grietas. Se discute el estado del conocimiento sobre microagrietamiento. Se analiza en detalle el control del agrietamiento debido a la contracción por secado. Se proporciona información útil para el ingeniero y el constructor en el desarrollo de programas prácticos y eficaces para el control del agrietamiento en estructuras de concreto.

Palabras clave: Acero de refuerzo, aditivos, agregados, agrietamiento, cemento, cementos expansivos, contracción por secado, concreto frío, contenido de humedad, cambio volumétrico, esfuerzos de tensión, expansión térmica, juntas de contracción, microagrietamiento, módulo de elasticidad, puzolanas.

CARLOS JAVIER MENDOZA ESCOBEDO

Grado:	Maestro en Ingeniería
Especialidad:	Estructuras (Materiales de construcción)
Tesis dirigidas:	19 de Licenciatura y 1 de Maestría
Publicaciones:	40 de distribución amplia y 39 informes de investigación de distribuc restringida
SNI:	Investigador Nacional Nivel 2
Cargo actual:	Subdirector, Instituto de Ingeniería, UNAM Profesor, Facultad de Ingeniería, UNAM

INTRODUCCION

Las grietas en estructuras de concreto pueden indicar la presencia de proble importantes y dañar la apariencia de la construcción monolítica. Pueden dejar al acero refuerzo expuesto al oxígeno y a la humedad y hacerlo más susceptible a la corros Aunque las causas específicas del agrietamiento son muchas, las grietas se orig normalmente por esfuerzos que se desarrollan en el concreto por la restricción al cam volumétrico o por las cargas que se aplican a la estructura. Dentro de cada una de es categorías hay un gran número de factores que las propician en cada obra. Un progr exitoso de control de agrietamiento debe reconocer estos factores y corregirlos.

En este trabajo se presenta una síntesis de las recomendaciones propuestas por miembros del Comité ACI 224 "Control de agrietamiento en estructuras de concreto y en el se discuten las principales causas del agrietamiento originado por la restricció cambio volumétrico y los procedimientos de control sugeridos.

MECANISMO DE AGRIETAMIENTO EN EL CONCRETO

Antes de la aplicación de carga, los cambios volumétricos en la pasta de ceme causan grietas interfaciales que se forman en la frontera mortero-agregado grueso, f Bajo cargas de compresión de corta duración, no se forman grietas adicionales hasta la carga alcanza aproximadamente 30 por ciento de la resistencia a compresión concreto arriba de este valor, se inicia un agrietamiento por falla de adherencia e agregado grueso y mortero. El agrietamiento por falla de adherencia se incrementa h que la carga alcanza aproximadamente 70 por ciento de la resistencia a compresión cuyo momento el microagrietamiento comienza a propagarse a través del mortero.

El agrietamiento del mortero continua a una velocidad acelerada hasta que el ma finalmente falla.

Para concreto en tensión uniaxial, el trabajo experimental indica que el agrietamiento importante comienza a aproximadamente 60 por ciento de la resistencia a tensión última.

Los estudios de la relación esfuerzo-deformación y cambio volumétrico del concreto indican que el inciso del agrietamiento importante del mortero corresponde con la observación de un incremento en la relación de Poisson del concreto, fig 2(2).

Desde que se estableció la existencia de grietas de adherencia ha sido usual atribuir toda la no linealidad del concreto a la formación de grietas microscópicas. Sin embargo, la relación de causa a efecto nunca se ha establecido. Estudios recientes indican que el grado de microagrietamiento se puede considerar más como un índice, del nivel de daño que como un factor que controla el comportamiento del concreto.

El trabajo experimental de Spooner et al(3) indica que la no linealidad del concreto está estrechamente ligada a la no linealidad de la pasta de cemento. Su trabajo muestra que el comportamiento de la pasta de cemento no es el de un material elástico, frágil, como se estableció en el pasado, si no el de un material no lineal con capacidad de deformación relativamente alta (0.005-0.007). El comportamiento no lineal de la pasta de cemento puede ser atribuido al daño ocasionado a la pasta de cemento, a cargas aún muy bajas, fig 3.

Usando un procedimiento de cargas cíclicas, Spooner et al han demostrado que tanto la pasta como el concreto presentan daños a deformaciones muy pequeñas (0.0004) a las que no se puede detectar un incremento en el microagrietamiento, como se muestra en la fig 4.

Estudios del comportamiento esfuerzo-deformación del concreto bajo carga de compresión cíclicas indican que el concreto se deteriora rápidamente una vez que el esfuerzo máximo excede aproximadamente 70 por ciento de la resistencia máxima a corto plazo, fig 5. Neville y Hirst(4) en su estudio de deformación diferida debida a cargas cíclicas, encontró que aún cuando los ciclos de carga se den a niveles inferiores a este valor se libera calor. Esto se atribuye al calor generado por el deslizamiento en la frontera de la interfase. Sin embargo, cuando se combina con el trabajo de Spooner, en el cual se muestra que la pasta sufre daño a cargas muy bajas, se concluye que el calor medido se puede deber a deslizamientos submicroscópicos dentro de la pasta.

La incapacidad de los modelos elásticos lineales para reproducir el comportamiento no lineal del concreto, utilizando solamente microagrietamiento, se ha explicado como debido al hecho de que el concreto es realmente un "material estadístico". Cuando se selecciona la variación estadística adecuada, el comportamiento no lineal del concreto se puede reproducir. Aunque la variación estadística indiscutiblemente juega un papel importante, la mayor parte del comportamiento no lineal puede alcanzarse también considerando la no linealidad de los constituyentes del mortero. La fig 6 ilustra los resultados obtenidos para un modelo muy simplificado del concreto bajo compresión uniaxial, usando una representación no lineal del concreto. La relación esfuerzo-deformación para el modelo sin agrietamiento difiere muy poco de los modelos con un microagrietamiento normal o a de lo normal.

El microagrietamiento tiene un efecto relativamente pequeño en el comportamie primario esfuerzo-deformación de los modelos. El efecto dominante del microagrietamie está en incrementar la deformación lateral. En cada caso la falla del modelo e gobernada por el aplastamiento del mortero el cual se presenta a una resistencia prom inferior a la del mortero solo.

Newman(5) y Tasuju, Slate y Nilson(6) han observado que la deformación princip tensión en el concreto al "esfuerzo de discontinuidad" parece ser una función del esfu normal medio, $\sigma_m=(\sigma_1+\sigma_2+\sigma_3)/3$. En su estudio de resistencia biaxial del conc Tasuji y colaboradores, observaron que la falla final de sus especímenes consiste e formación de grietas de tensión macroscópicas. Ellos también observaron que el esfu en la discontinuidad ocurre a aproximadamente 75 por ciento de la resistencia últ compresión y a aproximadamente 60 por ciento de la resistencia última para aquellos c que incluyen tensiones, alcanzando los niveles a los cuales comienza el agrietamient mortero. Su trabajo parece dirigirse fuertemente hacia una "deformación límite de tens como el factor gobernante en la resistencia del concreto.

Redondeando, el daño en la pasta de cemento parece jugar un papel importante e control del comportamiento primario esfuerzo-deformación del concreto bajo carga d de corta duración. En concretos de peso normal, las partículas de agregados actúan c incrementadores de esfuerzo, aumentando la rigidez inicial y disminuyendo la resiste de la pasta. Para carga cíclica y sostenida, una gran parte del agrietamiento adherencia se origina de la carga inducida por los cambios volumétricos dentro la p pero no tiene efecto importante en la resistencia.

CONTROL DE AGRIETAMIENTO DEBIDO A LA CONTRACCION POR SECADO

El agrietamiento del concreto debido a la contracción por secado es el aspecto mayor atención ha recibido por arquitectos, ingenieros y contratistas que ninguna característica o propiedad del concreto. Es uno de los problemas más serios que enfre las construcciones de concreto. Diseño y prácticas constructivas adecuadas pu minimizar la cantidad de agrietamiento y eliminar las grietas grandes visibles media empleo de refuerzo adecuado y juntas de construcción.

Aunque la contracción por secado es una de las principales causas de agrietami los esfuerzos por temperatura, las reacciones químicas, la acción de la congelación como los esfuerzos de tensión excesivos debidos a las acciones externas, responsables frecuentemente del agrietamiento en el concreto endurecido. agrietamiento también se puede presentar en el concreto antes del endurecimiento d a la contracción plástica.

Formación de grietas

Si la contracción del concreto originada por el secado puede presentarse sin ninguna restricción, el concreto no se agrieta. Sin embargo, en una estructura, el concreto está siempre sujeto a algún grado de restricción, ya sea por la cimentación o alguna otra parte de la estructura, o por el acero de refuerzo dentro del concreto. Esta combinación de contracción y restricción origina esfuerzo de tensión. Cuando estos esfuerzos de tensión alcanzan su resistencia, el concreto se agrieta, fig 7.

Otro tipo de restricción se origina por la diferencia de la contracción en la superficie y en el interior de un miembro de concreto, especialmente a edades tempranas. Ya que la contracción por secado es siempre mayor en la superficie expuesta, la parte interna del miembro restringe la contracción del concreto superficial, desarrollándose esfuerzos de tensión. Esto puede originar agrietamiento superficial, grietas que no penetran mucho en el concreto. Estas grietas superficiales, con el tiempo, pueden penetrar más, cuando la parte interna del concreto está sujeta a contracción adicional.

La magnitud del esfuerzo de tensión que se desarrolla durante el secado del concreto está influenciado por una combinación de factores, tales como (a) la magnitud de la contracción, (b) el grado de restricción, (c) el módulo de elasticidad, y (d) la defcrmación diferida o relajación del concreto. Así, la magnitud de la contracción es solamente uno de los factores que gobiernan el agrietamiento. En lo que concierne al agrietamiento, un bajo módulo de elasticidad y una deformación diferida alta del concreto son deseables, ya que estos reducen la magnitud de los esfuerzos de tensión. Por tanto, para minimizar el agrietamiento, el concreto debe tener baja contracción por secado y un alto grado de deformabilidad (bajo módulo y alta deformación diferida) así como una alta resistencia a la tensión. Sin embargo, una alta deformabilidad de los miembros de concreto sujetos a tensión causarán grandes deflexiones.

Contracción por secado

Cuando el concreto se seca, se contrae, y cuando se humedece de nuevo, se expande. Estos cambios de volumen, con cambios en el contenido de humedad, son características inherentes a los concretos del cemento hidráulico. Es el cambio en el contenido de humedad de la pasta de cemento la que causa la contracción o la expansión del concreto, mientras que los agregados proporcionan una restricción interna, la cual reduce en forma importante la magnitud de estos cambios de volumen.

Cuando el cemento se mezcla con agua, tiene lugar varias reacciones químicas. Estas reacciones llamadas comúnmente "hidratación", originan un producto de hidratación consistente principalmente de algunos materiales cristalinos (principalmente hidróxido de calcio) y una gran cantidad de gel de silicato de calcio endurecido llamado "gel de tobermorita". Este gel rígido está formado por particular de tamaño coloidal y tienen un área superficial extremadamente alta. En una pasta de cemento endurecida, algo de agua está en los poros capilares de la pasta, pero una cantidad significativa está en el gel de tobermorita. La contracción se debe a la pérdida de agua absorbida del gel. Durant secado el agua que se pierde primero es la que ocupa los espacios capilares más gran en la pasta de cemento. Esta pérdida de agua origina muy poca contracción. E pérdida del agua absorbida y la que se encuentra entre capas dentro del gel hidratad que causa la contracción de la pasta. Cuando un concreto está expuesto a condicio de secado, la humedad migra lentamente del interior de la masa de concreto haci superficie, donde se pierde por evaporación. Con el humedecimiento este proceso revierte, originando una expansión del concreto.

Adicionalmente, la pasta de cemento está sujeta también a la contracción carbonatación. La acción del dióxido de carbono, CO_2, presente en la atmósfera sobre productos de hidratación del cemento, principalmente hidróxido de calcio, $Ca(OH)_2$, orig la formación de carbonato de calcio, $CaCO_3$, la que está acompañada de una disminu en el volumen. Ya que el dióxido de carbono no penetra mucho en la masa de concr la contracción debida a la carbonatación es la de menor importancia en la contracción t de la estructura de concreto. Sin embargo, la carbonatación juega un papel important la contracción de los especímenes pequeños de laboratorio, particularmente cuando es expuestos al secado por mucho tiempo. Así, la contracción observada en especíme pequeños de laboratorio será más grande que la contracción del concreto en la estruct

Factores que influyen en la contracción por secado

Los factores que mayor influencia tienen en la contracción por secado incluye composición del cemento, el tipo de agregado, el contenido de agua, y proporcionamiento de las mezclas. La velocidad de pérdida de humedad, o la contrac de un concreto dado, está influenciada en gran medida por el tamaño y forma del miem de concreto, el ambiente y la duración de la exposición al secado.

Efecto del cemento - Los resultados de un estudio amplio realizado por Blaine, A Evans, del National Bureau of Standards(7) sobre un gran número de cementos port indican que la contracción más pequeña de la pasta está asociada con: 1. Menor rela C_3A/SO_3, 2. Menores contenidos de Na_2O y K_2O y 3. Mayores contenidos de C_4AF cemento.

Los ensayes realizados por la División de Carreteras de California(8) sobre morter pastas como medida del comportamiento del concreto indicaron que los cementos ti generalmente originan menor contracción que los cementos tipo I y mucho menos qu cementos tipo III. Pruebas realizadas por Lerch(9) muestran que la proporción de yes el cemento tiene un efecto importante en la contracción. Los productores de cem moderan las diferencias en la contracción, debido a la composición del cemento, optima el contenido de yeso.

La finura del cemento puede tener influencia en la contracción por secado. ensayes de Carlson(10) mostraron que los cementos más finos generalmente condu

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a mayores contracciones del concreto, pero el incremento en la contracción con el incremento en la finura no es grande.

Influencia del tipo de agregado - Los agregados gruesos y finos, que ocupan entre 65 y 75 por ciento del volumen total del concreto, tienen una influencia importante en la contracción. El concreto se puede considerar que consiste de una estructura de pasta de cemento cuya contracción potencial más grande está restringida por el agregado. La contracción por secado de un concreto será una fracción (aproximadamente de 1/4 a 1/6) de la correspondiente a la pasta de cemento. Los factores que influyen en la capacidad de los agregados para restringir la contracción incluyen (a) la compresibilidad del agregado y la deformabilidad de la pasta, (b) la adherencia entre la pasta y el agregado, (c) el grado de agrietamiento de la pasta de cemento, y (d) la contracción de las partículas de agregados debido al secado. De estos factores, la compresibilidad del agregado tiene la influencia más grande en la magnitud de la contracción por secado del concreto.

Mientras más grande es la rigidez o módulo de elasticidad de un agregado, mayor será su efectividad en la reducción de la contracción del concreto. La absorción de un agregado, la cual es una medida de su porosidad, influye en su módulo o compresibilidad. Un bajo módulo, usualmente está asociado a una alta absorción.

La gran influencia del tipo de agregado en la contracción por secado la mostró Carlson(11) Como ejemplo, algunos de sus datos de contracción para concretos con idénticos cementos y relaciones agua/cemento se muestran a continuación:

Efecto del tipo de agregado en la contracción del concreto

Agregados	Gravedad específica	Absorción, por ciento	Contracción a un año, por ciento
Arenisca	2.47	5.0	0.116
Pizarra	2.75	1.3	0.068
Granito	2.67	0.8	0.047
Caliza	2.74	0.2	0.041
Cuarzo	2.66	0.3	0.032

El tamaño máximo de agregado tiene un efecto importante en la contracción por secado. No solamente porque un concreto con tamaño grande de agregado requiere de menor consumo de agua, sino porque también es más efectivo para resistir la contracción de la pasta de cemento. La granulometría del agregado también tiene efecto en la contracción. El empleo de un agregado fino o grueso con granulometría deficiente puede dar como resultado una mezcla con demasiada arena, para alcanzar la trabajabilidad deseada, impidiendo el uso de la máxima cantidad de agregado grueso posible, dando como resultado un incremento en la contracción.

Efecto del contenido de agua y el proporcionamiento de las mezclas - El contenid agua de una mezcla de concreto es otro factor muy importante que influye en contracción por secado. Una relación típica entre el contenido de agua y la contracc por secado se muestra en la fig 8. Un incremento en el contenido de agua también red el volumen de los agregados que restringen la contracción y por tanto se incrementa és La contracción del concreto se puede minimizar manteniendo el contenido de agua de pasta tan bajo como sea posible y el contenido total de agregados del concreto lo más posible. Esto dará como resultado un contenido de agua más bajo por volumen unit de concreto y así tener una contracción más baja.

El volumen total de agregado grueso es un factor significativo en la contracción secado. Los concretos proporcionados para colocarse por medio de bomba con contenido de arena excesivamente alto presentarán contracciones significativame mayores que las que alcanzan mezclas similares con contenidos normales de arena.

La cantidad de agua de mezclado requerida para concreto de un revenimiento d depende importantemente del tamaño máximo del agregado. El área superficial de agregados la cual debe ser recubierta por la pasta de cemento, disminuye con el aume en el tamaño del mismo. El efecto más grande que el tamaño máximo de agregado t en el consumo de agua del concreto, se muestra en la fig 9.

Otro factor importante que tiene influencia en el consumo de agua de un concret por tanto en su contracción, es la temperatura del concreto fresco. El efecto de temperatura en el consumo de agua, como lo proporciona el U.S. Bureau of Reclama se muestra en la fig 10.

De lo discutido anteriormente se debe concluir que, para minimizar la contracción secado del concreto, el consumo de agua de una mezcla debe mantenerse a un mín Cualquier práctica que tienda a incrementar el consumo de agua, tales como el emple revenimientos altos, altas temperaturas del concreto fresco, o el uso de tamaños agregado grueso pequeño, aumentará en forma importante la contracción y por tant agrietamiento del concreto.

Efecto de los aditivos químicos - Los aditivos químicos se usan para impartir cie propiedades deseables al concreto. Los más comúnmente usados son aditivos inclus de aire, reductores de agua, retardantes del fraguado y acelerantes.

Se podría esperar que cuando se usa un aditivo inclusor de aire, el incremento e cantidad de vacíos aumenta la contracción por secado. Sin embargo, debido a qu inclusión de aire permite una reducción en el consumo de agua, sin reducción revenimiento, la contracción no se ve afectada en forma importante para contenidos aire de hasta 5 por ciento. Algunos agentes inclusores de aire son además retarda muy activos y, para compensar el retardo, incluyen acelerantes que pueden increme la contracción por secado de 5 a 10 por ciento.

Aunque el empleo de aditivos reductores de agua y retardantes del fraguado permitirá una reducción en el consumo de agua en las mezclas de concreto, usualmente esto no da como resultado una reducción en la contracción del concreto. En realidad, algunos de estos aditivos pueden aumentar la contracción a edades tempranas de secado, aunque a edades posteriores la contracción de estos concretos será aproximadamente iguales al que corresponde a las mezclas sin aditivos.

El uso de cloruro de calcio, un acelerante muy común, dará como resultado un incremento importante en la contracción por secado, especialmente a edades tempranas de secado.

Efecto de puzolanas - El empleo de algunas puzolanas naturales puede incrementar la demanda de agua así como la contracción por secado del concreto. Algunas de ellas incrementa la contracción por secado, aunque tengan poco efecto en el consumo de agua del concreto. Algunas cenizas volantes tienen poco efecto en la contracción por secado, mientras que otras pueden incrementarla. Todas estas observaciones están basadas en resultados de ensayes hechos en especímenes pequeños de laboratorio.

Efecto de la duración del curado húmedo - Carlson(10) reportó que la duración del curado húmedo del concreto no tiene efecto significativo en la contracción por secado. Esto se ve apoyado por los resultados de ensaye del Departamento de Transporte de California(8) los cuales muestran la misma contracción para los concretos con curado húmedo por 7, 14 ó 28 días, antes de que comience el secado. En lo que concierne a la tendencia del concreto al agrietamiento, el curado húmedo no necesariamente es benéfico. Aunque la resistencia se incrementa con la edad, el módulo de elasticidad también se incrementa, casi en el mismo porcentaje, y el resultado final es solamente un incremento pequeño en la deformación a tensión que el concreto puede soportar.

El curado a vapor a presión atmosférica, que se usa frecuentemente en la fabricación de elementos estructurales precolados, reducirá la contracción por secado. También, debido a que el curado a vapor producirá una resistencia del concreto alta a edades tempranas, se reducirá la tendencia al agrietamiento, ya que los miembros precolados no tienen restricciones.

Influencia del tamaño del miembro - El tamaño de los miembros de concreto influirá en la rapidez con que se elimine la humedad del concreto y por tanto influye en la velocidad de la contracción. Carlson(11) ha mostrado que para concreto expuesto a una humedad relativa de 50 por ciento, el secado penetrará solamente 7.5 cm en un mes y aproximadamente 60 cm en 10 años. La fig 11 muestra las curvas teóricas obtenidas para el secado de una losa. Hansen y Mattock(12) hicieron una investigación extensa de la influencia del tamaño y forma del miembro en la contracción y deformación diferida del concreto. Encontraron que ambas, la contracción y la deformación diferida, disminuyen a medida que es más grande el elemento.

Este efecto significativo del tamaño del elemento en la contracción del concreto se debe considerar cuando se evalúa la contracción potencial del concreto en estructuras, basándose en la contracción de especímenes de laboratorio. La velocidad y magnitud de la contracción en especímenes pequeños de laboratorio será más grande que la que tiene el concreto en las estructuras. Como ejemplo del efecto del tamaño de los especímenes en la contracción, están los datos presentados en la fig 12, que proporciona los resultados de los ensayes de contracción obtenidos de prismas de concreto de cuatro tamaños diferentes. Se notará que la contracción de los prismas que tienen una sección de 7.5 x 7.5 cm fue 50 por ciento mayor que la de los prismas con una sección de 12.5 cm x 1 cm.

Control del agrietamiento por contracción

El concreto tiende a contraerse debido al secado cuando sus superficies están expuestas al aire con baja humedad relativa. Debido a que varios tipos de restricciones evitan que el concreto se contraiga libremente, la posibilidad de agrietamiento debe esperarse a menos que la humedad relativa ambiente se mantenga a 100 por ciento, o que la superficie del concreto esté sellada para evitar la pérdida de humedad. El control de agrietamiento consiste en reducir a un mínimo la tendencia a agrietarse, mediante el uso de acero, colocado donde se requiera, y con el empleo de juntas de contracción. reglamento CEB-FIP da recomendaciones cuantitativas sobre el control de agrietamiento debido a la contracción, presentando varios coeficientes para determinar los niveles de contracción que pueden esperarse.

El agrietamiento también puede minimizarse mediante el uso de cementos expansivos para producir concretos con contracción compensada.

Reducción de la tendencia al agrietamiento - Como se mencionó previamente, tendencia al agrietamiento no se debe únicamente a la magnitud de la contracción, s también al grado de restricción, el módulo de elasticidad y la deformación diferida relajación del concreto. Algunos factores que reducen la contracción al mismo tiempo disminuyen la deformación diferida o relajación e incrementa el módulo de elasticidad dando como resultado poco o casi nulo beneficio en la tendencia al agrietamiento. Se debe poner énfasis, por tanto, en aquellos factores que producen una reducción neta en tendencia al agrietamiento.

Cualquier medida que se tome para reducir la contracción del concreto reduce también la tendencia al agrietamiento. La contracción por secado se puede reducir usando menos agua en la mezcla y tamaño máximo de agregado grueso más grande. contenido de agua menor se puede alcanzar usando agregados bien graduados consistencia más seca y temperatura inicial del concreto más baja. Sin embargo, reducción en el contenido de agua por el empleo de aditivos reductores de ag usualmente no reducirá la contracción.

Otra forma de reducir la tendencia al agrietamiento es mediante la aplicación de recubrimiento superficial al concreto, el cual evitará la pérdida rápida de humedad. Es manera de controlar el agrietamiento no se ha aprovechado en todo su potencial y se

debe dar mayor atención. Sin embargo, muchos recubrimientos superficiales, como las pinturas multiusos, no son efectivos porque permiten la pérdida de humedad casi tan pronto como llega a la superficie. El hule clorinado y las ceras o materiales resinosos son recubrimientos efectivos, pero probablemente hay muchos otros materiales que permitirán la evaporación suficientemente lenta para ser benéficos. Cualquier medio que retarde la velocidad de contracción será benéfico, porque el concreto tiene la importante cualidad de relajarse bajo esfuerzo sostenido. Así el concreto puede ser capaz de soportar dos o tres veces más contracción cuando ésta se presenta lentamente que cuando actúa rápidamemente.

Refuerzo - El refuerzo colocado en forma adecuada, y en cantidad conveniente, no solamente reducirá la cantidad de agrietamiento sino que también evita el agrietamiento desagradable a la vista.

La distribución de la deformación por contracción a lo largo del refuerzo por medio de esfuerzos de adherencia, hace que las grietas se distribuyan de tal manera que se forme un número importante de grietas muy finas, en lugar de unas cuantas grietas anchas. Aunque el uso de este refuerzo para controlar el agrietamiento es práctico en secciones de concreto relativamente delgadas, no se necesita en estructuras masivas, como presas, debido a la baja contracción por secado de estas estructuras de concreto masivo.

Juntas - El empleo de juntas es el método más efectivo para evitar la formación de grietas gruesas. Si se tienen longitudes largas o superficies extendidas de concreto, como en muros, losas o pavimentos, y no se proporcionan juntas separadas convenientemente para absorber las contracciones, el concreto hará sus propias juntas por agrietamiento.

Las juntas de contracción se hacen en muros, por ejemplo, sujetando a las cimbras tiras de madera o hule, las cuales dejan ranuras verticales estrechas en el concreto sobre las caras interior y exterior del muro. El agrietamiento del muro debido a la contracción se presentará en las ranuras, relevando los esfuerzos en el muro y evitando de esa manera la formación de grietas no controladas. Estas ranuras se sellarán en la superficie exterior de los muros para prevenir la penetración de la humedad. Las juntas cortadas se usan frecuentemente en pavimentos, losas y pisos.

La localización de las juntas depende de las particularidades de la colocación. Cada obra se debe estudiar en forma individual para determinar donde se deben colocar las juntas.

Concretos compensadores de la contracción

Los concretos compensadores de la contracción hechos con cementos expansivos se pueden usar para minimizar o eliminar el agrietamiento por contracción. Las propiedades y usos de los concretos con cementos expansivos están publicados en numerosos artículos e informes. De los varios tipos de cementos expansivos que se producen, el cemento expansivo compensador de la contracción tipo K es el más empleado.

En concreto reforzado, la expansión de la pasta de cemento durante los primeros d de curado desarrollará un bajo nivel de presfuerzo, induciendo esfuerzos de compresión el concreto y de tensión en el acero. El nivel de esfuerzo de compresión desarrollado los concretos compensadores de la contracción varía entre 2 y 7 kg/cm². Cuando presenta la contracción por secado, la contracción del concreto dará como resultado u reducción o eliminación de la precompresión. La precompresión inicial del concreto redu la magnitud de cualquier esfuerzo de tensión que se puede presentar debido a contracción, y así disminuye o elimina la tendencia al agrietamiento. Este concepto bás del uso de los elementos expansivos para producir concretos compensadores de contracción se ilustra en la fig 13.

La cantidad de acero de refuerzo normalmente empleado en concreto reforzado hec con cemento portland usualmente es más que suficiente para proporcionar la restricc elástica requerida para los concretos compensadores de la contracción. Para sa provecho de la potencialidad expansiva del concreto compensador de la contracción importante que se inicie un curado ininterrumpido con agua (recubrimiento húmedo o inundación) inmediatamente después del acabado. Un curado inadecuado de un concre compensador de la contracción puede dar como resultado una expansión insuficiente p estirar el acero y si esto no ocurre el concreto se agrietará durante la contracción secado, fig 14.

CONTROL DE AGRIETAMIENTO MEDIANTE PRACTICAS CONSTRUCTIVAS ADECUADAS

Las prácticas constructivas, incluyen diseño, especificaciones, materiales consideraciones de mezclado, así como la práctica constructiva misma en la obra. An de discutir el control de la práctica constructiva que afecta el agrietamiento, conveniente mencionar las causas básicas del agrietamiento: la restricción. Si todas partes del concreto en una estructura de concreto están libres para moverse cuand concreto se expande o contrae, particularmente esto último, no habrá agrietamiento deb al cambio volumétrico.

Obviamente, las componentes de las estructuras de concreto no están libres, y tanto, no están libres a responder en el mismo grado a los cambios volumétric Consecuentemente, se desarrollan deformaciones diferenciales y se inducen esfuerzos tensión. Cuando estas respuestas diferenciales exceden la capacidad del concret resistirlas, en ese momento se presenta el agrietamiento. Esto señala la importancia proteger al concreto en proceso de endurecimiento, tanto como sea posible, de la pérd de humedad o de una disminución en la temperatura. Estas consideraciones pueden como resultado esfuerzos capaces de causar grietas a una edad temprana, pero pue ser soportadas a una madurez mayor.

Restricciones

Las restricciones existen en muchas circunstancias bajo las cuales la estructura y sus elementos de concreto debe comportarse satisfactoriamente.

Un muro o parapeto anclado a lo largo de su base a la cimentación, o a los elementos estructurales inferiores, sujetos a cambios volumétricos menores, estará restringido a la contracción cuando su parte superior se acorte debido al secado o enfriamiento. El agrietamiento es inevitable a menos que se proporcionen juntas de contracción o ranuras de una profundidad no menor que 10 por ciento del espesor del muro, en ambas caras, en las cuales se puede presentar el agrietamiento a intervalos que varían de una a tres veces la altura de los muros, para muros altos y bajos, respectivamente.

El concreto exterior e interior, particularmente en las secciones más gruesas, cambiará de temperatura o contenido de humedad a diferentes velocidades y en diferentes grados. Cuando esto sucede, el concreto interior restringe al concreto exterior de contraerse, y las deformaciones de tensión desarrolladas puede originar que se agriete. Esto ocurre cuando la superficie se enfría, mientras el interior está aún caliente por el calor de hidratación, o cuando la superficie del concreto se seca más rápidamente que el concreto interior. Como se hizo notar anteriormente, a menudo es conveniente proteger la superficie por un tiempo a edades tempranas de tal forma que los esfuerzos diferenciales inducidos no se puedan presentar antes de que el concreto sea lo suficientemente resistente para soportar la deformación sin agrietarse.

De forma similar el refuerzo por temperatura restringe la contracción del concreto superficial, originando un mayor número de grietas pero más estrechas.

En cambios fuertes de sección, se presentarán restricciones, ya que el efecto del cambio de temperatura o de contracción por secado será diferente en las dos secciones. Si es posible, se usará una junta de contracción para relevar los esfuerzos en la restricción.

El anclaje de la losa en las losas perimetrales o zapatas origina restricciones trabajo del elemento en su plano. Cuando una losa tiene libertad para contraerse a partir de todos sus lados hacia su centro hay un agrietamiento mínimo. Las juntas de contracción y los apoyos perimetrales serán diseñados en concordancia con las restricciones.

Los muros, losas y recubrimiento de túneles colados contra la superficie irregular de una excavación en roca están restringidos del movimiento cuando la superficie se expande o contrae en respuesta a cambios en la temperatura o contenido de humedad. En estos casos se debe proporcionar juntas de contracción cercanamente espaciadas o ranuras profundas para evitar u ocultar las grietas que a menudo desfiguran esas superficies. En recubrimiento de túneles la contracción en las primeras semanas es principalmente térmica, y el empleo de concreto frío (10·C) reduce el agrietamiento en forma importante. Con el tiempo el secado es importante, el recubrimiento de concreto es mucho más resistente y capaz de soportar mejor el agrietamiento por contracción. Sin embargo, las grietas circunferenciales en recubrimientos de túneles y otros conductos de concreto y líneas de tuberías colados en el lugar se pueden reducir en forma importante en número y anch Como se señala en el Manual del Concreto del Bureau of Reclamation(13), esto se pue lograr si se usa una mampara para evitar el movimiento del aire a través del túnel, y si s deja agua en estanques poco profundos en la parte inferior del túnel, tan pronto como se posible y hasta ponerlo en servicio; si el túnel conducirá agua no habrá contracción po secado posterior. Sino, el concreto será mucho más resistente en un ambiente húme y será capaz de resistir mejor los esfuerzos de tensión inducidos por la contracción.

Contracción

A continuación se discute las causas principales de la contracción, la cual origina formación de grietas en el concreto.

Efecto del contenido de agua - A medida que mayor es el contenido de agua en concreto, más se contraerá durante el secado. Es importante el empleo del revenimien más bajo que sea práctico. De gran importancia resulta también la selección de la mezc que requiere la mínima cantidad de agua por metro cúbico, para la resistencia deseada d concreto. Esto significa evitar mezclas con exceso de arena (a medida que es más rico concreto, más gruesa puede ser la arena y menor cantidad se empleará en la mezcl emplear el agregado de tamaño máximo más grande que sea práctico; usar agregados q tenga la forma y la granulometría más favorable para alcanzar la mejor trabajabilidad; usar arena bien graduada con un mínimo de finos que pasen la malla No 100 y sin arci tal que su equivalente de arena no sea menor que 80 por ciento.

Contrario a lo que se cree comúnmente, el incremento en el consumo de cemento el concreto, no necesariamente origina un incremento en la contracción. Esto es porq la cantidad de agua requerida no cambia mucho con el contenido de cemento. contracción por secado es proporcional al contenido de agua, no al contenido de cemen Más aun, la reducción de la cantidad de agregados finos para compensar el ceme agregado, de acuerdo con los principios de correcciones del proporcionamiento concreto, eliminará cualquier tendencia a incrementar el requerimiento de agua.

Secado superficial - El secado superficial ocurrirá en algún momento, excepto cuan la superficie está sumergida. Originará deformaciones por contracción de 600 x 10 más. La cantidad de agrietamiento por contracción dependerá de 1. que tan seca lleg a estar la superficie, 2. que tanta agua de mezclado se empleó en el concreto, 3. carácter y el grado de restricción involucrado, 4. la deformabilidad del concreto. deformabilidad representa qué tanto el concreto puede ser deformado (acortado), exceder su resistencia a tensión y es la suma de la deformación diferida más la capaci de deformación elástica.

Donde el secado de la superficie puede ser rápido, mayor cuidado se le pondrá a curado ininterrumpido para alcanzar buena resistencia superficial. Los esfuerzos agrietamiento serán reducidos posteriormente por la deformación diferida, si se evita la superficie se seque rápidamente al finalizar la etapa de curado. Para lograr esto,

material de recubrimiento para el curado húmedo se permitirá que permanezca varios días sin humedecimiento posterior después de que la etapa de curado se ha terminado (preferiblemente de 7 a 10 días), hasta que la cubierta y el concreto bajo ella parecen haberse secado.

Contracción plástica - Las grietas por contracción plástica se presentan usualmente, y son objetables en la superficie de pisos y losas, cuando las condiciones ambiente de la obra son tan áridas que la humedad superficial se elimina más rápidamente de lo que se puede reemplazar con el agua de sangrado de abajo. Estas grietas se presentan antes del acabado final y del inicio del proceso de curado. Al mismo tiempo que se pierde humedad, la superficie de concreto se contrae, dando como resultado esfuerzos de tensión en el concreto en proceso de rigidización, esencialmente sin deformación, que origina grietas cortas distribuidas aleatoriamente o aberturas en la superficie. Estas grietas usualmente son anchas pero de solamente unos cuantos centímetros de profundidad. Las grietas generalmente varían desde pocos centímetros hasta el metro en longitud y están separadas entre unos cuantos centímetros y 60 cm.

Algunas veces las grietas por contracción plástica aparecen suficientemente pronto para hacerlas desaparecer con aplanado posterior o con las primeras operaciones con la llana. Cuando esto tiene éxito, es recomendable posponer estas operaciones tanto como sea posible para alcanzar un beneficio máximo sin que se vuelva a presentar el agrietamiento.

En otros casos, un aplanado más temprano que lo normal puede destruir la tensión creciente mediante el remoldeo de la superficie y evitar el agrietamiento plástico que de otra forma se presentaría. A los primeros indicios de agrietamiento, mientras el concreto aun es manejable, se trabajará vigorosamente sobre él para cerrar las grietas, ya sea llenándolas o golpeándolas con la plana. Si se cerrara firmemente, el concreto volverá a ser monolítico y es poco probable que vuelvan aparecer las grietas. Sin embargo, reaparecerán si sólo son alisados con la llana. En cualquier caso, el curado comenzará tan pronto como sea posible.

Las condiciones más favorables para originar agrietamiento por contracción plástica las altas temperaturas y viento seco. De acuerdo con esto, las especificaciones deberán estipular que se tomarán las precauciones necesarias para prevenir una pérdida importante de humedad superficial bajo tales condiciones. Entre estas precauciones está el uso de boquillas para producir niebla (no rociar agua) para mantener una brillantez de humedad en la superficie entre las operaciones de acabado. Se pueden colocar hojas de plástico antes y después del acabado con llana, de preferencia dejando expuesta solamente el área que se va a trabajar en ese momento. Menos efectivas pero útiles son ciertas películas monomoleculares rociadas en la superficie las cuales evitan la evaporación. Es deseable el uso de rompevientos, y si se usan, es deseable programar el acabado con llana después de colocarlos.

Otras prácticas útiles que puede aumentar el sangrado y contrarrestar la pérdida excesiva de la humedad superficial son: 1. usar una subbase bien húmeda, 2. enfriar los agregados, humedeciéndolos y manteniéndolos en la bomba, y 3. usar agua de mezcl fría o escamas de hielo como agua de mezclado para disminuir la temperatura del conc fresco.

Enfriamiento superficial - El enfriamiento superficial contraerá la superficie de concreto no restringido en promedio aproximadamente 10 x 10^{6} por cada ·C que dismin la temperatura. Esto significaría 9 mm en una longitud de 30 m con una reducción de ·C. La contracción se ve reducida por las restricciones y por la deformación diferida, p se inducen esfuerzos de tensión. Mientras más pronto se presente el enfriamiento secado y mientras más lenta sea la velocidad de que esto ocurra, menores serán esfuerzos de tensión. Esto se debe a la influencia relajante de la deformación diferida cual da mayor deformabilidad del concreto a las edades tempranas.

En trabajos de concreto común, la protección contra el invierno requiere evitar efectos más críticos del enfriamiento para el desarrollo de la resistencia adecuada. sistema de juntas de contracción y ranuras discutido previamente para el control agrietamiento por contracción servirá para el mismo propósito contra la disminuc importante en la temperatura superficial. Básicamente, estos controles disminuye incremento de la temperatura interior originado por el calor de hidratación, mediante el de: 1. no más cemento que el necesario, 2. puzolanas para una porción del cemento aditivos reductores del agua de mezclado, 4. inclusión de aire, 5. agregado grande, 6. revenimiento y 7. por último, pero no menos importante, donde sea posible, escamas hielo como agua de mezclado para mantener la temperatura del concreto fresco tan c de los 10 °C como sea posible. En ningún momento se quitarán las cimbras para exp la superficie caliente del concreto a las bajas temperaturas.

Decantación

Las grietas originadas por decantación se desarrollan mientras el concreto está estado plástico, después de la vibración inicial. No se debe a ninguna de las ca previamente comentadas, si no al resultado natural de que los sólidos más pesados asientan en un medio líquido.

Las grietas originadas por la decantación se presentan abajo del refuerzo horiz soportado rígidamente, de los pernos de las cimbras u otros elementos embebidos menudo las grietas aparecen en juntas de construcción horizontales y en losas p cubiertas de puentes, sobre el refuerzo o pernos de las cimbras con solamente unos p centímetros de recubrimiento. Las grietas en cubiertas de puentes pueden redu incrementando el recubrimiento de concreto. El revibrado posterior, ejecu adecuadamente puede emplearse para cerrar las grietas causadas por el asentamie mejorar la calidad y apariencia del concreto en la porción superior de tales colocació aun cuando la decantación se ha presentado y el revenimiento se ha perdido.

Construcción

Durante la construcción se puede hacer un gran esfuerzo para minimizar el agrietamiento, o en muchos casos para eliminarlo. Pero tales acciones deben ser requeridas por las especificaciones y por el cuerpo de ingenieros que las administran. Las acciones incluyen lo siguiente:

Agregados para concreto - Los agregados serán tales que produzcan concretos de alta capacidad de deformación. Los agregados finos y gruesos deben ser limpios y libres de material fino innecesario, particularmente arcilla. La arena tendrá un valor de equivalente de arena arriba de 80 por ciento, y esto se verificará frecuentemente. La arena estará almacenada por suficiente tiempo para que se estabilice el contenido de humedad a un nivel inferior a 7 por ciento sobre la base de seca al horno.

Cementos expansivos - Los cementos expansivos se pueden usar para retrasar la contracción durante el fraguado del concreto en elementos reforzados restringidos con el mínimo acero de contracción. La propiedad principal de estos cementos es que la expansión inducida en el concreto mientras fragua y endurece está diseñada para equilibrar la contracción por secado normal. Con un uso correcto (particularmente con un curado con agua amplio y a edad temprana de lo cual depende la expansión máxima), la distancia entre juntas puede algunas veces triplicarse sin incrementar el nivel del agrietamiento por contracción.

Lechadas, morteros o concretos sin contracción - Usualmente los sólidos en mezclas de lechadas, morteros y concretos se decantarán antes del endurecimiento, y parte del agua subirá hacia la superficie. Esta decantación puede ser objetable si el espacio hay que llamarlo sin dejar vacíos en la parte superior, tales como en las barras para maquinarias. Las medidas tomadas para evitar tales asentamientos ha producido lo que se conoce como lechada, mortero o concreto "sin contracción". Algunos de los materiales solamente previenen la decantación, otros además, proporcionan expansión a medida que la mezcla endurece.

Los materiales más ampliamente empleados contienen polvo de aluminio sin bruñir. Estos no contendrán estereatos, palmitatos o ácidos grasos. En una solución alcalina, como la que existe en mezclas de cemento portland, el aluminio reacciona para formar óxidos de aluminio e hidrógeno. El gas hidrógeno tiende a expander la mezcla y de esta forma evita la reducción y en muchos casos origina expansión. La cantidad de polvo de aluminio usado varía ampliamente con las condiciones, pero usualmente está entre 0.005 y 0.01 por ciento del peso de cemento. No es posible especificar un porcentaje exacto debido a que la cantidad a emplearse varía con factores tales como la temperatura, el contenido de álcalis del cemento y el consumo de cemento en la mezcla. Por lo tanto, es indispensable hacer mezclas de prueba con varios porcentajes de polvo de aluminio para encontrar el porcentaje de la expansión deseada bajo las condiciones prevalecientes. La cantidad de polvo de aluminio usado es tan pequeña que es posible mezclarlo con 50 partes de arena o ceniza volante. La mezcla tendrá suficiente volumen para que sea fácilmente medido y dispersado adecuadamente en la revoltura.

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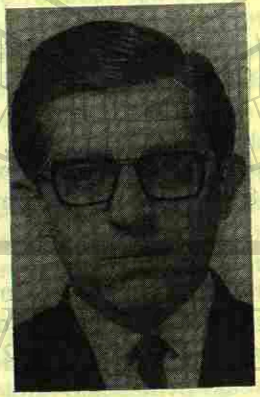

M. en C. Carlos J. Mendoza E.
Subdirector, Instituto de Ingeniería,
Universidad Nacional Autónoma de México
(UNAM), México, D.F.

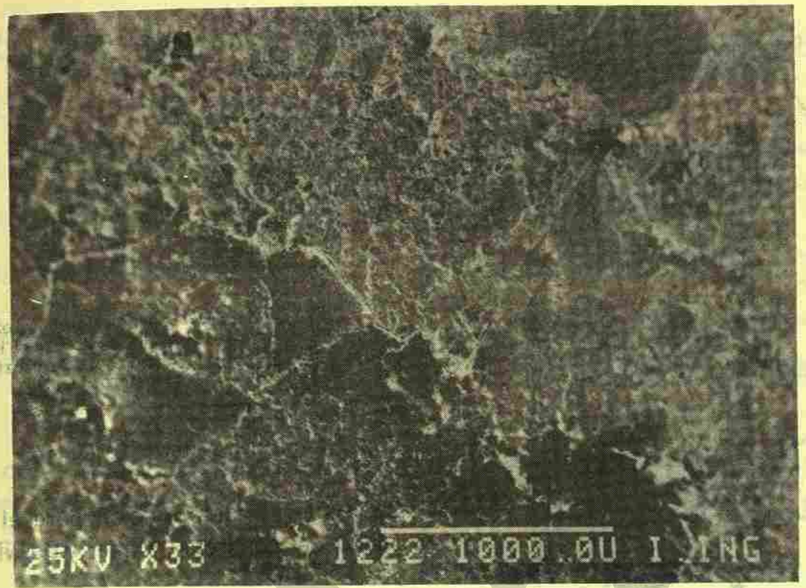

Fig 1 Agrietamiento en la interfase agregado grueso-mortero

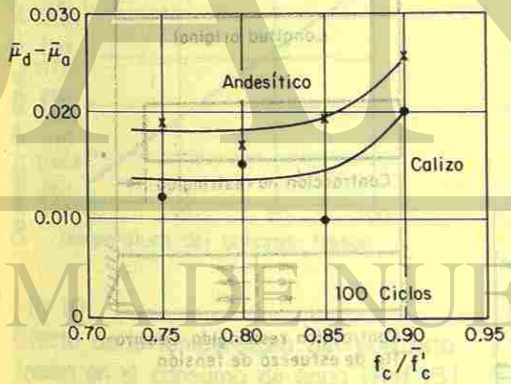

Fig 2
Incremento en la relación de Poisson con el nivel de esfuerzo máximo

Fig 3
Curvas esfuerzo-deformación para pasta de cemento a 7 días de edad (Spooner, Pomeroy y Dougill)

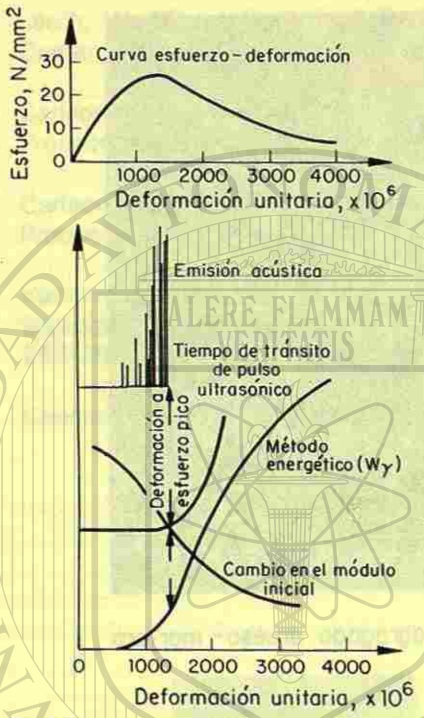

Fig 4
Comparación de métodos para detectar daños en especímenes de concreto (Spooner y Dougill)

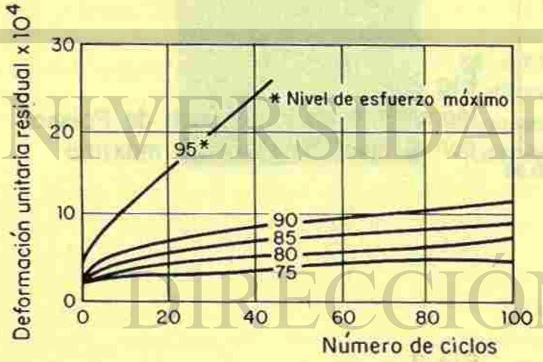

Fig 5
Incremento de la deformación unitaria residual con el esfuerzo máximo y el número de ciclos

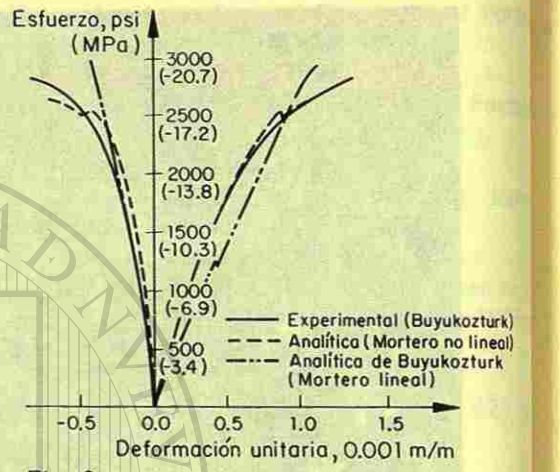

Fig 6
Curvas esfuerzo-deformación para el modelado del concreto (Maher y Darwin)

Fig 7
Agrietamiento del concreto debido a la contracción por secado (Ref 1)

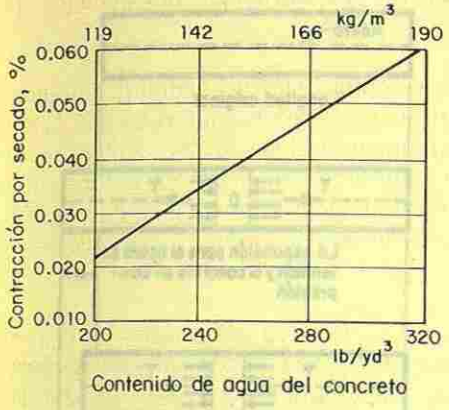

Fig 8
Efecto del contenido de agua del concreto en la contraccion por secado (Ref 13)

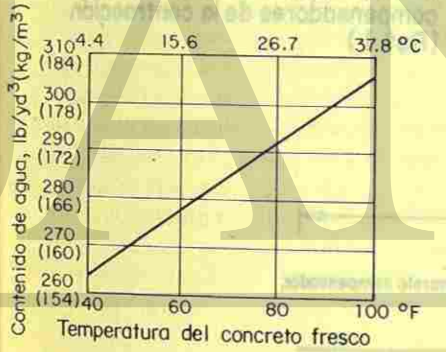

Fig 10
Efecto de la temperatura del concreto fresco en el consumo de agua (Ref 13)

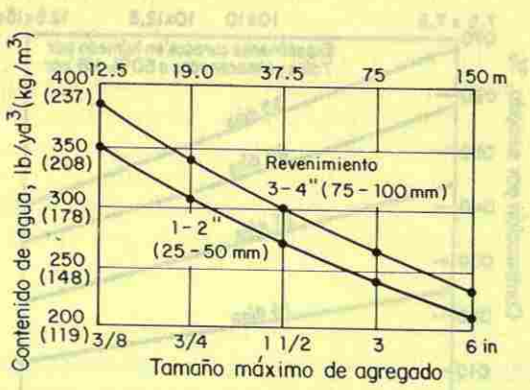

Fig 9
Efecto del tamaño del agregado en el consumo de agua de concretos sin aire incluido (ACI 211.1)

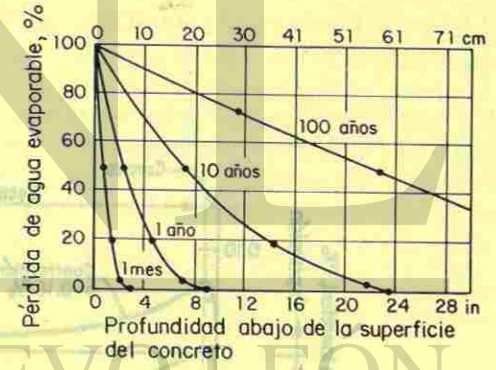

Fig 11
Velocidad de secado del concreto expuesto a 50% de humedad relativa (Ref 11)

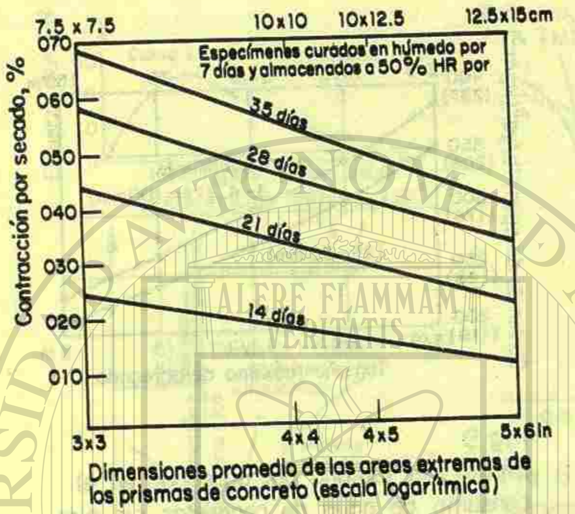

Fig 12
Efecto del tamaño de los especímenes en la contracción por secado del concreto (Ref 1)

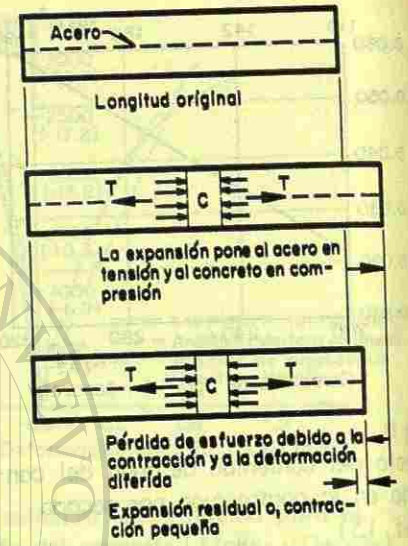

Fig 13
Concepto básico de los concretos compensadores de la contracción (Ref 1)

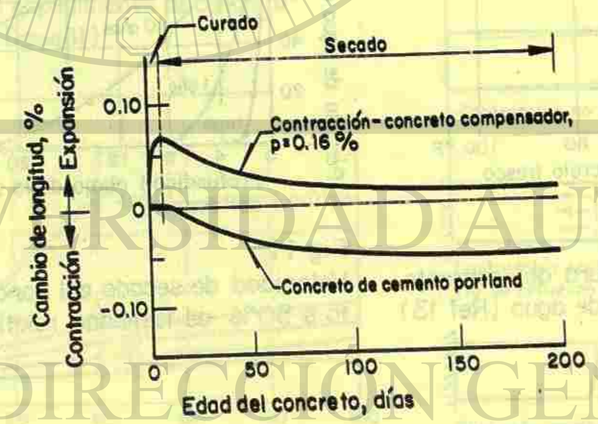

Fig 14
Características del cambio de longitud de concretos compensadores de la contracción y de cemento portland. Humedad relativa 50% (Ref 1)

ATAQUE DE SULFATOS EN EL CONCRETO UNA REVISION CRITICA

por

P. Kumar Mehta

Sinopsis: Este trabajo contiene un análisis crítico del estado actual de conocimientos so el problema del ataque de sulfatos en el concreto. Un análisis de los casos históricos estructuras sujetas a largas exposiciones a los sulfatos demuestran que, en la mayoría los casos, la pérdida de adhesión y resistencia, no la expansión y el agrietamiento, son principales manifestaciones del ataque por sulfatos. El examen microestructural muestras de concreto deteriorado confirman que, a menudo, la pérdida de adhesió resistencia está asociada con la descomposición de los productos principales de hidratac del cemento Portland. Se discutirán las hipótesis que explican las probables causas expansiones generadas por los sulfatos y se revisarán ampliamente los métodos de ens y especificaciones para la resistencia a los sulfatos. Parece ser que el daño al concr debido a causas químicas es el resultado del efecto combinado de diversos procesos con frecuencia están agrupados bajo el término de "ataque por sulfatos"; que uno de es procesos sea el más importante en una situación determinada, depende de la composic y propiedades del concreto y de las condiciones ambientales. La experiencia de campo el ataque de sulfatos demuestra que, no solamente las teorías sobre los mecanism responsables del ataque por sulfatos, sino también los métodos de ensaye y especificaciones, basados en estas teorías, están sujetos a cuestionamiento.

Palabras Clave: Ataque por sulfatos, expansión y agrietamiento, degradac descomposición, mecanismos, cristalización, expansión, métodos de ensaye.

SULFATE ATTACK ON CONCRETE: A CRITICAL REVIEW

by

P. Kumar Mehta

Synopsis: This paper contains a critical analysis of the state-of-the-art on the problem of sulfate attack on concrete. A review of the case histories of structures subjected to long-term sulfate exposure shows that, in most cases, the loss of adhesion and strength, not expansion and cracking, are the primary manifestations of sulfate attack. Microstructural examination of samples of deteriorated concrete confirms that, usually, the loss of adhesion and strength is associated with the decomposition of principal products of hydration of portland cement. Hypotheses explaining the probable causes of sulfate-generated expansions are discussed, and a comprehensive review of test methods and specifications for sulfate resistance is given. It seems that concrete damage due to chemical causes results from the combined effect of several processes which are frequently grouped under the term, "sulfate attack"; which one of these processes is the most important in a given situation depends on the composition and properties of concrete and environmental conditions. Field experience with sulfate attack shows that, not only the theories on mechanisms responsible for sulfate attack but also the test methods and specifications, based on these theories, are subject to question.

Keywords: Sulfate attack, expansion and cracking, degradation, decomposition, mechanisms, crystallization, swelling, test methods.

P. Kumar Mehta is Professor Emeritus in the Civil Engineering Department at th University of California at Berkeley. He is the author or coauthor of numerous pape on cementitious materials and properties of concrete including a textbook on th subject. A Fellow of the American Ceramic Society and the American Concre Institute, he has received several awards including the ACI's Wason Medal for materia research, CANMET/ACI award for outstanding con-tributions to research o performance of concrete in the marine environment, and Mohan Malhotra Award f research on supplementary cementing materials. He holds the Roy Carlso Distinguished Professorship in Civil Engineering at Berkeley, and has recently receive the highest campus honor, the Berkeley Citation, for exceptional contributions to h field and to the university.

INTRODUCTION

This paper is not intended to provide a comprehensive review of the publishe literature on the problem of sulfate-generated degradations in concrete. Instead, it attempted to present here a critical analysis of the state-of-the art including a review the mechanisms involved in sulfate attack. A proper understanding of the mechanism is important for designing durable concrete structures exposed to sulfate soils sulfate-bearing water. Since accelerated laboratory tests are not always helpful providing information that is applicable to long-term field behavior, the emphasis in th report is on lessons learnt from case histories of deterioration of concrete by sulfa attack rather than on laboratory test data. Also, not all sulfate-generated expans phenomena are covered. For instance, sulfate expansion resulting from the use cements or aggregates containing higher sulfate content than permitted by standa specifications are excluded from this review. Also excluded from this review a recently reported sulfate expansion phenomenon in steam-cured concrete produc Therefore, the term **sulfate attack** as used here, means deterioration involving sulfa interactions with cement paste at ordinary temperatures, when the source of sulfa ions is not internal but external (i.e., lies outside the concrete).

The physical-chemical processes involved in sulfate attack on concrete manife themselves into several forms, such as expansion, cracking, spalling, and loss strength. The strength loss is usually attributable to a reduction of adhesive ability the principal product of hydration of portland cement, namely the C-S-H; where cracking and spalling are generally associated with expansive reactions, such as formation of ettringite.[1-3] Comprehensive field and laboratory studies during 1920 1940 at the University of Saskatchewan,[4] University of California,[5] and University Minnesota[6] showed that there is a strong correlation between the C_3A content portland cement and sulfate-related expansion in concrete. As a result, in 1940, AS

introduced a specifi-cation for sulfate resisting portland cements (Type V cement, ASTM C 150), which must not contain more than 5% C_3A. For moderate sulfate resistance (150-1500 mg/liter SO_4) portland cements with less than 8% C_3A (ASTM Type II) are considered satisfactory.

A critical review of the literature shows several gaps in our knowledge. For instance, the formation of ettringite does not always lead to deleterious expansions. Are there different types of ettringite, one which is expansive and the other which is not expansive? Are there specific conditions under which alone the formation of ettringite is able to cause expansion? There is enough information on what happens on sulfate exposure to the constituents of a hydrated portland cement paste individually, but there is hardly any attempt to integrate this knowledge from the standpoint of overall physical manifestations of the chemical changes occurring simultaneously. Also, there is a tendency to treat the mechanisms underlying sulfate attack as a unique phenomenon. However, there are phenomena other than sulfate attack which are also known to cause expansion and cracking of concrete structures (viz, freezing and thawing, and alkali-silica reaction). No unified theory exists to explain the expansion and cracking of concrete from a fundamental standpoint. The specifications and test methods for sulfate resistance are subject to question because they are based on certain hypothetical mechanisms of sulfate attack which may not be entirely correct. Finally, the measures used to prevent sulfate attack do not always succeed. The proposed mechanisms underlying sulfate attack must provide satisfactory explanations for the observed anomalies.

After a brief description of the sources of sulfate ions in soil and water, a critical review of the following topics is presented in this chapter: selected case histories involving sulfate attack, mechanisms responsible for sulfate attack, specifications and test methods for sulfate resistance, and measures for prevention of sulfate attack.

Sources of Sulfate Ions

Elemental sulfur and sulfide sulfur in the form of minerals, such as sulfides of iron and copper, are found in earth's crust. Organic sulfur is present in animals and vegetable matter. Oxidation and bacterial action transform the sulfide sulfur into sulfates which find their way into soil and groundwater. As early as 1936, the U.S. Bureau of Reclamation had established that there is little or no likelihood of sulfate attack on portland cement concrete when the content of soluble sulfates in a soil is under 0.1 percent (or under 150 mg/liter sulfate in groundwater); however, there is potential for moderate attack in soils with 0.1 to 0.2 percent sulfate content (or 150 to 1500 mg/liter sulfate content in groundwater), and potential for severe attack in soils with more than 0.2 percent sulfate content (or more than 1500 mg/liter sulfate content in groundwater).

Soils containing calcium, sodium, potassium, and magnesium sulfates are the primary sources of sulfate ions in groundwater. Since the solubility of gypsum at ambient temperatures is rather low (1400 mg/liter SO_4), a higher sulfate content in a groundwater is generally indicative of the presence of magnesium and alkali sulfates This is important because, as discussed later, the severity of sulfate attack depends n only on the amount of sulfate but also on the other cations and anions present in th water. Due to evaporation, unusually high concentration of sulfate are found i seawater (2700 mg/liter), shallow lakes, mining pits, marshes, and reservoirs.

Effluents from fertilizer production facilities and drainage from fertilized soils a sources of ammonium sulfate, which is quite corrosive. In industrialized countries ashes and gaseous products from combustion of high-sulfur fuels are fast becoming major source of sulfurous and sulfuric acids, which are highly corrosive to concrete. I rainwater and coastal fogs near urban areas, disturbingly high levels of acidity (2.5-3 pH) due to SO_2 and NO_x have been recorded. An interesting case of sulfate attack du to accumulation of rainwater on a dry-packed joint, located approximately 100 fe above the ground level, is described.[3]

Selected Case Histories Involving Sulfate Attack

Since the advent of portland cement, many case histories of sulfate attack o concrete are reported in the published literature. Due to major changes in the physic chemical characteristics of portland cement and the concrete field practice, structur built during the 19th century are excluded from this review.

In North America, the Canadian prairies and some areas of the Western Unit States contain sulfate soils. Many case histories of sulfate attack on structures locat in Wyoming, South Dakota, Montana, and Southern California have been reported the published literature. Experience showed that concrete in hydraulic structures, su as canal linings, pipelines, dam galleries and floor slabs is especially vulnerable sulfate attack. According to Harboe[7] a concrete-lined, open-channel spillway in the l abutment of Alcova Dam (completed in 1938) on the North Platte River, Wyoming, h to be repaired in 1967 due to **severe spalling and erosion of the spillway floor.** Co tests showed that the compressive strength of concrete was still quite good (avera 5890 psi or 40 MPa), however the tensile strength was only 2.2 percent of t compressive strength, and the elastic modulus was about one-half of the expect value. Petrographic examination of the concrete specimen showed alteration produ of sulfate attack, such as ettringite and silica gel. The groundwater in the area has high sulfate content, and chemical analysis of water from a spring near the l abutment of the dam showed 1370 mg/liter SO_4. According to Bellport,[8] a severe ca of sulfate attack was reported in 1965 on several hydraulic structures, completed 1940, for the Belle Fourche Project, South Dakota. In this area, the soluble sulfa content of some soils is as high as 4.5 percent, and the groundwaters contain up 9900 mg/liter SO_4. **Practically no cohesion was left in the concrete of the spillw channel slab. Also, a column of the White Wood Creek Bridge was totally destroyed a result of disintegration of concrete.**

Reading[9] reported that a high-quality concrete mixture (340 kg/m³ cement w 6.8-9.4% C_3A content, < 0.5 w/c, 45 MPa average 28-d strength) was specified use in thick walls and floors at the Ft. Peck Dam, Montana, which was completed in 1936. The groundwater containing mostly sodium sulfate has a sulfate content in the range 2000-19000 mg/liter SO_4. By 1971, after 35 years of service, due to **decomposition of concrete by sulfate attack**, a large cavity (10 ft. by 5 ft. by 8 in.) appeared in the tailrace wall and had to be repaired by shotcreting. Large quantities of gypsum and ettringite were found in the deteriorated concrete.[10] In 1976, when the concrete was 40 years old, the results of an extensive investigation on concrete samples from the spillway walls, trailrace wall, and penstock floor showed that the sulfate attack was limited almost entirely to outer surfaces, areas adjacent to cracks and leaking joints and below drain holes. Core strengths were mostly in the 40 to 60 MPa range, which proved that in spite of the severe exposure the concrete was generally in good condition.

Novak and Colville[11] have reported the results of a recent investigation on floor slabs from 20-30 year old homes in Southern California. The soil and groundwater in this area contain high salt content mostly comprised of sodium salts of sulfate, nitrate, and chloride. **The deteriorated concrete exhibited several features of sulfate attack, such as expansion and cracking. However, mineralogical examination of concrete specimens showed that the characteristic products of sulfate attack, notably ettringite, thaumasite, and gypsum, were absent. Instead, large amounts of white deposits of crystalline sulfates of sodium ($NaSO_4 \cdot 10H_2O$, and Na_2SO_4) were found at the site of cracks.** The authors proposed that salt-bearing solutions must have risen to the surface by capillary action (i.e., the original concrete was of high permeability); subsequently, as a result of surface evaporation the solution phase became supersaturated and salt crystallization occurred. Salt crystallization is known to cause high pore pressures in saturated systems.[3] From the foregoing case study one may conclude, therefore, that the crystallization pressure can be a cause for expansion and cracking of concrete when suitable conditions of moisture movement, surface evaporation, and supersaturation of the solution phase are present. Under these conditions crystals of any salt, not just ettringite, when in contact with a continuous source of its supersaturated solution, would be able to exert an expansive pressure on the confining surroundings. Recent studies by Folliard and Sandberg[*] at the University of California, Berkeley shows that the polymorphic phase changes in sodium sulfate under ambient temperature and humidity conditions are a larger source of expansion than the crystallization pressure.

Price and Peterson,[12] Hurst,[13] and Hamilton and Handegord[14] have described the following case histories of sulfate attack on concrete structures in Western Canada. Occurrences of sulfate attack on spillways, conduits, tunnels, irrigation canals, and wastewater reservoirs were noted throughout the Prairie Provinces, where soils contain varying concentrations of sulfates of sodium, magnesium, and calcium. Footings,

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[*] K. Folliard and P. Sandberg, "Mechanism of Expansion Associated with Sodium Sulfate Crystallization," Paper to be presented at the ACI/CANMET Conference on Durability of Concrete, Nice, France, May 1994.

retaining walls, and slabs, which rest on moist soil and are exposed to the atmospher were found to be especially vulnerable because evaporation of moisture increases th sulfate concentration within concrete. In addition to sulfate attack, severe weatherin conditions, such as cycles of heating and cooling (40°C in summer to -40°C in winter) freezing and thawing, and wetting and drying, are believed to have contributed to th deterioration of concrete.

Price and Peterson[12] reported the case history of sulfate attack on a basemen floor in Saskatoon, constructed in 1936, which had to be replaced after 25 years o construction. The concentration of soluble sulfate in the clay beneath the floor was i the range of 0.6 to 0.8 percent. In addition to cracking, the floor gradually heave upward in the form of mounds paced at about 15-ft intervals. **The concrete was s weak that it could be broken easily with a hammer.** Also, pieces of decompose concrete were found in the spaces below the mounds. The authors also reported th case history of sulfate attack on the downstream portals of a 134-ft long concre conduit of the Little Bow Dam (constructed in 1914) located about 100 miles southeas of Calgary, Alberta. In 1965, 51 years after construction, the inlet structure was i excellent condition. However, the outlet portals had decomposed extensively, leavi the steel reinforcement exposed. It seems that seepage waters from the earthfill ha permeated the deteriorating concrete at the junction of barrel outlets and breast wa The sulfate concentration of water was 4200 mg/liter. Microscopic examination of th decomposed concrete showed the presence of ettringite and gypsum. The autho believe that **the prevalence of moisture and exposure to freezing and thawi conditions played a major role in the breakdown of concrete. Weathering weakene the concrete, making it vulnerable to more rapid permeation of water, and a stepped- rate of sulfate attack.**

According to Hurst,[13] in the Elmwood District of Winnipeg, part of a concre sewer pipe collapsed in 1910. It was established that groundwater containing hi concentration of sulfates of sodium and magnesium was percolating through t damaged structure and that **the action of percolating water had converted the concre into a putty-like mass without any strength.** Analyses of the decomposed a undecomposed concrete samples showed that the former contained a much high sulfate content. Crushed samples of undecomposed concrete, when exposed to 5 solutions of magnesium and sodium sulfate, confirmed that the concrete w vulnerable to sulfate attack. Within a few weeks the crushed material began to sw and turned into a putty-like mass. It was also found that a mixed solution of sodi and magnesium sulfate had a much more rapid action than either solution used alone

Hamilton and Handegord[14] studied the performance of several concrete structu exposed to sulfate soils. A thick floor slab made of relatively poor (weak) concre deteriorated over a 15-year period and had to be replaced. The footings below the s also showed considerable deterioration. The lower 3 to 4 in. of concrete in t footings had been reduced to a sandy, non-cohesive mass and the remainder of t concrete could be easily broken into small pieces. However, examination of concre from a tunnel at the University of Saskatchewan, after 45 years of service, indica

that even relatively permeable concrete preformed satisfactorily in sulfate soils as long as it was protected from external moisture. This observation underscores the importance of providing either a low-permeability concrete or adequate waterproofing for concrete structures in contact with sulfate soils.

Essentially, the same conclusion was reached by Mehta[3] from a comprehensive literature review of long-term performance of **concrete structures exposed to seawater** which, on the average, contains 2700 mg/liter SO_4. For instance, results from two separate investigations showed that approximately 70 years of exposure of test blocks to seawater in California[15] and in South of France[16] did not cause any deterioration when the permeability was low, in spite of the fact that portland cements containing 14-15% C_3A had been used for making concrete. Only high-permeability concretes showed deteriora-tion in the form of strength loss and erosion associated with chemical reactions involving the formation of brucite, gypsum, aragonite, ettringite, and thaumasite. The original products of hydration of cements, namely C-S-H and $Ca(OH)_2$, were not detected in the deteriorated specimens.

According to Lea,[1] it is a common observation that the deterioration of concrete in seawater is **not** characterized by expansion that is found in concrete exposed to a sulfate solution; instead, it takes the form of erosion or loss of constituents from the concrete mass. It seems that the presence of other ions in seawater, viz. 20,000 mg/liter Cl^- and 1400 mg/liter Mg^{++}, has a retarding effect on the typical expansive phenomenon that is so often encountered in the laboratory tests with high-C_3A cements exposed to Na_2SO_4 solutions. Lea notes that $Ca(OH)_2$ present in the hydrated cement paste is considerably more soluble in seawater than in plain water. He also notes that ettringite is unstable in the presence of $MgSO_4$ and eventually decomposes to form hydrated alumina, gypsum, and brucite. Furthermore, it should be noted that ettringite is unstable in the CO_2 environment and would decompose in seawater containing CO_2. Even the principal cementitious product formed by hydration of portland cement pastes, namely the C-S-H, eventually decomposes in solutions containing $MgSO_4$ or CO_2. Feld[17] reported that, in 1957, after 25 years of service, 750 precast concrete piles in Ocean City, New Jersey, had to be replaced due to heavy loss of material (550 mm original diameter reduced to 300 mm) in seawater containing more than normal CO_2 concentration.

It may be noted here that thaumasite, $CaSiO_3 \cdot CaSO_4 \cdot CaCO_3 \cdot 15H_2O$, which contains silicate and carbonate in the crystal structure, generally derives the silicate ions from the decomposition of C-S-H in an acidic environment. Therefore, any physical effects associated with the presence of thaumasite in a deteriorated concrete may have less to do with the properties of thaumasite and more to do with the decomposition of the C-S-H, which is the principal cement mineral in hydrated cement pastes.

Conclusions from Case Histories

The following conclusions can be drawn from the above case histories o deteriorated concrete in structures, which have been subjected to long-term sulfa exposure:

(1) Permeability of concrete rather than the mineralogy of cement appears t be the most important factor in sulfate attack.

(2) Sulfate attack is seldom the sole phenomenon responsible for deteriorati of concrete structures. Microstructural examination of samples o deteriorated concrete from structures exposed to severe weatheri conditions shows that microcracking and increase in permeability, as we as the presence of moisture in the environment, are conditions which a necessary for sulfate attack to occur.

(3) It is difficult to find case histories where expansion and cracking of concrete structure exposed to sulfate environment can be attributed sole to sulfate attack (although those phenomena are commonly reported fro laboratory immersion tests). Field experience shows that, in most cases the sulfate attack manifests itself in the form of loss of adhesion a strength. Long-term exposure of permeable concrete is known t transform the material into a putty-like mass, without any strength. The are cases when crystal growth has led to expansion and cracking concrete structures. However, these cases do not represent sulfate attac because the expansion is not due to ettringite formation but is usua associated with the crystallization of alkali sulfate salts from the attackin solutions.

(4) In spite of relatively high sulfate content, seawater exposure does n cause expansion and cracking from the ettringite formation. It seems th with permeable concrete the presence of chloride, magnesium and CO seawater accounts for a reduction in the alkalinity of the cement pas which leads to loss of adhesion and strength.

Review of Mechanisms Responsible for Sulfate Attack

The chemical processes involved in sulfate attack are rather straightforward, a are reviewed in several publications.[17] The principal solid phases in a well hydra portland cement paste are C-S-H, CH, and the alumina-bearing hydration produc When the amount of reactive alumina in portland cement is low, e.g. corresponding 5% or less C_3A, the amount of gypsum (5 or 6%) normally present in industrial portla cements is sufficient to convert all the reactive alumina from C_3A to ettring ($C_3A\cdot 3C\bar{S}\cdot 32H$), which is the stable sulfoaluminate phase in a sulfate environment.

However, ordinary portland cements usually contain more than 5% C_3A. When the reactive alumina in cement corresponds to approximately 8% C_3A, calculations show that the monosulfate hydrate ($C_3A\cdot C\bar{S}\cdot 12\text{-}18H$) would then be the eventual product of hydration with a portland cement containing 5% gypsum. Hydration of portland cements with more than 8% C_3A would result in the formation of calcium aluminate hydrate, $C_3A\cdot CH\cdot 12\text{-}18H$, in addition to the monosulfate hydrate. In the presence of CH, which is produced on the hydration of C_3S and C_2S, both the monosulfate hydrate and the calcium aluminate hydrate are converted to ettringite when the hydrated cement paste comes into contact with a sulfate solution:

$$C_3A\cdot C\bar{S}\cdot 12\text{-}18H + 2CH + 2\bar{S} + 12H \rightarrow C_3A\cdot 3C\bar{S}\cdot 32H$$
$$C_3A\cdot CH\cdot 12\text{-}18H + 2CH + 3\bar{S} + 11H \rightarrow C_3A\cdot 3C\bar{S}\cdot 32H$$

It is generally believed that the above reactions represent the chemistry of sulfate-related expansion and cracking associated with the ettringite formation when a sulfate solution permeates into hardened concrete made from ordinary portland cement.

An experimental study by Mehta et al[18] on C_3S pastes showed that, as a result of long-time sulfate attack, gypsum formation also caused expansion and spalling. However, in this case, the loss of adhesion and strength rather than expansion was clearly evident, and it was attributed to the decomposing action of sulfate ions on the hydration products of C_3S, namely CH and C-S-H. Since field experience shows that sulfate attack usually manifests itself in the form of loss of adhesion and strength, any hypotheses attempting to explain sulfate attack from the standpoint of ettringite-generated expansion alone have to be considered as incomplete.

The physical mechanism, or mechanisms, by which ettringite causes expansion and cracking, is a matter of controversy. Topochemical formation of ettringite with directional crystal growth and swelling of ettringite by water adsorption are among the two proposed hypotheses. Experimental verification of these hypotheses has been difficult because it requires the observation of processes that leave behind only the effects, but no trace of the actual causes which produced the effects.[19] The following discussion on the various hypotheses explaining the mechanisms responsible for sulfate-related expansion and cracking in concrete, is based on a recent publication by Odler and Jawed.[20]

Topochemical Reactions and Directional Crystal Growth

A topochemical reaction involves a solid-state hydration mechanism in which the product is formed at the surface of one of the solid phases involved in the reaction and grows in a direction perpendicular to the original solid-liquid interface. This is opposed to the through-solution hydration mechanism in which the reaction product crystallizes randomly within the whole volume of the liquid phase of the system. It was Lafuma[21] who suggested first that a solid state reaction between C_3A and sulfate ions could cause expansion. Among others, support for this hypothesis has come from the work of Hansen[22] and Bentur and Ish-Shalom.[23] Also, Ogawa and Roy,[24] observed that, in hydrated expansive cement pastes, radially oriented crystals of ettringite were found around C_4A_3S particles. However, since the crystal structures of the reactants (C_3A or C_4A_3S) and the reaction product (ettringite) are totally different from each other, and since direct microstructural observations showed that in most cases ettringite crystals precipitate randomly in hydrated cement pastes, several investigators including Mehta[19,26] have questioned the validity of this hypothesis.

Since the formation of ettringite in sulfate attack on concrete occurs as a result of reactions involving the **hydration products** of C_3A, a more broad view of the topochemical reaction mechanism is presented by Odler and Jawed.[20] According to this approach, the sulfate and calcium ions from the solution phase move on to the surface of the aluminate phase (including $C_3A{\cdot}CS{\cdot}12\text{-}18H$ and $C_3A{\cdot}CH{\cdot}12\text{-}18H$) and react to form ettringite just as the surface dissolves to produce the aluminate ions. Since the volume of ettringite formed is larger than the volume of the reactant solid, it is assumed that the reaction will result in an overall expansion of the system. But the formation of ettringite consumes a large amount of water and, in a sealed system, if the water-filled space is considered a part of the space occupied by the reactants, calculations show that there would be a net chemical shrinkage. However, in an open system, the newly formed pore space (as a result of the chemical shrinkage associated with the ettringite formation), may be partially or totally filled with water if the concrete is in contact with an outside source of water and, in this case the pressure caused by an anisotropic crystal growth cannot be ignored. The authors[20] note that even oriented crystal growth of ettringite from a topochemical reaction would not always result in expansion, such as when concrete is in the plastic state or when concrete is sufficiently rigid to withstand stresses generated by the topochemical formation of ettringite.

Swelling by Water Adsorption

From a review of the influence of electrolytes and other factors on sulfate expansion, Thorvaldson[25] has suggested that the volume changes are controlled by osmotic forces involved with the shrinkage and swelling of a microcrystalline ettringite. The chemical reactions condition the microcrystalline product and destroy the cementing substances, while the formation of new crystalline products, as a result of these reactions, is incidental to them and not the primary cause of expansion. The hypothesis was based on observations from a laboratory study in which mortar bars made with a high-C_3A cement (25% C_3S, 50% C_2S, 24% C_3A) disintegrated in a 2% Na_2SO_4 solution, without showing much expansion, although microscopic examination of disintegrated samples showed very large quantities of ettringite present.

Based on experimental observations reported in several papers,[26-28] Mehta and his associates have advanced Thorvaldson's hypothesis further. It is proposed that, under **certain conditions**, the swelling of ettringite by water adsorption, rather than the formation and crystal growth of ettringite, is probably the primary mechanism responsible for sulfate-related expansions. What are the conditions? First, the system should have a sufficient supply of sulfate, hydroxyl, and calcium ions which are necessary for the formation of poorly crystalline ettringite. Second, the concrete must be in a moist environment and sufficiently permeable (as a result of interconnected large pores, or microcracks caused by weathering and other causes) so that an adequate supply of water is available for adsorption onto the large surface of ettringite crystals, which are held together by physical forces of attraction only (i.e., van der Waal's bonds). Third, the elastic modulus or the stiffness of the system should be low so that the pressure exerted by the swelling of ettringite crystals manifests itself into a volumetric expansion. Note that the substitution of hydroxyl ions by sulfate and other acidic ions in a hydrated cement paste would result in the loss of adhesion and stiffness, attributable largely to the decomposition of C-S-H. The formation of gypsum under these conditions may, therefore, be incidental and not one of the causes of expansion.

This hypothesis has several points in its favor. **First**, it explains satisfactorily why supersulfated cements, which derive their strength by the formation of relatively large quantities of rather big crystals of ettringite (up to 100 μm long and several micrometer thick), show little or no expansion. **Second**, it explains satisfactorily why it takes many months before the cements containing 8-10% C_3A begin to show any significant expansion in sulfate immersion tests (such as ASTM C 1012). It is generally observed from the results of sulfate immersion tests that there is no direct correlation between the amount of ettringite formed and the rate of expansion. As long as the strength and the elastic modulus of the cement paste remain high, the ettringite formation alone is not sufficient to cause any significant expansion. Large expansions due to water absorption by ettringite are registered only when the system loses its strength considerably. As long as the cement paste environment remains saturated with hydroxyl ions, the C-S-H--which is the primary source of strength--continues to be stable and strong. When the hydroxyl ions are replaced by the sulfate ions, a stage which is indicated by the disappearance of CH and the appearance of gypsum in the cement paste, the C-S-H loses its adhesion and strength. And this is the condition under which expansion of microcrystalline ettringite by osmotic forces will be possible. Note that, compared to the hypothesis explaining the sulfate attack phenomenon from topochemical formation and directional crystal growth of ettringite, the osmotic pressure hypothesis is comprehensive in the sense that it takes into consideration the effect of sulfate ions on all the constituents of the hydrated cement paste. **Third**, the hypothesis explains satisfactorily the results from laboratory studies showing that sulfate attack is usually accompanied by the loss of strength and mass. Also, sulfate expansions are accelerated in the presence of certain other ions, such as chloride and magnesium. The chloride ion attack has the ability of dissolving the solid calcium hydroxide from a hydrated cement paste and eventually lowering the hydroxyl ion concentration, which is detrimental to the strength of C-S-H. Magnesium salt solutions react with calcium hydroxide to form the relatively insoluble brucite (magnesium hydroxide), which also has the effect of lowering the hydroxyl ion concentration. **Fourth**, it explains satisfactorily why sulfate expansion does not occur in a dry environment. Investigations show that concrete deterioration by expansion and cracking is usually associated with more than one cause and that microcracking as well as moisture ingress into concrete are the two necessary prerequisites for almost eve expansive phenomenon, such as freezing and thawing, corrosion of reinforcing stee and swelling of alkali-silicate gels.

Specifications and Testing Methods for Sulfate Resistance

As will be discussed here, the absence of a conclusive hypothesis on th mechanisms by which concrete deteriorates from long-term sulfate exposure ha prevented the development of appropriate acceptance tests and specifications f sulfate resistance. Since it is easier from laboratory tests to prove a direct correlati between the C_3A content of a portland cement and expansion in mortar prisms expose to a sulfate solution, most of the specifications and test methods have ignored the los of strength and elastic modulus which are also associated with the sulfate attac During the early part of this century sulfate resisting portland cements, with low-C content (0-5% C_3A), were developed in several countries, such as by Ferrari in Ita and by Fleming in Canada. In 1940, ASTM C 150--the Standard Specification f Portland Cement--approved a sulfate resisting cement which is required to contain le than 5% C_3A. Since mortar prisms made with the sulfate resisting portland cemen show a little or no expansion on immersion in a dilute sulfate solution for perio ranging from several weeks to several months, this approach has become the basis f specifications and acceptance tests for sulfate resistance.

For a laboratory test to be really meaningful, the following criteria ought to satisfied: (a) it should yield reliable and reproducible information within a relativ short time (e.g., 4 weeks or less) if the test is to be of practical value to a ceme manufacturer or user; (b) it should correlate well with the field experience, i.e. sho reproduce as accurately as possible the mechanisms by which the sulfates attack cement hydration products; (c) it should be applicable to both portland and blen portland cements since both cement types are commonly specified for use un conditions of sulfate attack.

Early experiments showed that mortar bar expansions in sulfate soluti depended greatly on permeability, therefore the use of graded sand, high water/ceme ratio, and low cement content was favored for the purpose of accelerating the ti needed for sulfate expansion. For instance, Thorvaldsen et al[4] developed a meth according to which 1.55 by 1.55 by 10-cm mortar prisms, containing 20-30 mesh s with 1:10 cement-sand ratio, cured for 3-8 weeks, were exposed to a sulfate solu and the time required to show a given amount of expansion was measured. Based the results of a long-term laboratory investigation by the authors, the relative resista of some synthetic mixtures of portland cement compounds to solutions contai 1.8% $MgSO_4$, 2.1% Na_2SO_4, and a saturated $CaSO_4$ solution, is shown in Table 1.

From the data in Table 1, it should be noted that the $MgSO_4$ solution was foun be more aggressive than the Na_2SO_4 solution due to the former's pH lowering pote (associated with the chemical reaction involving decomposition of calcium hydro and precipitation of brucite and gypsum). Therefore, for application to those

conditions where the pH lowering phenomenon is not involved during sulfate attack, the use of a Na_2SO_4 or a $CaSO_4$ solution is more valid. In the Na_2SO_4 and $CaSO_4$ solutions, the two cements with no C_3A took a long time to register expansion. However, with the C_3S-C_2S cement a very rapid expansion was recorded after 12 years of exposure in the Na_2SO_4 solution. This expansion was probably preceded by slow transformation of calcium hydroxide and C-S-H to gypsum, and a corresponding loss in stiffness. The high C_3A cement responded rapidly to the sulfate expansion phenomenon in all three test solutions.

The ASTM Type I portland cements usually contain 8 to 12% C_3A and, therefore, take many months to show significant expansions in dilute sulfate solutions. Wolochow[28] used 1:4 cement-to-sand mortars, cured for 7 days, and exposed them to a 5% Na_2SO_4 solution. The results from an interlaboratory testing program showed that a 28-days immersion period was sufficient to detect portland cements of low resistance to sulfate attack. Since the reproducibility between different laboratories was not satisfactory, and since it was found difficult to agree upon a permissible expansion limit between the laboratory test results and the field behavior, the test did not become popular.

In immersion tests, considerable time is lost before the sulfate solution is able to permeate into the interior of a mortar or concrete specimen. In 1960, an accelerated test procedure was adopted by ASTM (Standard Test Method C 452) according to which the 14-day expansion is measured on 25 by 25 by 285 mm, water-cured mortar bars made from a mixture of portland cement and added gypsum, such that the total SO_3 content of the mixture before the hydration is 7%. The method does what it is supposed to do, i.e., in a relatively short period of 14 days it is successful in differentiating between the high-C_3A and the low-C_3A portland cements by accelerating the expansion associated with the ettringite formation. However, it does not simulate the field exposure conditions typical of the sulfate attack. **Instead of chemical interaction between an external sulfate solution and the hydrated constituents of cement paste, this method involves chemical interaction between anhydrous cement compounds and an internally incorporated source of sulfate ions.** Thus, the method is unsuitable for testing blended portland cements because, compared to ordinary portland cement, the blended cements have to be well hydrated before they develop sulfate resisting characteristics. Even for evaluation of the sulfate resistance of neat portland cements, the method has a limited value because it is much easier to determine the relative C_3A content of a portland cement by other ways than by this procedure. More importantly, it fails to take into consideration that the long-term deterioration of concrete by sulfate exposure involves not only the sulfate expansion phenomenon associated with the ettringite formation but also the loss of adhesion and strength associated with the formation of gypsum from CH and C-S-H.

The motivation to develop a relevant test procedure resulted in studies which culminated in a new ASTM Standard Method for Length Change of Hydraulic Cement Mortars Exposed to a Sulfate Solution, ASTM C 1012. Based essentially on Wolochow's[28] sulfate immersion test for expansion of mortar prisms, the procedure involves making a mortar mixture (1:2.75 cement-sand ratio, 0.485 water/cementitious ratio), molding 25 by 25 by 285 mm specimens, curing the specimens in warm water (35°C) for 24 hours before demolding, subsequently curing them in saturated lime water at 23°C until such time that a mean compressive strength of approximately 20 MPa or more has been achieved, and then immersing the prisms in a standard sulfate solution for a period of six months. The immersion solution consists of 0.352 mol or 50 g/liter of Na_2SO_4. Alternatively, a mixture of 0.176 mol/liter of each of sodium and magnesium sulfates may be used.

The ASTM C 1012 Test Method is found suitable for both portland and blended-portland cements. According to the ASTM C 595-89, Standard Specification for Blended Hydraulic Cements, the criterion for failure in the ASTM C 1012 is 0.1% expansion. Test data reported by Mather[29] showed that a portland cement with 9.4% C_3A took more than 5 months to reach 0.1% expansion. Using nine different fly ashes, the portland-fly ash cements containing 30% fly ash showed that five of the cements failed to meet the specification limit (0.1% expansion) in 6 months of sulfate exposure; however the four cements which met the specification time in 6 months immersion thereafter continued to show significant expansion (Fig. 1). Therefore, the long time to get the results is one of the shortcomings of this test. Also, no attempt is made to evaluate the loss of stiffness and strength from sulfate attack. This is indicated in the Figure by the continued expansions beyond the 180-day test period. After 202 days of sulfate immersion, a blended cement containing 30% of a Class F fly ash (Fly Ash No. 511) reached the failure criteria (0.1% expansion), and thereafter continued to show considerable expansion. This is inconsistent with the general field experience with Class F fly ashes and, therefore, both the validity of the test method and the specification are subject to question.

Instead of measuring expansion for sulfate attack, several researchers have developed procedures based on determination of strength loss. Koch and Steineger used 1 by 1 by 6-cm mortar prisms (1:4 cement-sand ratio), cured in water for 21 days and subsequently immersed in a Na_2SO_4 solution containing 2.5% SO_3. The relative sulfate resistance of a cement is assessed from changes in the flexural strength at different ages up to 77 days of sulfate immersion. Markestad[31] reported pronounced changes in the compressive strength of high-C_3A portland cement mortar cubes (12.5 mm, 1:3 cement-sand ratio), normally cured for 28 days, then immersed for 7 days in a magnesium sulfate solution containing 0.5% SO_3. Using similar mortar cubes Forrester[32] found significant changes in the compressive strength of portland cements containing 9 or 5% C_3A, when immersed for 90 days in a Na_2SO_4 solution containing 3.5% SO_3.

Clearly, the strength loss measurements provide a comprehensive assessment of the deterioration of a hydrated cement paste due both to reduction of stiffness (i.e. attack on CH and C-S-H) and microcracking (i.e., excessive expansion on the ettring formation from alumina-bearing products). However, the 3 to 4 months needed to g the test results from mortar prisms or cubes, although small in dimension, is still t long. According to Biczok,[2] Merriman had described a qualitative test for sulfa

resistance, using **neat cement paste** specimens (0.26 water/cement ratio) cured for 3 days, and thereafter suspended in a 10% solution of Na_2SO_4. Daily the sulfate content of the test solution is brought back to the original concentration by neutralizing it with sulfuric acid. This tends to accelerate the sulfate attack by a mechanism which is discussed later. At the end of the 28-days immersion, a visual estimate of the deterioration is obtained. It may be noted that due to the short curing period, the Merriman test is not applicable to blended portland cements.

To make the Merriman Test quantitative and applicable to both portland and blended portland cements, Mehta and Gjorv[33] used small cubes (12.5 mm) of high-porosity cement pastes (0.5 water/cementitious ratio), which are subjected to accelerated curing at 50°C for 7 days before immersion in a 4% Na_2SO_4 solution (2.1% SO_3). Once in every 24h period, the pH of the immer-sion solution is brought back to the original level by manual titration with 1N H_2SO_4 . Using five different types of cements, with known field performance history to sulfate attack, it was shown by the authors[33] that a 28-days immersion period was sufficient to differentiate between the good and the bad cements on the basis of the compressive strengths data before and after the immersion.

Subsequently, Mehta[34] automated the sulfuric acid titration procedure by using a pH-stat, which permits a continuous control of the pH of the immersion solution by releasing a few drops of 0.1N H_2SO_4 solution whenever the hydroxyl ion concentration of the solution goes up to indicate alkaline pH. Later, Brown[35] was able to establish conclusively that the maintenance of the pH of a sulfate solution (in which portland cement mortar specimens were immersed) at a constant and predetermined value through controlled sulfuric acid additions, ensured that the sulfate ion concentration in the solution remained invariant with time. The author compared the rates of sulfate attack both with and without the environmental control (i.e., the pH control), and reported that the environmental control significantly increased the rate of sulfate attack as measured by either strength loss or linear expansion. This is because in conventional immersion tests as a result of dissolution of CH, within a few hours after immersion of test specimens, the hydroxyl ion concentration of the solution rises from about 6-7 (which is the typical pH of a solution of technical grade Na_2SO_4) to 12-12.5. At this pH, only the ettrringite formation can occur; the gypsum formation and the C-S-H decomposition do not take place until the sulfate solution has a lower pH (e.g., 8-11.5 pH range). Since field experience with structures exposed to sulfate waters that are regularly replenished often shows the presence of gypsum and the absence of C-S-H and CH, it should be apparent that the laboratory tests involving stagnant sulfate solutions (i.e., without control of pH and sulfate concentration) are not only time-consuming but also have limited validity.

Recent work by Reardon[36] confirmed the significance of contributions by Mehta[34] and Brown[35] to the development of a suitable accelerated test for sulfate resistance. Using a chemical model, the author simulated the phase changes that occur with the progressive addition of sulfuric acid to a kg of water containing one mole each of CH, C-S-H (with Ca/Si ratio of 2.12), and C_4AH_{13}. Such a simulation is relevant to the understanding of mineralogical changes that occur in concrete as a result of attack by atmospheric sulfur dioxide. With decreasing pH, the following order of phase changes is indicated: in the 12-12.5 pH range, CH and C_4AH_{13} (and possibly the monosulfate hydrate) dissolve and ettringite precipitates; in the 10.6 to 11.6 pH range, gypsum precipitates; at 10.6 pH, ettringite decomposes to form gibbsite and gypsum; and at 8.8 pH, C-S-H decomposes to form silica gel. The model predicts an overall decrease in the Ca/Si ratio from 2.12 at a pH of 12.5, to a Ca/Si ratio of 0.5 at a pH of 9 below which C-S-H will decompose to form amorphous silica (Fig. 2). Thus, at pH values lower than 12.5 the changes in the composition of C-S-H resulting in eventual decomposition have obvious implication to the loss of adhesion and strength of products which contain C-S-H as the principal mineral after cement hydration. Since portland-pozzolan and portland-slag cements are believed to contain C-S-H with much lower Ca/Si ratio (viz., Ca/Si ≈ 1) than hydrated portland cements, the former are inherently more stable to acidic solutions, and even to solutions in the 9 to 11 pH range (Fig. 2).

Measures for Prevention of Sulfate Attack

Assuming a situation where sulfate water cannot be prevented from coming into contact with a concrete structure, the only defense against sulfate attack would lie in the control of concrete quality, especially the permeability. High cement content, low water/cement ratio, proper compaction and curing, and control of cracking during service are among the important factors that contribute to low permeability of concrete. If cracking in service is unavoidable due to one or more of the several possible causes (such as drying shrinkage, thermal shrinkage, cycles of freezing and thawing, and corrosion of reinforcing steel), additional safeguard can be provided by the use of sulfate-resisting cements and/or mineral admixtures in concrete. On the other hand, by controlling the chemistry of cement alone it is not possible to provide long-term protection to a permeable concrete which is exposed to moderate or high sulfate concentrations, as discussed next.

With moderate conditions of sulfate attack (up to 0.2 sulfate content of soil, or up to 1500 mg/liter sulfate in water), experience shows that ASTM Type II portland cement (less than 8% C_3A) can perform satisfactorily provided the water/cement ratio of the concrete is held below 0.5. With severe conditions of sulfate attack (0.2 to 2% sulfate in soil, or 1500-10000 mg/liter sulfate in water) ACI Building Code 318 recommends the use of ASTM Type V portland cement (less than 5% C_3A), and a water/cement ratio below 0.45.

The control of C_3A content for the purpose of sulfate resistance, which was incorporated in 1940 into the ASTM C 150 Standard Specifications for Portland Cement, addresses only the problem of sulfate expansion associated with the ettringite formation. A low-C_3A cement does not eliminate the susceptibility to sulfate attack of other phases of hydrated cement paste, such as CH and C-S-H, and the subsequent loss of strength. For example, Gjorv[37] found that the flexural strength of concrete test blocks made from several portland cements, including a cement with 3% C_3A, were

significantly affected on 30 years of exposure to seawater. As discussed next, the use of portland-pozzolan or portland-slag cements offers a better solution to the overall problem of sulfate attack, and field experience seems to confirm this view.[37] Note that instead of using a portland-pozzolan or a portland-slag cement, the same purpose is served when a pozzolan or a slag is used in the concrete mixture as a partial replacement for cement.

The source of sulfate concentrations below 1500 mg/liter in groundwater may be gypsum in the soil. However, sulfate concentrations significantly larger than 1500 mg/liter are generally associated with the presence of magnesium, sodium, or potassium sulfates. Long-term field experience and accelerated laboratory tests show that under these conditions the control of C_3A content of cement alone does not offer an effective protection against the exchange reactions involving decomposition of CH and C-S-H. Since the dissolution of CH begins to take place as soon as the pH of the environment drops below 12.5, and the dissolution of C-S-H does not occur until the pH drops below 8.8, the CH is more vulnerable to sulfate attack. This is why blended cements containing a little or no CH on hydration perform better under severe conditions of sulfate attack. Examples of such cements are: calcium aluminate cement, portland blast-furnace slag cements with more than 70% slag, and portland-pozzolan cements with at least 25 percent of a highly siliceous pozzolan (such as volcanic ash, calcined clay, and low-calcium fly ash). It may be noted that, besides reducing the CH content, the incorporation of a pozzolan or slag helps to reduce the permeability of the cement hydration product by a pore-refinement mechanism.[3] It should also be remembered that normal pozzolans hydrate rather slowly and, therefore the beneficial effects from the pozzolanic reaction will not be available until a concrete mixture it has been cured for a sufficient length of time (viz, 4 to 6 weeks). Poor performance of pozzolanic or slag cements, prematurely exposed to aggressive solutions, is usually attributable to inadequate curing and not to any inherent deficiency in these cements.

Pozzolans and slags also vary widely in physical and chemical characteristics, and consequently in their ability to prevent sulfate attack. Examples described by the following paragraphs show that a proper understanding of both the mechanism of sulfate attack on concrete and the mechanisms by which pozzolans and slag improve the sulfate resistance is essential to insure a satisfactory performance of the materials selected for use in concrete.

Rosner et al[38] showed that, compared to a reference portland cement, the use of a high-calcium fly ash, irrespective of the proportions used, either failed to cause any improvement or caused a reduction in the sulfate resistance of test mortars made with blended cements. Mehta[39] used 16 different fly ashes to investigate the effect of fly ash composition on the sulfate resistance of blended cements containing a Type I portland cement (11% C_3A), and 25 or 40% fly ash. The calcium, alumina and sulfate contents of the fly ashes varied widely. When the mineralogical composition of the fly ash was such that the formation of ettringite occurred in the hydrated pastes of the blended cement even before the immersion in the sulfate solution, the blended cement performed well in the test. On the contrary, when prior to the sulfate exposure the hydrated paste of the blended cement contained alumina-bearing phases that are known to be vulnerable to sulfate attack (viz, the monosulfate hydrate), the cement performed poorly in the test. The X-ray diffraction patterns of hydrated pastes of blended cements containing two different fly ashes are shown in Fig. 3. The cements containing Fly Ash No. 6 (which, upon hydration, formed ettrringite instead of the monosulfate hydrate) performed well in the sulfate immersion test, whereas the cements containing Fly Ash No. 10 (which, upon hydration, formed the monosulfate hydrate) performed rather poorly.

Probably for similar reasons, the sulfate resistance of portland-blast furnace slag cements is dependent on the composition of the slag and the slag content of the cement. Based on the findings of many researchers, ASTM C 989-1985 contains the following summary of the state-of-the-art on the sulfate resistance of blended cements containing ground granulated blastfurnace slag:

> *Effect of slag on sulfate resistance* - The use of slag will decrease the C_3A content of the cementing materials and decrease the permeability and calcium hydroxide content of the mortar or concrete. Tests have shown that the alumina content of the slag also influences sulfate resistance, and that high alumina content can have a detrimental influence at low slag-replacement percentages. The data from these studies of laboratory exposure of mortars to sodium and magnesium sulfate solutions provide the following general conclusions.
>
> The combinations of slag and portland cement in which the slag content was greater than 60 to 65%, had high sulfate resistance, always better than the portland cement alone, irrespective of the Al_2O_3 content of the slag. The improvement in sulfate resistance was greatest for the cements with the higher C_3A contents.
>
> The low alumina (11%) slag increased the sulfate resistance independently of the C_3A content of the cement. To obtain adequate sulfate resistance higher percentages were necessary with the higher C_3A cements. The high alumina (18%) slag adversely affected the sulfate resistance of portland cements when blended in low percentages (50% or less).

As stated before, the hydrated cement pastes containing approximately 70% sla are expected to contain a little or no free calcium hydroxide. It is the calcium hydroxide which, on sulfate exposure, contributes greatly to sulfate attack b promoting the formation of gypsum and microcrystalline ettringite. Also, th impermeability of portland-slag cement pastes increases with increasing slag content.

Stark[40] reported the results of five years of sulfate exposure of concrete specimens exposed to a sulfate-rich soil at a testing facility in Northern California. Concrete mixtures made with an ASTM Type II portland cement (4% C_3A) showed resistance to sulfate attack that was equivalent to corresponding concretes made with a Type V portland cement. Fly ash and ground granulated slag, when used as cement replacement materials, generally reduced the sulfate resistance of concrete. According to the author, a possible reason for this inconsistency in results (compared to the general experience of many other researchers) may lie in the fact that concrete prisms in the Northern California test facility are subjected to wetting and drying cycles in sulfate water, whereas most other conclusions are based on continuous immersion. It was observed that the deterioration due to sulfate attack was confined to the upper portion of the concrete prisms where drying had taken place, while the lower portion of the concrete prisms continued to remain in perfect condition. Stark[40] concluded that it was the salt weathering phenomenon (i.e. crystallization of sodium sulfate salt), rather than the classical sulfate attack (i.e. ettringite formation), which caused the concrete deterioration at the California testing facility. This shows that testing conditions must be carefully evaluated before the results from two different test methods are compared.

In conclusion, with regard to sulfate attack the anomalous behavior of pozzolan or slag cements in field or laboratory tests reported in the literature does not appear to be due to something which is inherently wrong with these cements. The anomalous behavior is usually due to the use of improper tests methods or mix proportions, or due to inadequate curing of concrete before the sulfate exposure. Only when, in some exceptional cases, the mineralogical composition of a mineral additive has been unsatisfactory, its use resulted in poor performance of the concrete on sulfate exposure.

CONCLUSIONS

Many natural and industrial waters contain sulfate ions in concentration that can be deleterious to concrete structures on long-term exposure. Effluents from fertilizer production facilities and drainage waters from heavily fertilized soils are sources of ammonium sulfate which is quite corrosive. New sources of sulfate-related corrosion in industrialized environments are the ashes and gaseous products from combustion of high-sulfur fuels. For instance, high levels of acidity in rainwater and fogs due to SO_2 and NO_x have been reported from many urban and industrial areas.

A review of case histories of deteriorated concrete in structures which have been subjected to long-term sulfate exposure shows that only permeable concretes in moist environments are vulnerable, and that the sulfate attack is seldom the sole phenomenon responsible for concrete deterioration. Microcracking of concrete, increase in permeability, and a high level of water saturation are generally preceded by sulfate attack and other types of chemical as well as physical attacks, which result i expansion and cracking. Field experience shows that in most cases the sulfate attac manifests itself in the form of loss of adhesion and strength by the cement past present in concrete.

Topochemical formation of ettringite with directional crystal growth, and swellin of ettringite by water adsorption are two hypotheses that have been proposed amon others to explain expansion and cracking. Experimental verification of thes hypotheses has been difficult because it requires the observation of processes tha leave behind only the effects but not the actual causes which produced the effect Although, under certain environmental conditions, directional crystal growth of salt form a permeating solution is known to cause expansion and cracking, it is conclude that osmotic forces involved with the swelling of poorly crystalline ettringite in concret are probably important in sulfate expansion after the concrete has been weakened b decomposition of CH and C-S-H, as a result of sulfate attack. This hypothesis has th benefit of being more universal in the sense that it involves sulfate attack on a components of the cement paste (rather than on the alumina-bearing phases alone and it has elements that are common to other expansive phenomena in concret technology.

The absence of a conclusive hypothesis on mechanisms responsible for sulfat attack on concrete has prevented the development of appropriate test methods an specifications for sulfate resisting cements. For a variety of reasons, discussed in th report, the current ASTM standard test methods and specifications are not entire satisfactory. Recent work shows that, both from theoretical and practical standpoin immersion tests, involving continuous control of the pH of the sulfate solution, woul be more suitable for adoption as accelerated laboratory tests for evaluation of sulfat resistance of cements.

A review of the measures for prevention of sulfate attack shows that control o the permeability of concrete is more important than control of the chemistry of cemen The use of a low water/cement ratio and good quality pozzolans or slags in appropriat amounts, therefore, provides a better approach to the problem of sulfate attack tha the use of a sulfate resistance portland cement alone. Enough information is no available on the mineralogy and proportions of pozzolans or slags needed for long-ter durability of concrete to sulfate environments.

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Table 1. Relative Rates of Sulfate Expansion of Mortar Prisms, % (Reference 4)

Compound composition of cement	1.8% $MgSO_4$	2.1% Na_2SO_4	Saturated $CaSO_4$
80% C_3S + 20% C_3A	0.5% in 4 days	0.5% in 7 days	0.5% in 11 days
40% C_3S + 40% + 20% C_4AF	0.5% in 43 days	0.06% in 3 years	0.07% in 3 years
50% C_3S + 50% C_2S	0.5% in 65 days	0.04% in 12 years, then more rapid expansion	0.19% in 18 years

P. Kumar Mehta, Professor Emeritus in the Civil Engineering Department at the University of California at Berkely

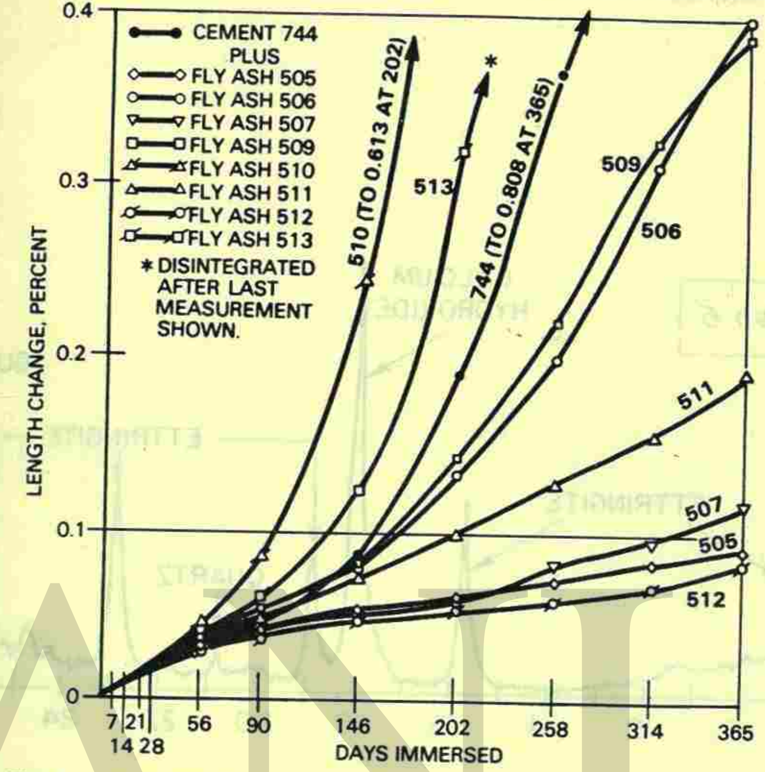

Fig. 1. Length change on sulfate immersion in cement-fly ash mixtures containing 30% fly ash (Reference 29)

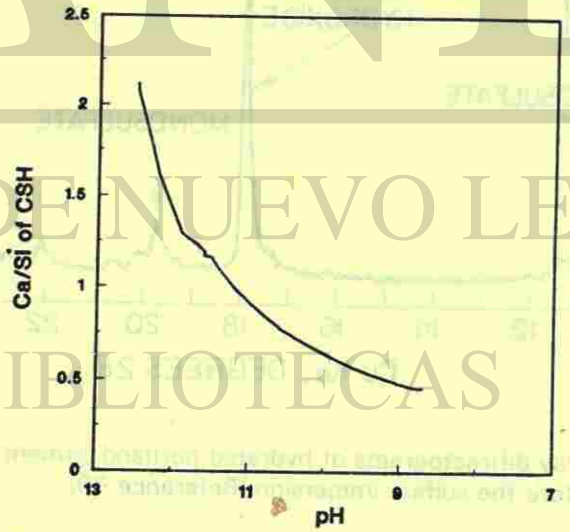

Fig. 2. Change in C-S-H composition with pH (Reference 36)

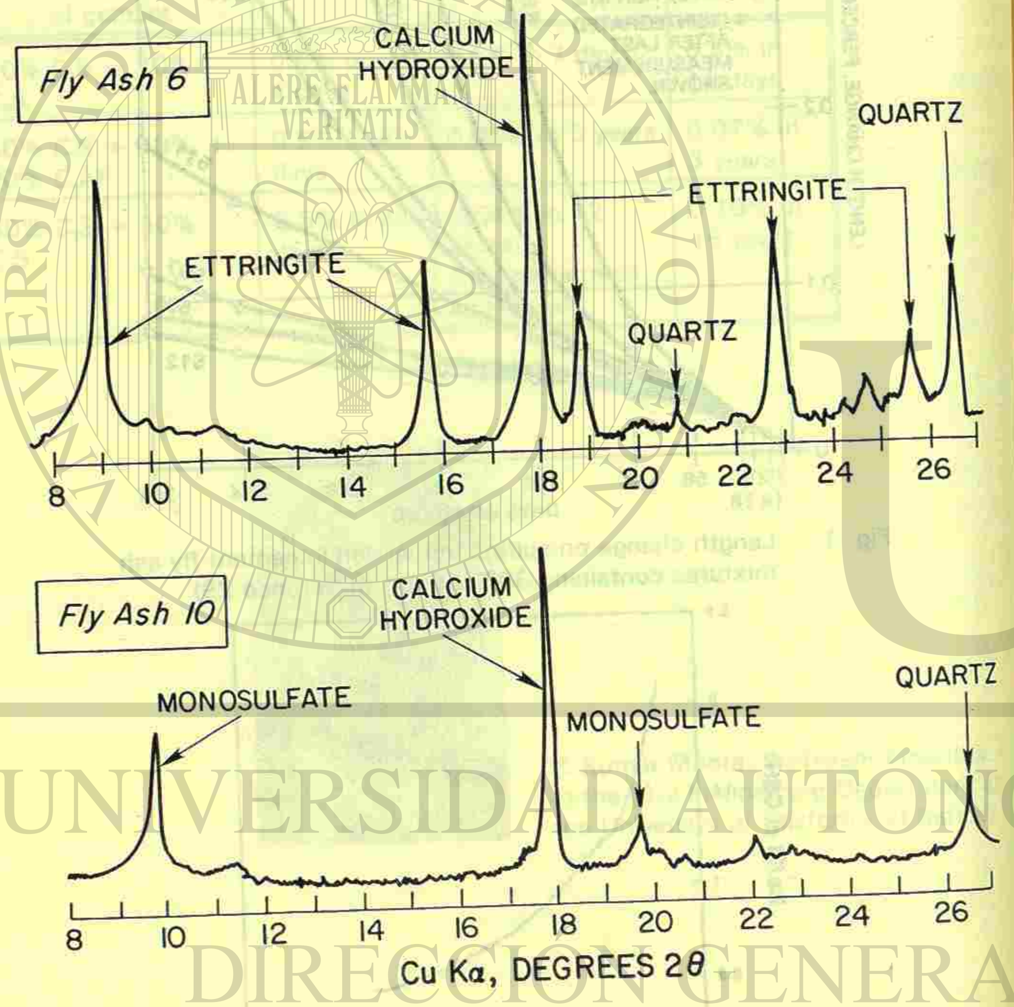

Fig. 3. X-ray diffractograms of hydrated portland cement - fly ash mixtures before the sulfate immersion (Reference 39)

RESISTENCIA AL CONGELAMIENTO Y DESCONGELAMIENTO DEL CONCRETO DE ALTA RESISTENCIA CON CENIZA VOLANTE CON ALTO CONTENIDO DE CALCIO Y MICROSILICA CONDENSADA

por

K.W. Nasser y Sujit Ghosh

Sinopsis: Se está desarrollando un estudio experimental sobre la resistencia a la compresión y al congelamiento de concretos de alta resistencia con varios niveles de sustitución de ceniza volante (0 a 80%) y un porcentaje prestablecido de microsílica (10%). Se añadió aire incluido a la mezcla para propósitos de comparación. Se estableció la relación agua/adherente óptima a un valor bajo de 0.27 por consideraciones de resistencia y trabajabilidad. Se usó superfluidificante para proporcionar una trabajabilidad adecuada y se mantuvo en un rango de 1.5 a 2.2%. Las relaciones agregado/cemento y agregado grueso/fino se mantuvieron en 5 y 1.22, respectivamente.

El programa de ensayes consistió en evaluar la resistencia a la compresión y dos series de especímenes fueron expuestos a pruebas de congelamiento y descongelamiento. Se encontró que un período de curado de 14 días, como lo especifica el Procedimiento A de ASTM C 666, era inadecuado para las mezclas con alto contenido de ceniza volante debido a la lenta ganancia de resistencia de ese tipo de mezclas. De modo que, todos los especímenes fueron curados en agua con cal por 28 días, antes de someterlos a ciclos de congelamiento y descongelamiento. La primera serie no tenía aire incluido, mientras que las segunda contenía ya sea 4 u 8% de aire incluido.

Los resultados de los ensayes han mostrado que el concreto con hasta un 60% de sustitución de ceniza volante con 10% de microsílica y sin aire incluido, indicaron igual o superior resistencia a la compresión a los 28 y 56 días, cuando se compararon con una mezcla de control con 100% de cemento. Las sustituciones de cemento de hasta 35% de ceniza volante y 10% de microsílica, indicaron una mejora en la resistencia al congelamiento, sin nada de aire incluido. La adición de 8% de aire incluido en la mezcla de 20% de ceniza volante + 10% de microsílica, aumentó el factor de durabilidad en cerca de 10%. También se proporcionan otros resultados incluyendo un estudio utilizando un microscopio electrónico de barrido (SEM).

Palabras clave: Resistencia al congelamiento, compresión, ceniza volante con alto contenido de calcio, superfluidificante, intrusión de aire, durabilidad, SEM.

Freezing and Thawing Resistance of High Strength Concrete containing High Calcium Fly Ash and Condensed Silica Fume

by

K.W. Nasser and Sujit Ghosh

Synopsis: An experimental study was undertaken on the compressive strengths and frost resistance of high strength concrete containing various levels of fly ash replacements(0 to 80%) and a fixed percentage of silica fume(10%). Air-entrainment was added to some of the mixtures for comparison purposes. Optimum water/binder ratio was fixed at a low value of 0.27 for strength and workability considerations. Superplasticizer was used to give proper workability and was maintained in the range of 1.5 to 2.2%. Aggregate/cement and coarse/fine aggregate ratios were maintained at 5 and 1.22, respectively.

The test program consisted of evaluating compressive strength in cylinders and two series of specimens were exposed to freezing and thawing tests. A curing period of 14 days, as specified by ASTM C 666 Procedure A, was found inadequate for the high fly ash mixtures because of the slow strength gain of such mixtures. Hence, all specimens were cured in lime water for 28 days, before subjecting them to freezing and thawing cycles. The first series had no entrained air whereas the second series contained either 4 or 8% entrained air.

The test results have shown that concrete with upto 60% fly ash replacement with 10% silica fume and without entrained air, indicated superior or similar 28 and 56-days compressivestrengths, when compared to the 100% cement-control mixture. Replacement of cement by upto 35% fly ash and 10% silica fume, indicated enhanced frost resistance, without any air-entrainment. The addition of 8% air-entrainment in the 20% fly ash + 10% silica fume mixture increased the durability factor by about 10%. Other results were found. A study of matrix morphology and microstructure bonding, using the scanning electron microscope (SEM), helped to explain the observed test results in a comprehensive manner.

Keywords: Frost resistance, compression, high calcium fly ash, superplastizicer, air entrainment, durability, SEM.

Karim W. Nasser Professor, Dept. of Civil Engineering, University of Saskatchewan, Saskatoon, Canada S7N 0W0.

Sujit Ghosh Ph.D Candidate, Civil Engineering, University of Saskatchewan, Saskatoon, Canada S7N 0W0.

INTRODUCTION

Chemical and / or mineral admixtures are used widely in concrete production in North America for enhanced performance[1]. The increase in use of two such admixtures in concrete, namely, fly ash and silica fume, has been encouraged over the past several years, in order to produce high performance concrete (HPC). In general, high performance is defined by not high strength alone but by high durability and economy as well. In Canada, durability against freezing and thawing is often of overriding importance.

Cracking and loosening up of the binding matrix caused by expansion of freezing water in the void system of the cement paste or the aggregates, is the major deterioration process of frost action on concrete. Detailed discussion, on some of the existing mechanisms are available in the papers published by Powers et al., Litvan and Meier et al.[2, 3, 4].

The use of fly ash in concrete in regulated amounts can enhance the frost resistance of concrete [5]. Fly ash alters the principal frost-damage parameters : namely, porosity and strength of paste, amounts of freezable water and number of voids, in a favorable way by producing a denser and stronger paste due to secondary pozzolanic reactions between the fly ash particles and the $Ca(OH)_2$ [6, 7].

The use of condensed silica fume along with large volumes of lignite fly ash (ASTM type C) in concrete, have been limited worldwide. The work reported herein is part of a research program being conducted on concrete containing 10% silica fume and several levels of fly ash replacements, in order to identify and explain the frost durability aspects of such concrete.

EXPERIMENTAL PROGRAM

Six concrete mixtures were made with CSA Type 10 normal portland cement (ASTM Type 1), Saskatchewan (lignite) fly ash and condensed silica fume. Fly ash content of the mixtures was either 20, 35, 50, 60, 70 or 80% of the weight of binder. Silica fume used was held constant at 10% of the weight of binder. In addition, two control mixtures, one made of 100% CSA type 10 cement and the other of 10% silica fume + 90% CSA type 10 cement, were also made and subjected to the same freezing and thawing tests. The aggregate to binder ratio and the coarse/fine aggregates ratio by weight, were maintained at 5 and 1.22 respectively. The weight of superplasticizer was varied between 1.5 and

2.2% of the weight of cement while the water to binder ratio was maintained at a low value of 0.27. No air entrainment was added initially, to any of the eight mixtures, so as to delete the effect of air-entrainment on strength. However, a second series consisting of 20 and 50% flyash + 10% silica fume with 4% and 8% entrained air, was tested under the same freezing and thawing conditions in order to understand the effect of air entrainment on frost resistance of flyash + silica fume concrete. For the air entrained mixtures, regular liquid organic air entraining agent MB-AE10 meeting ASTM C260 specifications, was used. Physical and chemical analysis of cement, fly ash and condensed silica fume are given in Table 1.

Mixing of concrete was carried out in a pan mixer in accordance with ASTM C 192. Initially, the pan was wetted and the mixer was started. Then the mixing sequence followed was that recommended by CANMET, Ottawa : coarse aggregate, part of water, cement and silica fume (if any), fly ash (if any), remaining part of water, fine aggregate, and finally superplasticizer (if any). The mixing was continued for three minutes, left to rest for two minutes and then remixed for another two minutes to prevent false setting. The air entraining agent was dissolved in the mixing water prior to mixing, whenever air entrainment was used.

The mixtures were tested according to ASTM C 666 Procedure A (rapid freezing and thawing in water). Before subjecting to freezing and thawing cycles, the 90x110x400 mm (3.5x4.5x16 in.) prism specimens were moist cured in lime water for 28-days since, 14 days of curing, as specified by ASTM C 666, was found inadequate for the high flyash mixtures because of the slow strength gain of such mixtures. Besides the freezing and thawing specimens, thirty 75x150 mm (3x6 in.) cylinders were cast in cardboard moulds, for every mixture, in accordance with ASTM C 31 and C 193, and they were moist cured. The cylinders were later tested for compressive strengths at 1, 3, 7, 14, 28 and 56 days. Also, from every batch, three 75x150 mm compressive strength test specimens were cast for the accelerated K-5 strength testing. Details of the physical properties of fresh and hardened concrete are presented in Table 2. The accelerated strength test cylinders were cast in three stainless steel moulds of the K-5 tester which contained plastic linings. Then they were mounted onto the K-5 tester and kept at a temperature of 160 deg.C (300 ± 5 deg.F) and a pressure of 10.34 ± 0.17 MPa (1500 ± 25 psi) for 3 hours and were tested after 2 hours of cooling.

Freezing and thawing Test

After moist curing in lime water for 28-days, four specimens of each mixture, were put inside long, vertical, rectangular, prismatic metal containers which were filled with water and placed inside the freezing and thawing tank, as shown in Fig. 1. Spacers of narrow rods were inserted between the four sides of each specimen and the corresponding faces of the container walls to ensure that the specimens were completely surrounded by approximately 3 mm (1/8 in.) of water on all sides. The specimens were placed on a flat piece of wire mesh at the bottom of the metal container to facilitate water and heat circulation next to the specimen.

The specimens were weighed and their natural frequencies were determined so as to evaluate their relative dynamic modulus of elasticity. The same process was repeated periodically (i.e. after every 23 and 32 cycles). The instrument for measuring the frequencies was a "Sonic Star" assembled in the Engineering Shops with a Type 602 Display Unit of Tektronix, Oregon, as outlined in ASTM C 666 and C 215. Every time the specimens were taken out, they were thoroughly washed in lukewarm water and were surface dried by wiping with a dry bath towel. The containers were also rinsed. The specimens were returned to the containers, turning them end-for-end (that is, top and bottom ends reversed), so that the same ends of the specimens do not always remain at either the bottom or at the top, during the entire freezing and thawing test. A predetermined rotation scheme was employed to shift the specimens to different locations inside the tank, so as to ensure that each specimen was subjected to conditions that prevailed in all parts of the freezing and thawing tank.

The freezing and thawing cycles were stopped after 300 cycles or until the relative dynamic modulus of elasticity reached 60 % of the initial modulus, whichever happened first. The durability factor was calculated as per ASTM C 666. The visual appearance of the specimens and any defect that developed were noted. Photographs of the specimens and their surfaces were taken before and after the freezing and thawing test. The surfaces were examined for texture, cracks, scaling off and exposure of the aggregates. A microstructure study with the help of a scanning electron microscope (SEM) was carried out on samples taken from the specimens, before and after the freezing and thawing tests.

Scanning Electron Microscopy

At the end of the freezing and thawing tests, small samples of about 1 cm^3 (0.06 in each, were taken from the control specimens and from the mid-section of the freezing and thawing specimens, for the scanning electron microscopy (SEM) tests. Each sample was coated with a very thin layer of gold as an electrical conductor and its microstructure was examined under a Philips SEM 505 scanning electron microscope. The results of the SEM study will be discussed later so as to explain changes in microstructure as a result of freezing and thawing. Microprobe Analysis was also performed on some of the specimens in order to identify the elemental composition of the crystals. JEOL : JXA - 8600 Superprobe was used for this purpose. The Energy Dispersion Spectrums (EDS) obtained are presented in the next section.

TEST RESULTS AND DISCUSSION

Workability and Compressive Strength

The optimum water to binder ratio was obtained from preliminary testings for maximum compressive strength and proper workability. A water/binder ratio of 0.27 was found suitable and was used for all subsequent testings. The water content of a fresh mixture (used for determination of water/binder ratio), was calculated by subtracting from the total

water added, the water absorbed by the coarse aggregates, and adding the water in the superplasticizer that was used in the mixture. The water content of the concrete mixtures containing high quantities of fly ash were slightly reduced so as to maintain close consistency and workability values. Table 2 indicates the properties of fresh and hardened concrete mixtures.

All mixtures with fly ash and silica fume generally showed lower strength than the control, at the age of 7 days (Table 2). However, with an increase in age, concrete with 20 to 60% fly ash along with 10% silica fume, gave 28 and 56-days strengths similar to that of the control specimens (HP C0-100% cement concrete), as shown in Fig. 2. With further increase in fly ash content beyond 60%, the strength level dropped, possibly due to poor matrix bonding because of the presence of too many unreacted fly ash particles, as evident from SEM micrographs to be discussed later. The 10% silica fume + 90% cement concrete, as well as the 10% silica fume + 20% fly ash + 70% cement concrete, had superior strengths compared to the control, at all ages. Table 2 also indicates that compressive strengths of the air-entrained 20% and the 50% fly ash concretes show lower values compared to their non air-entrained mixtures. Also, high air entrainment (8%) lowers the compressive strengths of the 20 and 50% fly ash mixtures, at all ages.

Resistance to Freezing and Thawing

Mather [8] observed that non-air-entrained concrete with a low water to binder ratio, will be durable against freezing and thawing if it has a compressive strength of about 24 MPa (3500 psi) when subjected to freezing and thawing; this strength reflects the desired strength of the paste which can withstand the pressure in the concrete due to volumetric expansion of the freezing water under frost action. Also, if the water to binder ratio is quite low, all mixing water can combine with the cementitious material and the concrete will develop low permeability; hence producing a condition in which it will be quite unlikely that the paste will critically saturate when freezing takes place. If the paste does not critically saturate on freezing, a satisfactory air-void system is not required for concrete to resist frost. In the present program the water to binder ratio was maintained at a low value of 0.27 and the majority of the concrete specimens were non-air entrained. The 28-days strength of the specimens of all mixtures was higher than 40 MPa (5714 psi) except for the 80% fly ash mixture which had a strength of 28 MPa (4000 psi), when they were subjected to rapid freezing and thawing cycles.

From visual inspection of the specimens after 300 freezing and thawing cycles, significant surface damage and cracks, exposed aggregates and substantial weight loss was observed, for the specimens with 50% or higher fly ash replacement levels. However, compared to the control, the 20% and the 35% fly ash replacement concretes along with 10% silica fume, performed better with regards to loss of dynamic modulus and surface damage. The 10% silica fume concrete showed a very good resistance to freezing and thawing and did not suffer any surface damage, except for minor cracks, after 300 freezing and thawing cycles, as evident in Fig. 3. The average weight loss of specimens of all series versus the mixture-type is shown in Fig. 4. The results for weight loss and durability of the air-entrained 20% and 50% fly ash concretes, have been plotted in Fig. 4 and 5, respectively, alongwith results of the non air-entrained specimens for better comparison. The average weight loss of the air-entrained mixtures was found to be 20 to 38% lower than the non air-entrained mixtures indicating lesser surface spalling under freezing and thawing cycles, as evident from Fig. 6.

Yuan and Cook [9] observed that durability of non-air-entrained concrete increases as the percentage of fly ash replacement increases up to 30%. In the present series of tests non-air-entrained concrete was used in the majority of the mixtures. The durability factor at 300 cycles (DF_{300}), was computed from the measured relative dynamic modulus and the results for all series were plotted against the fly ash replacement levels as shown in Fig. 5. Concrete with either 20 or 35% fly ash + 10% silica fume and also the 10% silica fume concrete exhibited better performance compared to the 100%-cement control concrete, after 300 freezing and thawing cycles. Durability of 50% fly ash concrete (with 10% silica fume) was found to be comparable to the durability of 100%-cement control concrete. Fly ash replacement beyond the 50% level reduced the frost resistance of concrete substantially and exhibited " poor " performance as per Neville's[10] acceptance recommendation. The 50% fly ash mixture did not show any appreciable increase in its durability factor, upon inclusion of 4 to 8% air-entrainment in the mixture. However, the 20% fly ash mixture with 8% air-entrainment showed a 10% increase in its frost durability factor.

Considering the frost durability factor of concrete (DF) with respect to the presence of silica fume, it is evident from Fig. 5 that the DF decreased linearly as the percentage of fly ash in the mixture was increased. Hence, the presence of fly ash in the mixture did not enhance the frost resistance of CSF concrete at all. However, compared to the frost durability factor of the control (100% type 10 cement) concrete (d_o), the presence of fly ash alongwith 10% CSF, did enhance the frost durability factor (DF) upto a fly ash content of 35% only, as shown in the same figure.

Discussion of microstructure and its effects on bonding and matrix morphology as evident from SEM micrographs, are presented next.

SEM Examination

COMPRESSIVE STRENGTH:

From the compressive strength test results presented in Table 2 and Fig. 2, it is evident that the low fly ash + silica fume mixtures exhibit higher strengths than the 100% cement control concrete. However, increasing fly ash contents upto 50% or more in the mixture brings about a loss of compressive strengths. SEM micrographs presented in Fig. 7, 8 and 9 may explain the observed results. Fig. 7 shows the matrix of a 100% cement control concrete. Well formed platy crystals of $Ca(OH)_2$, are evident in the microstructure. In contrast, the SEM micrograph of Fig. 8 shows a dull, dense matrix of 20% flyash + 10% silica fume concrete with no trace of $Ca(OH)_2$ platelets. The $Ca(OH)_2$ appears to have been consumed up in the secondary pozzolanic reactions, thereby providing a denser matrix of additional CSH gel deposition. This dense paste structure of flyash + silica fume concrete provides for higher compressive strengths. However, very high fly ash content in the mixture, say 50% or more, leaves numerous rounded unreacted fly ash particles in the matrix after 28 days of moist curing. This is evident in the SEM micrograph of 50% flyash + 10% CSF concrete presented in Fig. 9. The unreacted flyash particles do not bond well with the paste and a vey fine gap seems to exist in between the paste and the flyash particle. This presence of too many unreacted flyash particles with poor bonding to the paste, brings about a loss of compressive strengths of the high fly ash concrete mixtures. Fig. 10 shows the microstructure of 20% flyash + 10% silica fume concrete containing 8.3% entrained air, after 28 days of moist curing. Comparing with the microstructure of non air entrained 20% flyash + 10% silica fume concrete as presented in Fig. 8, it is clear that Fig. 10 reveals a honeycombed matrix with numerous air voids, which bring about a loss of compressive strength of the air-entrained mixtures.

SPECIMENS SUBJECTED TO FREEZING AND THAWING:

The microstructure of the paste which influenced the amount of freezable water in the matrix, played a definite role in affecting the frost resistance. The following principal parameters control the frost damage to concrete - porosity of the paste which determines the amount of freezable water in the matrix; paste microstructure and strength and the number of air-voids and cavities. Concrete mixtures with 10% silica fume + 90% cement and mixtures with 20 and 35% fly ash + 10% silica fume, exhibited a relatively dense microstructure, compared to the 100%-cement control concrete (Fig. 11, 12 and 13), due to additional deposition of CSH gel because of secondary pozzolanic reaction. From their dense cementitious matrix, it could be inferred that they contained very little freezable water and hence, were resistant to freezing and thawing cycles. A similar kind of dense matrix was observed in concrete with 50% or more fly ash. However, the presence of unreacted and unhydrated fly ash particles (as evident from the spherical particles and the rounded sockets in Fig. 14a), weakened the matrix bonding thereby adversely affecting the frost resistance. Nonetheless, 50% fly ash concrete exhibited comparable frost resistance to the 100%-cement control concrete . But, with 50% or more fly ash replacement, the presence of too many unreacted fly ash particles brought about early disruption of concrete when subjected to rapid freezing and thawing cycles. Based on the SEM micrographs of Fig. 11 through 16, the following distinct observations and frost deterioration mechanisms may be proposed :

<u>100%-cement control concrete</u> - With decrease in temperature, portlandite migrates through capillaries to cracks and pores and deposits platy $Ca(OH)_2$ over hydration products scattered throughout the matrix (Fig. 15), thereby loosening up the binding matrix.

<u>10% silica fume + 90% cement concrete</u> - Exhibits a dark dense matrix due to additional CSH gel formation as a result of secondary pozzolanic reaction. No platy $Ca(OH)_2$ crystals are evident (Fig. 12). The dense matrix made this concrete very resistant to frost cycles.

Fly ash + silica fume concrete - The densification of matrix due to additional CSH formation from secondary pozzolanic reactions and the absence of platy $Ca(OH)_2$, was also observed in this concrete (Fig. 13 and 14a). Hence, fly ash + silica fume concrete (upto 35% fly ash) exhibited good frost resistance. However, with increasing fly ash contents, the presence of unreacted fly ash particles (Fig. 14a), which did not bond well with the matrix (Fig. 9), weakened the microstructure bonding thereby adversely affecting the frost resistance of such concrete. The Energy Dispersion Spectrum (EDS) on the surface of an unreacted flyash particle is shown in Fig. 14b. Heavy presence of silicon, some aluminium and very little calcium marks the spectrum. The needle-like crystals, strewn across the matrix of 50% flyash + 10% CSF concrete as shown in Fig. 14a, were identified as ettringite crystals from the EDS shown in Fig. 14c. Calcium, sulphur and aluminium in the spectrum confirmed the presence of ettringite which is calcium sulphoaluminate.

Previous investigators [5] observed that under freezing and thawing cycles, air-entrained fly ash concrete deteriorated by migration of ettringite and $Ca(OH)_2$ from the paste structure to the air-voids in the concrete, thereby weakening up the matrix. In the present series, majority of the tests were non-air entrained concrete and the deposition of needle-like ettringite and $Ca(OH)_2$ in the air-voids were not evident since very little air-voids were present. However, needle-like ettringite crystals were found scattered throughout the matrix (Fig. 14a) and over the unreacted fly ash particles (Fig. 16) after the freezing and thawing tests. From the above observation, the following process is postulated : With depletion of $Ca(OH)_2$ due to secondary pozzolanic reactions, the solubility of ettringite increased. And, with the freezing of water due to frost cycles, volumetric expansion occured and the excess of water, now containing dissolved ettringite is expelled from the pores, through capillaries, into micro-cracks and fine gaps (Fig. 9) in between the unhydrated fly ash and the CSH matrix. The migration of ettringite in solution, weakens the binding matrix. The fine water gaps which were observed by other investigators too [6, 11, 12], failed to close even after 28-days of lime water curing (Fig. 9) and thus played a role in the frost deterioration mechanism.

For the air entrained mixtures, the presence of 8.2% entrained air in the 20% fly ash concrete, enhanced the frost durability factor by about 10%. Fig. 17 show the matrix of 20% flyash concrete before and after the freezing and thawing cycles. In contrast to Fig. 13 which exhibits dense non air-entrained matrix of 20% flyash concrete, Fig. 17 shows a honeycombed like structure caused by rounded sockets of entrained air in the 20% flyash air-entrained concrete. This brought about an average 12% loss of compressive strength of the air-entrained 20% flyash + 10% silica fume concrete. However, for the 50% fly ash concrete, air-entrainment of the order of 8.7% did not enhance its frost durability appreciably, as the presence of too many unreacted fly ash particles in the matrix with poor bonding to the paste, brought about early disruption of such concrete (Fig. 9 The presence of too many unreacted fly ash particles in high fly ash concrete could be either due to lack of $Ca(OH)_2$ as a result of fast pozzolanic reactions of condensed silica fume, or due to insufficient curing time of only 28 days. Hence, freezing and thawing resistance of high fly ash concrete warrants further research to fully understand its frost deterioration mechanism.

CONCLUSIONS

Based on the analysis of the test results and the microscopic analysis of concrete microstructure, the following conclusions are drawn :

1. Concrete with 10% CSF and high quantities of fly ash showed low early strengths. However, beyond 28 days, concrete with upto 60% fly ash and 10% CSF showed superior or at least equal compressive strengths to the type 10 cement-control concrete. The high compressive strengths of fly ash + CSF mixtures was attributed to denser paste structures compared to the 100% cement-control concrete. Air entrained fly ash + silica fume concrete mixtures showed lower compressive strengths compared to the non-air entrained mixtures.

2. Using 10% CSF only, as a cement replacement, improved the strength and durability of concrete exposed to freezing and thawing cycles.

3. Using up to 35% fly ash and 10% condensed silica fume (CSF) by weight, in the binder, provided satisfactory frost durable concrete. Durability of 50% fly ash concrete (with 10% silica fume) was found to be comparable to the durability of 100%-cement control concrete. Fly ash replacement beyond the 50% level reduced the frost resistance of concrete substantially. Using air-entrainment of 4 to 8 % in concrete mixtures, enhanced the frost resistance of 20% fly ash + 10% CSF concrete. However, for the 50% fly ash + 10% CSF concrete, air entrainment did not enhance its frost resistance significantly.

4. The presence of too many unreacted fly ash particles in high fly ash + silica fume concrete, adversely affected its frost resistance. The presence of unreacted fly ash particles could possibly be due to the depletion of available $Ca(OH)_2$ as a result of the fast secondary pozzolanic reactions of condensed silica fume or due to insufficient curing time of 28 days.

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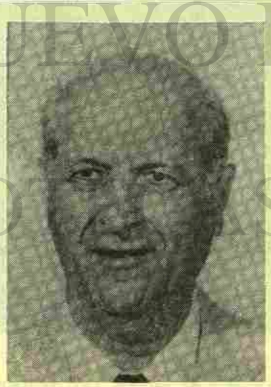
Table 1. Chemical Analysis and Physical Properties of CSA Type 10 (ASTM Type 1) cement, Saskatchewan Fly ash and Condensed Silica Fume :

Chemical Analysis	Cement		Fly Ash		Silica Fume	
		ASTM C 150		ASTM C 618		CSA.A 23.5
SiO_2	21.19	--	47.40	--	95.41	85.0 min
Al_2O_3	4.10	--	21.5	--	1.09	-
Fe_2O_3	2.34	--	5.3	--	-	-
CaO, total	62.84	--	14.40	--	0.32	
CaO, free	0.99	--				
$SiO_2+Al_2O_3+Fe_2O_3$			74.2	50.5 min	0.24	1.0 max
SO_3	2.56	3.0 max	0.58	5.0 max		
MgO	4.56	6.0 max	2.90	5.0 max	0.73	
Alkalies as Na_2O	0.66		1.77	1.5 max	-	
C_3S	56.6	--				
C_2S	18.1	--				
C_3A	6.90	--				
C_4AF	7.10	--				
Loss on ignition	1.40	3.0 max	0.65	6.0 max	1.80	6.0 max
Moisture content			0.09	3.0 max		
Insoluble residue	0.20	0.75 max				
PHYSICAL PROPERTIES						
Autoclave expansion	0.74	0.8 max	0.13	0.8 max		0.2 max
Time Vicat, minutes	75	45 - 375				
Compressive Strength						
at 3 days, MPa (psi)	24.5(3550)	12.4(1800)				
7 days, MPa (psi)	31.7(4600)	19.3(2800)				
28 days, MPa (psi)	37.0(5370)	27.6(4000)				
45-mm sieve retained, %	0.5	--	26.32	34 max		10 max
Blaine fineness, m²/Kg	401	--			20,000	
Drying shrinkage increase at 28-days, %			0.09	0.03 max		0.03 max
Specific gravity			2.21	--		
Pozzolanic Activity Index:						
with portland cement at					119.1	85 min
28 days, % of control			82.00	75.0 min		
with lime at 7 days (psi)			871	800 mix		
Water Requirement: % of control			94	105 max	139	



Karim W. Nasser,
Professor, Dep. of Civil Engineering, University of Saskatchewan, Saskatoon, Canada

Table 2. Proportions of Concrete Mixtures and Properties of Fresh and Hardened Concrete

Mix No.	Fly ash wt. %	Cementitious Materials Kg/m³			w/c	super-plasticizer Kg/m³	Air %	K-slump mm	Unit weight Kg/m³	Compressive Strengths (MPa) (ave. of 5-75 x 150 mm cylinders)		
		cement	flyash	CSF						5 hrs*	28days	56days
HP-C0	-	393.2	0.0	0.0	0.28	11.8	-	35	2494.3	32.1	44.3	54.4
HP-C1	-	365.1	0.0	40.5	0.28	12.0	-	38	2526.8	59.0	66.7	68.0
HF-20	20	278.3	79.6	39.8	0.27	7.78	-	35	2503.1	56.7	72.1	69.3
HF-35	35	210.0	134.4	38.6	0.27	7.55	-	35	2429.6	37.7	56.1	59.3
HF-50	50	156.4	193.4	39.3	0.27	7.67	-	38	2468.5	28.9	53.4	55.4
HF-60	60	117.0	233.9	39.4	0.27	7.69	-	25	2475.6	15.9	54.1	54.8
HF-70	70	76.9	273.5	39.3	0.26	6.51	-	35	2472.0	12.0	41.7	42.1
HF-80	80	39.4	312.6	38.9	0.25	6.95	-	20	2443.8	4.4	27.9	34.2
HA-20	20	273.2	78.1	39.0	0.27	9.96	4.3	35	2450.9	-	67.5	70.6
		264.2	75.5	37.7	0.27	9.63	8.2	40	2369.6	-	57.4	66.6
HA-50	50	154.7	193.2	38.7	0.27	9.87	4.4	20	2429.7	-	50.5	58.3
		145.0	181.1	36.3	0.27	9.25	8.7	40	2277.8	-	48.3	49.7

* For 5-hour accelerated curing, the strength was calculated as the average of 3-75x150 mm cylinders
Aggregate to cement ratio was maintained at 5.0.
Coarse aggregate to fine aggregate ratio was maintained at 1.22.
Maximum size of coarse aggregates was 3/4 in. (20 mm).

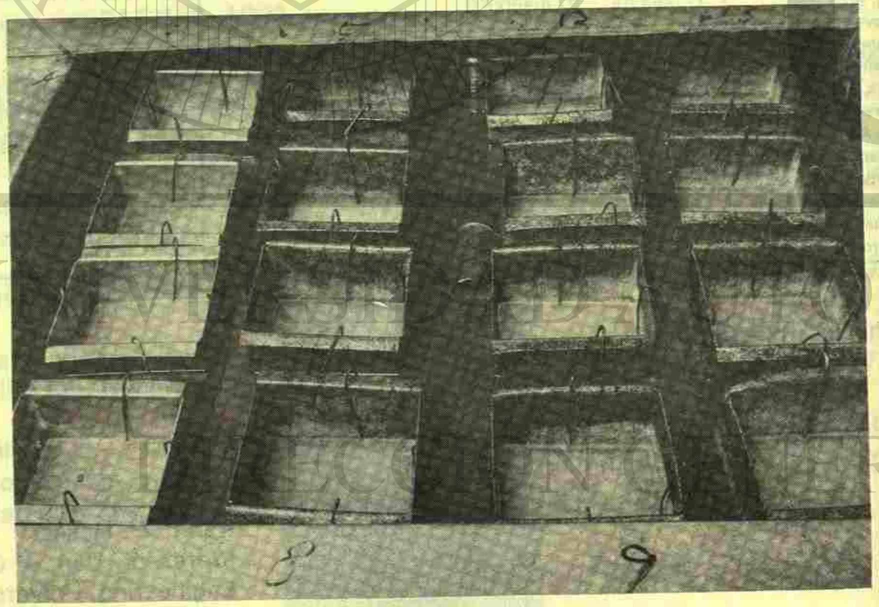


Fig. 1. Specimens in metal containers in the Freezing and thawing Tank.

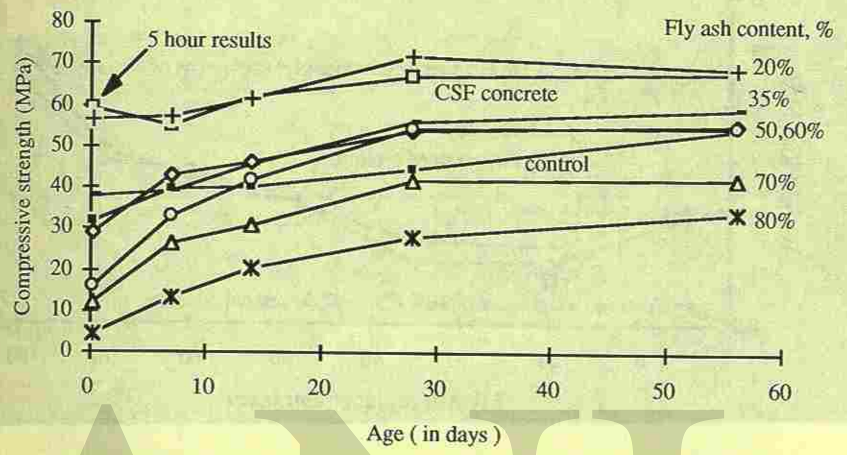


Fig. 2. Compressive strength of concrete versus Age.

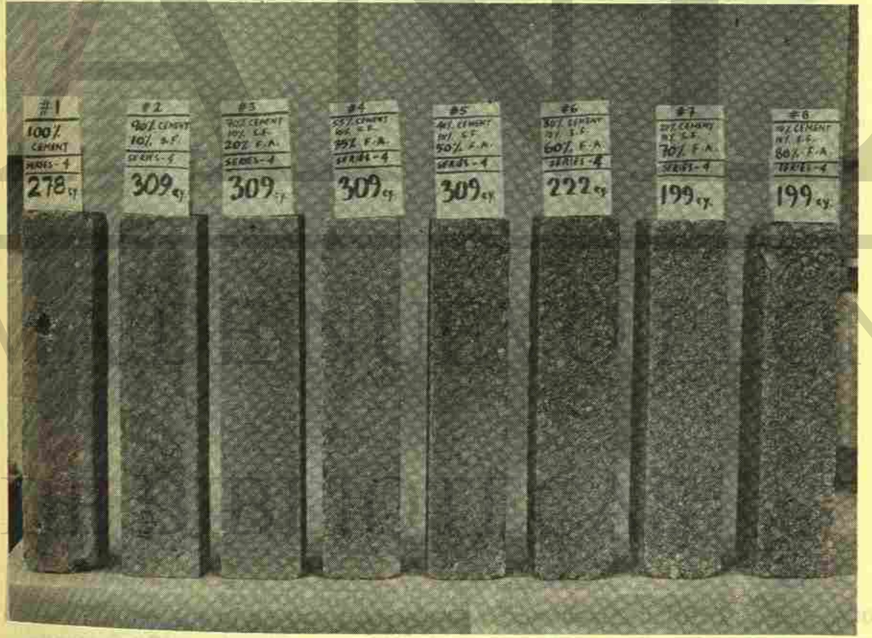


Fig. 3. Non air-entrained Concrete specimens after the Freezing and thawing test.

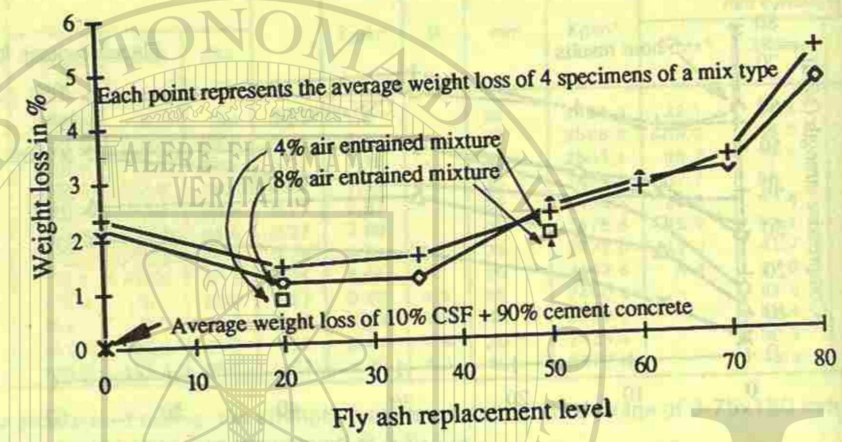


Fig. 4. Average weight loss of freezing and thawing specimens versus mixture typ

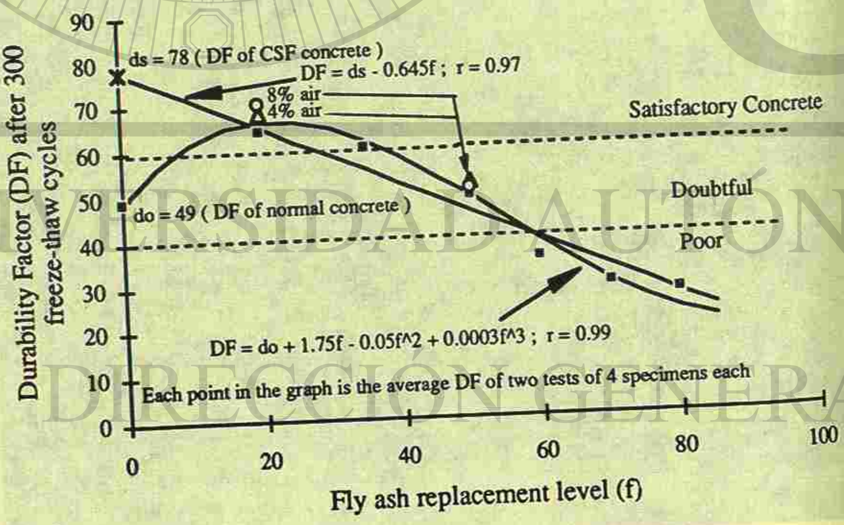


Fig. 5. Durability Factor (DF_{300}) vs. Fly ash replacement levels.

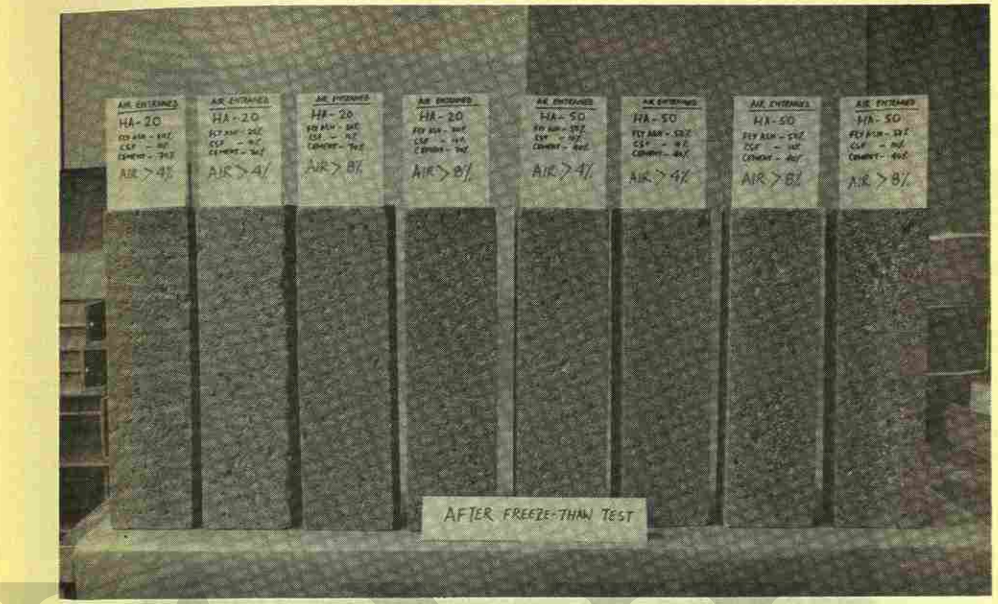


Fig. 6. Air-entrained Concrete specimens after the Freezing and thawing test.

Fig. 7. SEM micrograph of normal 100% type 10 cement-concrete [Magnification x 1850], shows $Ca(OH)_2$ platelets and CSH fibres bonding well with coarse aggregates after 28-days of curing.

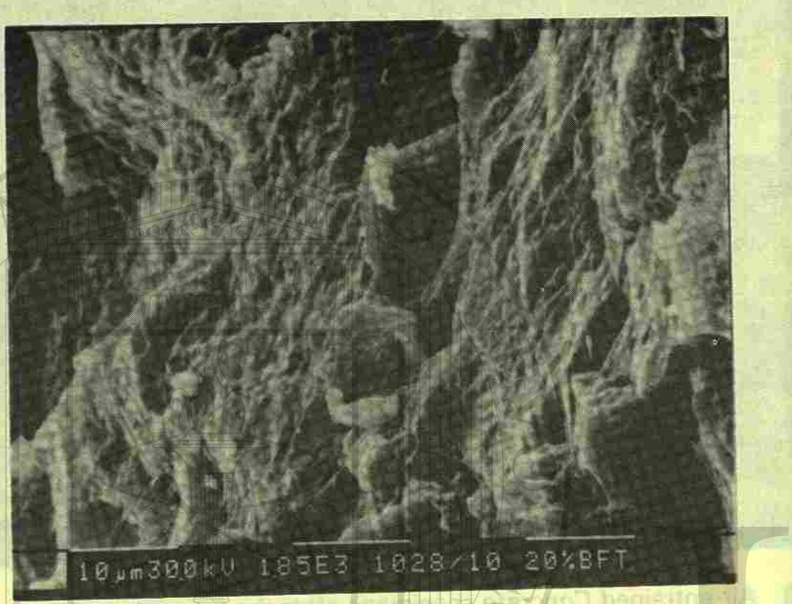


Fig. 8 SEM micrograph of 20%flyash+10%CSF concrete after 28days moist curin
Magnification x 1850.

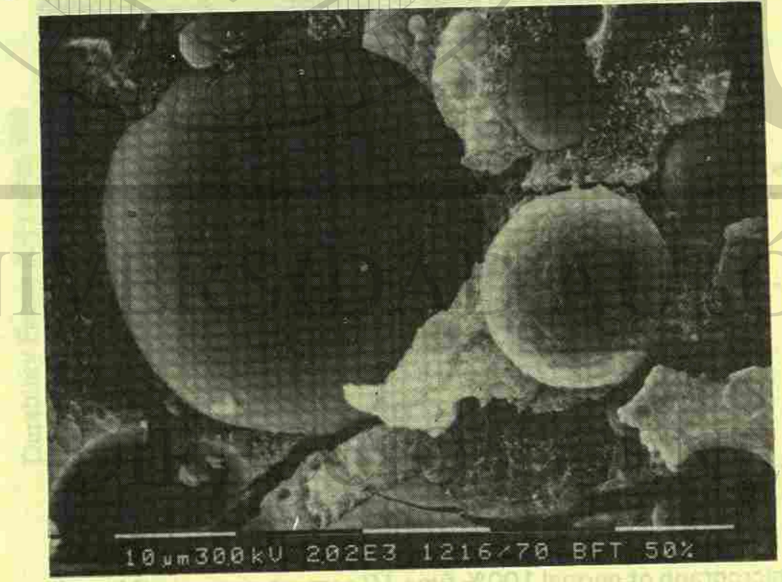


Fig. 9. SEM micrograph of high flyash concrete (50%flyash+10%CSF), reveals bondi of unreacted flyash with the paste separated by a fine gap [Magnification x 202

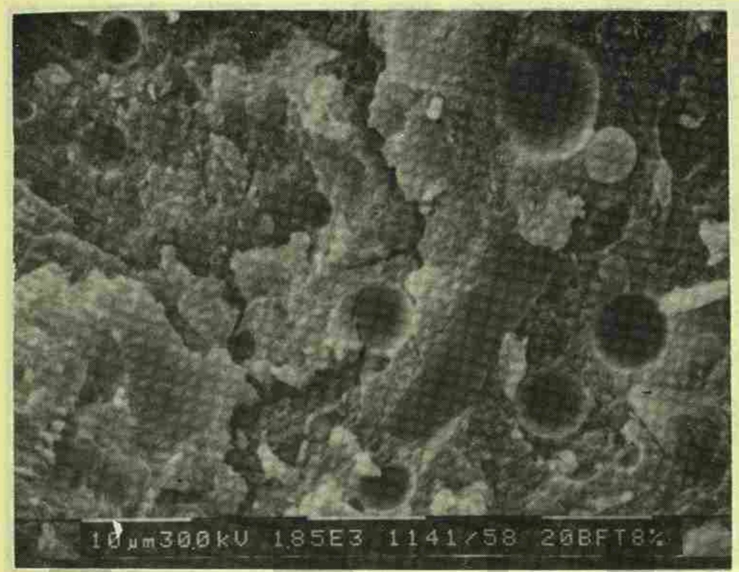


Fig. 10. SEM micrograph of 20%flyash+10%CSF concrete after 28days moist curing containing 8.2% entrained air; Magnification x 1850.

Fig. 11. SEM micrograph of normal concrete after freezing and thawing test [Magnification x 655]. Heavy deposition of whitish $Ca(OH)_2$ and radial growth of hydration products evident.

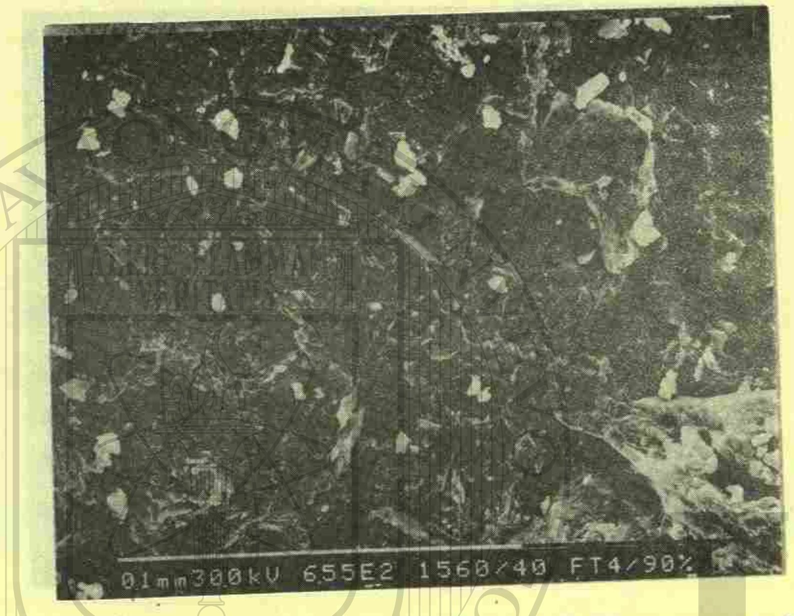


Fig. 12. SEM micrograph of silica fume concrete after freezing and thawing [Magnification x 655]. Dark, dense matrix with absence of radial growths.

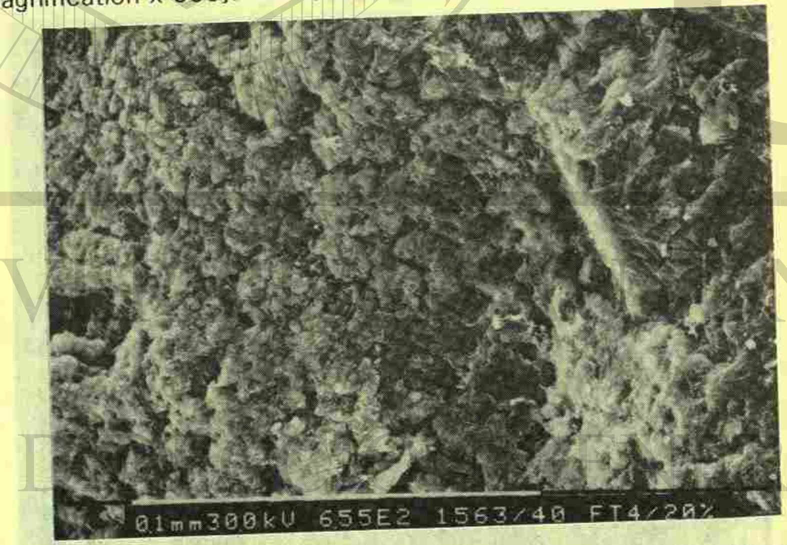


Fig. 13. SEM micrograph of 20% flyash + 10%CSF concrete after freezing and thawing [Magnification x 655]. Fairly dense CSH matrix, formed by consumption of $Ca(OH)_2$ in secondary pozzolanic reactions. Platy $Ca(OH)_2$ and rounded unrea flyash particles, absent.

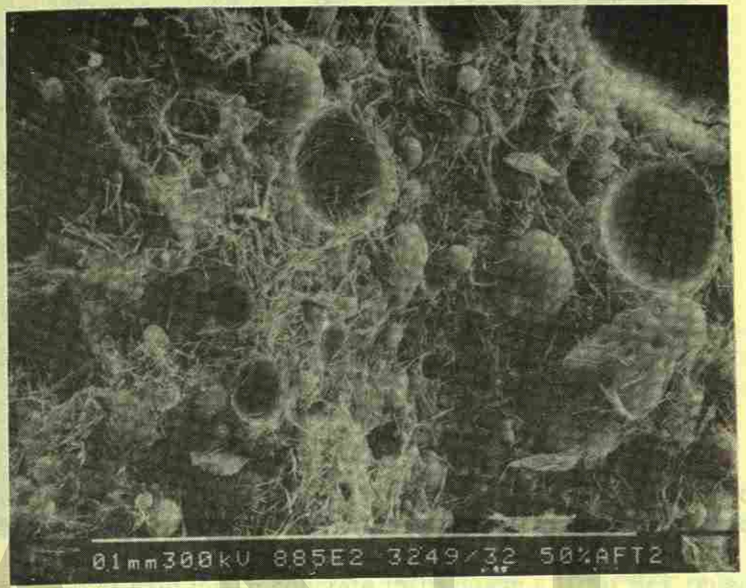


Fig. 14a SEM micrograph of high flyash (50%) + 10%CSF concrete after freezing and thawing test [Magnification x 885]. Needle-like crystals of ettringite scattered throughout the matrix - on hydration products and in the rounded sockets of unreacted flyash particles.

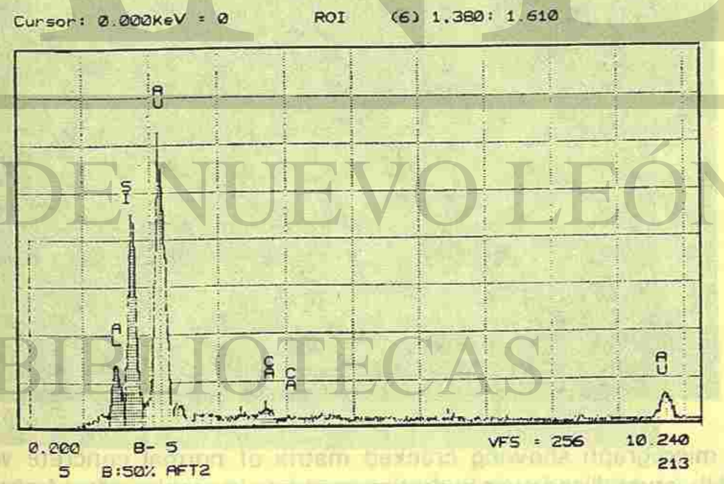


Fig. 14b EDS of the surface of a rounded unreacted fly ash particle of Fig. 14a (Al-aluminium, Ca-calcium, Si-silicon, Au-gold coating)

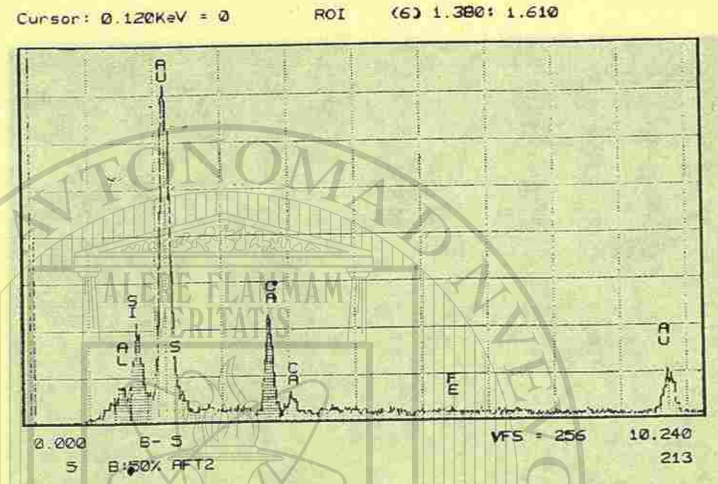


Fig. 14c EDS of the needle-like ettringite in matrix of Fig. 14a (Al-aluminium, Ca-calcium, S-sulphur, Si-silicon, Au-gold coating)

Fig. 15. SEM micrograph showing cracked matrix of normal concrete with platelets of $Ca(OH)_2$ crystallised over hydration products in cracks, after freezing and thawing cycles [Magnification x 1310]. Some needle-like ettringite crystals are also observed.

Fig. 16. SEM micrograph of high flyash concrete (50%flyash + 10%CSF), reveals a partly reacted flyash particle with its envelope broken and with needle-like crystals of ettringite scattered around, after 300 freezing and thawing cycles [Magnification x 2300].

Fig. 17. SEM micrograph of 20% flyash + 10% CSF air-entrained concrete reveals a matrix with numerous air-void sockets filled with needle-like crystals, after 300 freezing and thawing cycles [Magnification x 655].

ROLE OF ADMIXTURES IN CONCRETE DURABILITY

By

Raymundo Rivera Villarreal

Synopsis: In searching to increase concrete durability against the widely environmenta adverse effects, under which it is exposed, it has been developed cementitio supplements, fibers and admixtures to add them in concrete. This paper is focused on in the use of admixtures. Although they have been developed to improve many concre properties, only those admixtures which main function is directly related with concre durability will be commented, and its side effects, which in some cases will be harmles will be discussed. The admixtures commented, and which general information is presente are anti freezing, water reducers and high range water reducers admixtures, setti retarders, air entraining, inhibiting steel corrosion, and those to reduce alkali-aggrega reaction.

Keywords: Admixtures, durability, anti freezing, water reducer, high range water reduce plastic shrinkage, dry shrinkage, freezing, thawing, air entraining, corrosion inhibito

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FUNCION DE LOS ADITIVOS EN LA DURABILIDAD DEL CONCRETO

Por

Raymundo Rivera Villarreal

Sinopsis: En la búsqueda por aumentar la durabilidad del concreto contra los diversos efectos adversos del medio ambiente a que esta sujeto, se ha desarrollado el uso de suplementos cementantes, fibras y de aditivos. Este trabajo se enfoca solamente al uso de aditivos, que aunque estos se han desarrollado para mejorar diversas propiedades del concreto, se comentarán únicamente aquellos aditivos cuya función principal está relacionada directamente con la durabilidad del concreto. Se comentan los efectos secundarios que en algunos de los casos pueden ser adversos. Los aditivos que se discuten y se presenta información son los anticongelantes, los reductores y super reductores de agua, los retardantes del fraguado, los inclusores de aire, los anticorrosivos y los que reducen la reacción álcali-agregado.

Palabras Clave: Aditivos, durabilidad, anticongelante, reductores de agua, super reductores de agua, retracción plástica, retracción por secado, congelamiento, descongelamiento, inclusores de aire, anticorrosivos, reacción álcali-agregado.

Raymundo Rivera Villarreal es Ingeniero Civil, M. en C. y Dr. Hc de la Universidad Autónoma de Nuevo León, Maestro decano de la Facultad de Ingeniería Civil y Jefe de Investigación en Tecnología del Concreto. Investigador Nacional Nivel 2, Miembro Fellow del ACI y de los Comités 110 y E 701, Ex-presidente del Comité RILEM 84 AAC. Miembro Honorario del Colegio de Ingenieros Civiles de Nuevo León y de la Sociedad de Ingenieros y Técnicos de Monterrey. Receptor de entre otras preseas la "Manuel Martínez Carranza" de la Federación de Colegios de Ingenieros Civiles de la República Mexicana y "El Registro" del IMCYC.

INTRODUCCION

El concreto es uno de los materiales más versátiles utilizados en la construcción. Se ha estado usando modernamente durante 200 años y continuará usándose porque no hay una alternativa más barata y eficiente. Sus propiedades pueden variar ampliamente en función del tipo de cemento y proporción utilizada en la pasta, el tipo, calidad y granulometría del agregado, además las propiedades pueden variar de acuerdo a la forma de fabricación, de colado y de curado. Sin embargo, para mejorar algunas propiedades específicas y lograr concretos con características de alto comportamiento aún en condiciones ambientales adversas, se han desarrollado ingredientes para el concreto clasificados como: cementantes suplementarios, fibras y aditivos.

Los cementantes suplementarios incluyen escorias y cenizas naturales y artificiales que son activados por los compuestos resultantes de la hidratación del cemento Portland impartiendo al concreto diversas características, algunas de ellas relacionadas directamente con el aumento de la durabilidad, minimizando efectos adversos. Algunos cementantes suplementarios se discuten ampliamente en otros trabajos incluidos en esta Memoria del Seminario sobre Durabilidad del Concreto, del cual forma parte este tema.

Las fibras son materiales distribuidos al azar en la masa del concreto para modificar sus propiedades mecánicas y elásticas, pueden contribuir grandemente a aumentar la durabilidad del concreto en la medida que limiten los agrietamientos del concreto. En la misma Memoria se presenta un tema sobre Concreto Reforzado con Fibras.

En muchos países los aditivos para concreto se han estado utilizando cada vez más ampliamente para diversas aplicaciones y su tecnología es progresivamente mejor entendida y utilizada. El concreto de alta resistencia no sería posible producirlo rutinariamente sin el uso apropiado de aditivos. Constantemente nuevos aditivos y combinaciones de estos han estado surgiendo para mejorar alguna propiedad en particular del concreto. Una evidencia de esto es el gran número de patentes registradas en los últimos años. Ramachandran y co-autor (1) reportan que entre los años de 1985 y 1989 aparecieron aproximadamente 600 patentes y el enorme interés en su desarrollo lo confirma el hecho de que solamente en el año de 1989 fueron registradas 242.

Este trabajo se enfocará hacia el empleo de los aditivos cuyos efectos estén orientados a minimizar las causas que propicien baja durabilidad al concreto. Asi mismo se discutirán las precauciones necesarias si al utilizar un aditivo con fines distintos al de mejorar su durabilidad, sus efectos colaterales puedan llegar a originar baja durabilidad.

Dos de los grupos de aditivos que tienen que ver con la durabilidad se discutieron en forma más amplia en el Seminario y estos fueron: los Superfluidificantes y los Inhibidores de la Corrosión del Acero de Refuerzo.

ADITIVOS

Los aditivos para concreto se definieron inicialmente como aquellos materiales distintos del agua, del cemento y de los agregados. Una definición más específica es la establecida por el Comité 84 AAC de RILEM (2), como sigue:

"Los aditivos para concreto, mortero o pasta son materiales orgánicos o inorgánicos, incluyendo minerales, en estado sólido o líquido, que se añaden a los componentes normales de la revoltura, en la mayoría de los casos hasta un máximo de 5 % de la masa del cemento o materiales cementantes.

"Los aditivos actúan en el sistema hidratante-cementante por acción física, química o física-química, modificando una o más propiedades del concreto, mortero o pasta en la etapa de fresco, fraguando, endureciéndose y endurecido.

"Los materiales tales como ceniza volante, escorias, puzolanas o microsílica, los cuales pueden ser componentes del cemento y/o concreto, incluyendo productos que actúen como refuerzo, no son clasificados como aditivos."

Existen muchos libros, memorias y boletines que pueden servir como fuentes de referencia para el conocimiento de los aditivos y sus aplicaciones (2, 3, 4, 5, 6, 7).

Los aditivos pueden afectar alguna o algunas de las propiedades del concreto que se presentan en cada una del las etapas de su vida, las cuales son, de acuerdo a la Fig. 1: fresco, fraguando, endureciéndose y endurecido.

El concreto puede estar trabajando solo o con refuerzo metálico, por adherencia o presforzado.

Por lo que respecta a la durabilidad del concreto reforzado, ésta puede referirse al concreto mismo o a efectos destructivos relacionados con la corrosión del acero de refuerzo.

Aditivos en la Durabilidad

La durabilidad del concreto es la propiedad más importante deseable en su etapa de endurecido. Sin embargo, en ésta influyen el proporcionamiento y los cuidados que se tengan con él en sus etapas de fresco, fraguando y endureciéndose.

Las causas para una baja durabilidad del concreto pueden ser físicas, químicas o mecánicas, originadas por agentes externos provenientes del medio ambiente que pueden ingresar a la masa de concreto o de agentes internos existentes dentro del propio concreto que se introdujeron durante su fabricación, o por cargas abrasivas.

Entre las causas físicas tenemos la acción por congelación del agua dentro de la masa del concreto y por problemas asociados con los cambios volumétricos térmicos y los gradientes térmicos diferenciales.

Las causas químicas son aquellas asociadas al cambio volumétrico que ocurre al producirse la reacción química en la masa del concreto, formando compuestos que se endurecen, entre estos tenemos los formados por el ataque de los sulfatos solubles y por la reacción alkali-agregado, o por la acción química de agentes externos que atacan al concreto formando compuestos suaves y solubles, como son el ataque por ácidos. La acción del CO_2, carbonatando el concreto y la presencia o el ingreso de iones cloro a la masa del concreto inducen a la corrosión electro-química del acero, originada por el ingreso de humedad y oxígeno, acelerada por el incremento de la temperatura, y por ósmosis. En la corrosión, aparte de disminuirse la sección transversal efectiva del acero, el cambio volumétrico originado por la reacción química expansiva, puede llegar a botar el concreto de recubrimiento.

Las causas mecánicas son aquellas que originan un desgaste por abrasión, como puede ser el rodamiento de llantas metálicas o de hule o, en estructuras hidráulicas la acción de sólidos arrastrados por el agua o por la cavitación.

El trabajo del concreto se inicia a partir del momento en que alcanza la resistencia de diseño. Sus características de durabilidad, en lo que se refiere a aquellas que tienen que ver con sus ingredientes, quedarán establecidas al darse el proporcionamiento, entre estos quedan comprendidos los aditivos.

En el uso de los aditivos y su dosificación, para producir características deseables en el concreto endurecido, deberán preverse los posibles efectos en las diversas etapas de la vida del concreto.

Los aditivos, cualquiera que sea su fin, se agregan durante la revoltura del concreto generalmente disueltos en el agua o inmediatamente después de mezclados todos los ingredientes; en esta primera etapa de la vida del concreto, su influencia se refleja en la trabajabilidad, su dosificación puede llegar a producir efectos secundarios adversos a la durabilidad.

ADITIVOS ANTICONGELANTES

El uso de aditivos anticongelantes en el concreto colado en clima frío y muy frío, es una alternativa más económica que la práctica de termo, que consiste en mantener calientes los ingredientes del concreto y la revoltura misma durante el período de protección. Esta práctica consume una cantidad importante de energía y requiere de una supervisión eficiente.

Los aditivos anticongelantes pueden utilizarse en combinación con la práctica de termo para lograr combinaciones económicas en el consumo de aditivos y de energía.

En Rusia, con una experiencia de más de 40 años, el avance tecnológico en el uso de aditivos anticongelantes es importante y se ha llegado a usar aún en regiones con clima frío severo hasta -30° C. Ratinov y Rozemberg presentan un estudio muy amplio en la Referencia 8.

Clasificación

Los aditivos anticongelantes trabajan en dos grupos:

1. Aditivos que disminuyen el punto de congelación de los líquidos.
2. Aditivos que aceleran considerablemente el proceso de fraguado y endurecimiento del cemento.

Los aditivos del grupo 1 son además débiles acelerantes o retardantes de los tiempos de fraguado y endurecimiento del cemento, como ejemplo tenemos el nitrito de sodio comúnmente usado en la antigua URSS (9), el cloruro de sodio (10), urea y otros.

Los aditivos del grupo 2, al acelerar el proceso de fraguado y endurecimiento, generan calor que ayuda además al anticongelamiento. Como ejemplo tenemos la potasa, el cloruro de calcio, el nitrito-nitrato de calcio y mezclas de los anteriores.

Dosificación

En los aditivos anticongelantes, debido a que actúan disminuyendo el punto de congelación del agua, su dosificación se da en función de la masa del agua de mezcla y es conveniente prepararlos en forma de solución acuosa.

La dosificación recomendada para cada tipo de aditivo se da en función de la temperatura de endurecimiento de diseño y en porciento de la masa del agua de mezcla, estos porcentajes son promedio, ya que además influyen el procedimiento de construcción, el tipo de cemento, el tipo y cantidad de suplementos cementantes, la temperatura y cantidad de los ingredientes del cemento y otros factores. La dosificación se da también para el caso en el que la temperatura del concreto sea menor que aquella que pueda desarrollar una resistencia tal, que un congelamiento no cause detrimento de sus propiedades.

El colado del concreto en clima frío utilizando aditivos anticongelantes no excluye el uso de otros aditivos como son los inclusores de aire, los reductores y los altos reductores de agua o superfluidificantes. La dosificación deberá determinarse experimentalmente. Mayores cantidades de estos aditivos son requeridas cuando se combinan con otros aditivos.

Durabilidad

La efectividad de los aditivos anticongelantes en la durabilidad, cuando se usan solamente estos en la revoltura del concreto, está en función de la medida en que puedan evitar el congelamiento del concreto fresco, aunque su aplicación dé como resultado un congelamiento parcial de las partículas de agua. Cuando se usan en combinación con otros aditivos, su efecto dependerá del tipo de aditivo y las propiedades físicas, químicas y mineralógicas del cemento hidráulico.

La inclusión de aditivos anticongelantes no afecta la porosidad total del concreto; sin embargo, hay un aumento de los microporos a expensas de los macroporos. Esto mejora la zona de contacto entre la pasta de cemento y los agregados cuando se utilizan anticongelantes a base de calcio y, por lo tanto, aumentan la impermeabilidad y aún más en combinación con superfluidificantes.

La durabilidad se ve incrementada en la medida en que el aumento en impermeabilidad disminuye el ingreso a la masa del concreto de agentes químicos destructivos y del CO_2.

Con respecto a la corrosión del acero de refuerzo, los aditivos anticongelantes a base de nitrito de sodio y mezclas de nitrito-nitrato de calcio inhiben la corrosión del acero de refuerzo cuando está cubierto por una capa suficientemente gruesa de concreto. Deberán tomarse en cuenta ciertas precauciones especiales contra la corrosión del acero de refuerzo con el uso del resto de los productos anticongelantes.

El concreto endurecido a temperatura $\geq 0°$ conteniendo anticongelantes como $CaCl_2$ y NaCl, presentan de un 5 al 10 % más de retracción comparado con los concretos de referencia sin aditivos (8). En proporcionamientos con otros aditivos conteniendo además anticongelantes, el efecto de retracción es aproximadamente el mismo que cuando se usan separadamente.

REDUCTORES Y SUPER REDUCTORES DE AGUA

Los aditivos reductores de agua efectivos, deben disminuir los requerimientos de agua de la revoltura de un 5-12% para una consistencia y consumo de cemento dados, dependiendo del proporcionamiento, tipo de aditivo y dosificación.

De acuerdo a los ingredientes básicos existen tres clases de aditivos reductores de agua:

1. Sales de ácido lignosulfónico.
2. Sales de ácidos carboxílicos hidroxilados.
3. Polímeros hidroxilados.
4. Otros

Un efecto secundario que se presenta en la mayoría de los reductores de agua convencionales es la tendencia a retardar los tiempos de fraguado del concreto. El efecto retardante de estos aditivos puede, en algunas formulaciones, incluir productos acelerantes, pero deberán tomarse precauciones con la adición de cloruros para que no se excedan los límites especificados por ACI-318. El grado del efecto depende de las proporciones de cada ingrediente en las formulaciones.

El efecto que sobre la durabilidad tiene la acción retardante, se discutirá al comentar los aditivos retardantes.

Otro efecto secundario es la inclusión de aire en la masa de concreto, que origina una disminución en la resistencia a la compresión.

Los super reductores de agua se comportan como los reductores de agua convencionales pero con mucha mayor eficiencia, estos provocan una acción antifloculante reduciendo las fuerzas interparticulares que existen entre los granos del cemento en la pasta fresca (18), incrementando, por consiguiente, la fluidez de la pasta, pueden utilizarse para obtener reducciones de agua entre 20-30%. Además estos difieren de los reductores de agua convencionales en que no afectan significativamente la disminución de la tensión superficial del agua y por lo tanto pueden utilizarse en altas dosificaciones sin una excesiva inclusión de aire. No producen retardo excesivo aún con altas dosificaciones.

Los aditivos super reductores de agua normalmente se clasifican en dos familias:

1. Formaldehído condensado sulfonatado de melamina.
2. Formaldehído condensado sulfonatado de naftalina.

Al reducirse la relación A/C para una consistencia dada, el efecto en el concreto endurecido es un aumento en la resistencia a la compresión y una reducción en la permeabilidad. Debido al mejoramiento en la eficiencia de la hidratación del cemento, en la mayoría de estos aditivos, la ganancia en resistencia a la compresión es mayor que la indicada solamente por la reducción en la relación A/C.

La influencia que sobre la durabilidad tienen los aditivos reductores y super reductores de agua cuando se les utiliza para una consistencia dada, en comparación con los de referencia, está relacionada directamente con el aumento de la resistencia a la tensión y, por lo tanto, a la resistencia a todas las causas de agrietamiento, como pueden ser aquellas originadas por las reacciones químicas expansivas, por el efecto expansivo debido a la corrosión del acero de refuerzo, por la congelación del agua en el concreto saturado, por los cambios volumétricos debidos a cambios de temperatura o por la retracción por secado.

Retracción Plástica

La retracción plástica es la disminución en el volumen del concreto superficial, originada por una pérdida de humedad por evaporación rápida, antes de que pueda ser reemplazada por agua de sangrado proveniente de las capas inferiores. La retracción plástica ocurre durante las primeras horas después de colado el concreto.

La pérdida de agua del concreto fresco da como resultado una retracción plástica, la cual puede causar agrietamiento si los esfuerzos de tensión inducidos en la capa superficial, exceden el esfuerzo de tensión del concreto plástico en cualquier momento y por el hecho de estar empotrado en el concreto interior que no se retrae.

En la segunda etapa de la vida del concreto, es decir, cuando está fraguando, pueden ocurrir agrietamientos al final de ésta por retracción plástica o por secado, que culminan en una baja durabilidad.

La retracción plástica es mayor entre mayor sea la velocidad de evaporación del agua, ésta depende de la temperatura del aire y del concreto, la humedad relativa y la velocidad del viento, de acuerdo con el Comité ACI 305-R-77 (revisada en 1982). Las velocidades de evaporación mayores de 1.0 Kg/hr/m² de la superficie expuesta del concreto, deben evitarse para prevenir agrietamientos por retracción plástica. El Comité ACI 305 hace diversas recomendaciones para disminuir la velocidad de evaporación actuando sobre los cuatro parámetros mencionados.

En los estudios realizados por R.L. Hermite (11), en proporcionamientos variando solamente el consumo de cemento, a 20 °C, 50 % de humedad relativa y velocidad del viento de 1.0 m/s, encontró que la retracción plástica disminuía al aumentar el contenido de agregado y aumentaba al elevar el consumo de cemento. Por lo tanto, aquellos aditivos que reduzcan la cantidad de agua para la misma consistencia y misma relación A/C en el concreto, se verá reducido además, el consumo de cemento y, por lo tanto, la retracción plástica y la posibilidad de agrietamiento. De aquí la ventaja es usar aditivos reductores de agua y sobre todo los super reductores de agua. Sin embargo, en el uso de estos aditivos deberá verificarse el efecto secundario relacionado con el retardo en el tiempo de fraguado, el cual puede llegar a aumentar la retracción plástica como se verá más adelante.

Contenido de Agua

Ravina y Shalon (12, 13) determinaron el efecto del contenido de agua en la retracción plástica en condiciones ambientales de clima caliente y seco. En la Fig. 2 se presentan los valores de retracción plástica para un mismo concreto, variando su contenido de agua para

consistencia semi-plástica, plástica y fluida con el mismo consumo de cemento de 550 Kg/m³, para una temperatura de 20° C y una velocidad del viento de 20 Km/hr. Las revolturas con mayor contenido de agua presentan mayor retracción plástica que las secas, presentándose el valor máximo a la edad de 5 a 6 hrs, con un retracción plástica para el concreto semi plástico de 3.4 mm/m y para el plástico de 7.7 mm/m, lo cual significa un 126% de aumento. En los morteros de consistencia semi plástica, no ocurrió agrietamiento plástico, mientras que en los plásticos y los fluidos se agrietaron severamente.

En la Fig. 3 se presentan los resultados del mismo concreto cuyos resultados aparecen en la Fig. 2 con consistencias semiplásticas y plásticas, pero para una temperatura de 30° C y la misma velocidad del viento de 20 Km/hr. Se observó de nuevo un gran aumento en la retracción plástica al pasar el concreto de consistencia semi-plástica a plástica con retracciones de 5.0 mm/m a 10.2 mm/m, lo cual representa un incremento de más del 100% pero alcanzando la máxima retracción plástica en aproximadamente 3 hrs. Un bajo contenido de agua en el concreto simple, implica una consistencia más rígida y, por lo tanto, menos susceptible a la retracción plástica.

De aquí la importancia del uso de los super reductores de agua para disminuir la cantidad de agua y minimizar el agrietamiento por retracción plástica.

Retracción por Secado

La retracción por secado es la disminución en el volumen del concreto cuando pierde humedad por evaporación. La deformación por retracción por secado es asintótica con el tiempo.

La retracción por secado origina cambios volumétricos que pueden llegar a producir agrietamiento. La extracción del agua del concreto endurecido en ambiente no saturado causa retracción por secado. Si llega a ocurrir un rehumedecimiento, éste origina expansión, pero una parte de la retracción es irreversible (14).

Aunque la pasta de cemento es la que mayor influencia tiene en la retracción por secado del concreto, las características elásticas del agregado y su porciento de participación en el volumen del concreto, tienen influencia en la retracción total.

La calidad de la pasta de cemento como principal causante de la retracción por secado, influye en la magnitud de ésta; entre mayor sea la relación A/C, mayor será la retracción, como se puede apreciar en la Fig. No. 4, que presenta el resultado de los estudios de S.T. Odman (15) sobre la influencia de la relación A/C y contenido de agregado en la retracción.

La magnitud de la retracción por secado que puede llegar a originar agrietamiento está en función de diversos factores como son: las características del agregado, en la que interviene su tamaño máximo, su módulo de elasticidad y textura, la velocidad de endurecimiento del tipo de cemento utilizado, el porcentaje de acero de refuerzo, las dimensione del elemento de concreto, las condiciones ambientales de temperatura y de humeda relativa, la relación A/C y el grado de carbonatación.

La carbonatación del concreto que se origina en la superficie expuesta progresa hac el interior lentamente. La velocidad de carbonatación depende de la permeabilidad d concreto, el contenido de CO_2 y la humedad relativa del medio. La carbonatación origi retracción, la cual se acumula con la originada por el secado. En la Fig. 5, como resultad de los estudios realizados por G. Verbeck (16), se puede apreciar la retracción total p efectos de secado y carbonatación.

Una de las causas para aumentar la velocidad de la carbonatación es la permeabilida y ésta será menor, cuanto menor sea la relación A/C.

El efecto benéfico de los aditivos reductores y super reductores de agua en l retracción, a igualdad de los parámetros antes mencionados, está en función de la medid en que estos puedan disminuir la relación A/C.

Durabilidad

Una reducción en la permeabilidad, que propician los aditivos super reductores de agu al disminuir el volumen de poros capilares y su tamaño promedio, además de producir u red más discontinua, minimiza el ingreso de agua, de productos agresivos solubles e agua, de gases agresivos como el CO_2 y el ingreso de iones cloro. Al disminuir el ingres de productos agresivos solubles en agua, como sulfatos, sales, ácidos en solución menores serán sus efectos en reacciones químicas destructivas, de sales que facilitan e fenómeno corrosivo del acero de refuerzo, del agua congelable en la masa del concret o del efecto carbonatador del CO_2 que también facilita la corrosión del acero.

Los aditivos super reductores de agua afectan adversamente el sistema de huecos d aire, cuando se utilizan conjuntamente con un aditivo inclusor de aire, produciendo u factor de esparcimiento mayor que el requerido (200 μm), para la protección de ciclos d congelamiento y descongelamiento. Sin embargo, cuando estos concretos se someten a las pruebas de la norma ASTM C-666, los resultados son satisfactorios.

ADITIVOS RETARDANTES DEL FRAGUADO

Los principales componentes de los aditivos retardantes del fraguado son los mism que se usan en los aditivos reductores de agua.

El retardo producido por estos aditivos está limitado por la norma ASTM C-494-92 a no ser mayor de 3 ½ hrs con relación al de referencia tanto para el fraguado inicial com para el final, determinándose los tiempos de fraguado mediante la norma ASTM C-403-90

Los aditivos retardantes del fraguado del concreto han surgido para contrarrestar la reducción en el tiempo de fraguado cuando se presenta una alta evaporación en el colado en clima caliente. Se han estado utilizando indiscriminadamente, siendo que su uso es aplicable fundamentalmente para reducir la aparición de juntas frías.

El efecto en la durabilidad de los aditivos retardantes del fraguado está en función de su influencia en la retracción plástica. La retracción plástica máxima depende, además de lo mencionado en el párrafo en donde se discutió ésta, de la rigidez de la revoltura y del tiempo de fraguado. Entre mayor sea la rigidez de la revoltura y menor el tiempo de fraguado, la retracción esperada será menor, considerando invariables el resto de las condiciones. Por lo mencionado anteriormente, se concluye que el uso de retardantes o aditivos cuyo efecto secundario sea el retardo en el tiempo de fraguado, aumentará la retracción plástica, lo cual confirmaron Ravina y Shalon mediante ensayes de retracción plástica en morteros de consistencia plástica, con y sin aditivo retardante, con un contenido de 550 kg/m³ de cemento Portland, temperatura 30° C, radiación infrarroja y velocidad del viento de 20 km/hr. El resultado se aprecia en la Fig. 6 alcanzándose en el mortero de referencia una retracción máxima de 10.3 mm/m y en el que se utilizó un aditivo retardante de 14 mm/m, que represente un 36% más de retracción. Un aumento en la retracción plástica aumenta el riesgo de agrietamiento. De aquí que el uso de retardantes del fraguado en ambiente caliente y seco debe considerarse con las debidas reservas considerando los posibles agrietamientos, particularmente en superficies planas.

ADITIVOS QUE AUMENTAN LA DURABILIDAD CONTRA LOS EFECTOS CICLICOS DE CONGELAMIENTO Y DESCONGELAMIENTO

Una causa de baja durabilidad del concreto en clima frío es el congelamiento y descongelamiento del concreto endurecido saturado y que, al presentarse en forma cíclica, daña progresivamente al concreto desintegrándolo.

Cuando los poros capilares de la pasta de cemento del concreto, estando parcialmente saturados se congelan, hay un aumento en el volumen de aproximadamente 9%. El congelamiento ocurre gradualmente en tal forma que el agua, aún no congelada en los capilares, está sujeta a presión hidráulica, principalmente por el incremento del volumen del agua al irse formando el hielo. Esta presión, si no se disminuye, origina esfuerzos de tensión, que si la pasta de cemento no los puede soportar, se agrietará. Esto puede ocurrir en capilares completamente saturados en los cuales no puede haber movimiento del agua al iniciarse el congelamiento. Al ocurrir el agrietamiento, aparecen nuevos espacios inmediatos al capilar, los cuales se saturan en el nuevo ciclo incrementándose el volumen de agua congelable. Los ciclos repetidos de congelamiento y descongelamiento tienen un efecto acumulativo deteriorante en el concreto.

Existen otros dos procesos que contribuyen al aumento de la presión hidráulica en el agua aún no congelada en los capilares, la difusión del gel de agua y la ósmosis debida al incremento de la concentración de solubles (17).

La eficacia de un concreto para resistir adecuadamente el ataque de las heladas se puede determinar, mediante el ensaye ASTM C-666, sometiendo el concreto a ciclos de congelamiento y descongelamiento. Se considera que un concreto es adecuado para soportar heladas cíclicas, si es capaz de soportar un mínimo de 300 ciclos y una disminución en el módulo dinámico de no más del 60%.

Inclusores de Aire

Para proteger al concreto de los daños debidos a los ciclos alternativos de congelamiento y descongelamiento, se ha desarrollado el procedimiento de incluir, intencionalmente, burbujas de aire en la pasta de cemento del concreto mediante un aditivo. Los aditivos inclusores de aire (AIA), se definen como aquellos que originan la formación de micro-burbujas de aire uniformemente distribuidas en el mortero o en el concreto, las cuales permanecen después del endurecimiento. Diversos materiales han sido utilizados comercialmente como AIA, una clasificación es la siguiente: (18)

1. Sales de maderas resinosas.
2. Detergentes sintéticos.
3. Sales de lignina sulfonatada.
4. Sales de ácido de petróleo.
5. Sales de materiales proteináceos.
6. Acidos grasos y resinosos y sus sales.
7. Sales orgánicas de hidrocarburos sulfonatados.

La mayoría de los AIA comerciales se expenden en estado líquido, pero hay algunos en polvo, escamas o semisólidos.

Los aditivos inclusores de aire protegen solamente la pasta de cemento y no protegen al concreto por fallas originadas por congelamiento y descongelamiento en el agregado grueso. El constituyente esencial de los aditivos inclusores de aire es un agente tensioactivo que disminuye la tensión superficial del agua, facilitando la formación y estabilización de las burbujas. Las burbujas son independientes del sistema capilar y nunca llegan a llenarse con los productos de hidratación del cemento (17). Las burbujas de aire interceptan los capilares, dando como resultado un menor ingreso de agua y permitiendo un espacio donde se aloje el agua sujeta a la presión hidráulica originada por aquella que se va congelando, comprimiendo el aire en la burbuja y minimizando la presión; ver fig. 7. Si el agua está por arriba del punto crítico de saturación, habrá un espesor crítico más allá del cual la presión hidráulica excede la resistencia del material.

Para cada revoltura de concreto existe un volumen mínimo de huecos requeridos para la protección de las heladas. Se encontró que este volumen corresponde al 9 % del volumen de mortero (19) y es esencial que las burbujas de aire se distribuyan uniformemente en la pasta de cemento. La eficacia de la inclusión de aire se puede estimar por medio del factor de espaciamiento. Este factor es la máxima distancia de un punto en la pasta de cemento a la periferia de la burbuja de aire más cercana (20). Se requiere un factor de espaciamiento máximo de 0.20 mm para una protección total de los daños por congelamiento (21), entre más pequeño sea el factor de espaciamiento mejor será la durabilidad del concreto (22). Mediante la norma ASTM C-457 se puede determinar el factor de espaciamiento en el concreto endurecido. Las burbujas de aire no son todas del mismo tamaño, deberán ser tan pequeñas como sea posible, alcanzan hasta 300 μm, si las burbujas son mayores de 300 μm, la inclusión de aire no es efectiva (23). El tamaño se expresa como la superficie específica por unidad de volumen, en un concreto con una calidad satisfactoria de aire incluido, la superficie específica variará de 16 a 24 mm^2/mm^3 incluyendo el aire atrapado (24).

Para que la inclusión de aire resulte efectiva, se requiere una relación A/C baja para que la pasta de cemento tenga un volumen relativamente bajo de capilares.

De acuerdo con la frecuencia de ciclos de congelamiento esperado, según las condiciones climáticas del sitio donde se localice la estructura, ACI-318 recomienda además de la inclusión de aire, una relación A/C máxima para que el concreto resulte durable a esta acción.

Factores que Influyen en la Cantidad de Aire

Hay diversos factores que influyen en la cantidad de aire resultante en el concreto cuando una cantidad dada de AIA se añade, debido a variaciones en sus componentes, mezclado, manejo y temperatura, algunos de ellos debido a que alteran la tensión superficial del agua y por lo tanto la formación de las burbujas de aire (25); entre estos tenemos:

1. El mezclado con agua dura reduce el contenido de aire incluido.
2. A menudo se requieren mayores dosificaciones de AIA al aumentar la finura de los productos cementantes.
3. Los cementos altos en álcalis aumentan el contenido de aire incluido.
4. Los concretos con altos contenidos de cemento incluirán menos aire que aquéllos con bajos contenidos.
5. Los concretos con ceniza volante pueden incluir menos aire.
6. Un aumento en el contenido de carbón de la ceniza volante, microsílica o bentonita, disminuye la cantidad de aire incluido. Investigaciones recientes sugieren que añadir compuestos polares que se absorben dentro del carbón mejoran el comportamiento de ciertos agentes inclusores de aire en presencia de ceniza de combustible pulverizada.

7. A mayor proporción de finos en el agregado, mayor será el contenido de aire en el concreto. Sin embargo, el material dentro del rango de 300-600 μm lo aumenta.

8. Cuando el tamaño máximo del agregado aumenta, el requerimiento de aire en el concreto disminuye. Este efecto es indirecto, ya que entre mayor sea el tamaño máximo, menor será el porcentaje de mortero.

9. El uso de agregado anguloso aumenta la dosificación requerida de aire incluido, no así el redondeado, para la misma trabajabilidad.

10. Revenimientos menores de 7.5 cm requieren mayores dosificaciones de aditivo. Al aumentar el revenimiento hasta 15.0 cm, aumentará el contenido de aire.

11. Para revenimientos mayores de 15.0 cm, el contenido de aire disminuye y las burbujas de aire son menos estables.

12. Un aumento en la relación A/C da por resultado un aumento del contenido de aire.

13. El uso de aditivos reductores de agua conlleva a un aumento del aire incluido, aún cuando este tipo de aditivo no tenga la propiedad de incluir609Xaire.

14. Se debe optimizar el tiempo y velocidad de mezclado, ya que bajos valores causan la dispersión no uniforme de las burbujas, mientras que mezclar en demasía gradualmente se expulsa algo de aire.

15. La muy rápida rotación de la revolvedora aumenta la cantidad de aire incluido.

16. La prolongada transportación y vibración reduce la cantidad de aire incluido. Por lo que el contenido de aire en el concreto debe ser determinado antes de su colocación.

17. Una alta temperatura del concreto fresco da por resultado una baja cantidad de aire y viceversa. El efecto es más significativo en revenimientos mayores.

18. El vapor a presión atmosférica al curar el concreto puede causar expansión de las burbujas de aire y conducir a un agrietamiento incipiente.

Efectos secundarios

Los AIA mejoran la trabajabilidad y consistencia del concreto fresco como un efecto secundario y además reducen la segregación y el sangrado, de aquí que en el proporcionamiento se pueda reducir la cantidad de agua. La resistencia del concreto endurecido disminuye en la medida que la cantidad de aire aumenta. Los efectos anteriores pueden compensarse con cambios apropiados en el proporcionamiento.

Concreto Resistente al Congelamiento y Descongelamiento sin Aditivo Inclusor de Aire.

R. Philleo (26) sugirió que con el advenimiento del concreto de alta resistencia no era necesario incluir aire en el concreto, considerando que con el uso de un aditivo super reductor de agua y microsílica se podía fabricar un concreto con una relación agua/material cementante (A/CM) muy baja, que hace al concreto tan impermeable y con poros tan pequeños, que el agua de saturación existente por exposición natural no puede congelarse en ellos a temperaturas ordinarias. En esta forma se evitaría la desventaja de la disminución de la resistencia al incluir aire al concreto.

Para confirmar esta teoría, recientemente, agosto de 1992, W. Dolch y coautores (27) realizaron una investigación en concreto de alta resistencia con microsílica, aditivo superfluidificante y sin aditivo inclusor de aire, utilizando una relación A/CM de 0.35. El resultado indicó un mejoramiento en las propiedades de resistencia al congelamiento; sin embargo, falló en cumplir con la especificación ASTM C-666, procedimiento A. Es necesaria mayor investigación con relaciones A/CM mucho más bajas, en donde el concreto no contenga agua que se pueda congelar.

ADITIVOS ANTICORROSIVOS

La corrosión del acero de refuerzo es causa importante para una baja durabilidad del concreto reforzado. Cierto tiempo después de iniciado el proceso de corrosión del acero, la fuerza ejercida por el aumento en volumen de la reacción de oxidación, llega a vencer la reacción ofrecida por el concreto de recubrimiento y, dependiendo del tipo de elemento, puede aparecer originalmente una grieta longitudinal o una botadura del recubrimiento. La corrosión del acero se acelera, disminuyendo progresivamente su sección efectiva de trabajo.

La corrosión del acero de refuerzo en el concreto se produce mediante un complejo mecanismo, el cual está en función de:

1. El proporcionamiento del concreto.
2. El tipo y superficie específica del cemento.
3. La relación agua-cemento (A/C).
4. El contenido de sales.
5. Las condiciones de curado, edad e historia ambiental del concreto.

6. La carbonatación.
7. La temperatura y humedad relativa del concreto.
8. La rugocidad y limpieza del refuerzo.
9. El recubrimiento de concreto.

La corrosión se inicia cuando llega a presentarse un fenómeno electro-químico de flujo de corriente eléctrica (28), conocido como la formación de una media celda, que se presenta en un tramo de la varilla, que puede ir desde 1.0 cm hasta 6.0 m o más. Uno de los extremos de este tramo actúa como cátodo donde ocurre la reducción electro-química y el otro como ánodo, que es donde se produce la oxidación electro-química, formándose óxido de fierro, el cual puede expandirse a ocupar de dos a casi siete veces el volumen del acero original (29).

El flujo de la corriente sólo es posible si existe un medio conductor eléctrico. El concreto seco y sin la presencia de sales no actúa como tal, pero cuando contiene sales solubles en cantidad suficiente, como son los sulfatos o cloruros y con la presencia de humedad se convierte en un efectivo electrólito capaz de conducir corriente eléctrica mediante el flujo de iones. Para que se produzca la corrosión es indispensable además, la presencia de oxígeno (28). Por lo tanto para que se produzca la corrosión es necesaria, simultáneamente, la presencia del electrólito, un medio húmedo y oxígeno.

La estructura, la distribución de los tamaños y la interconexión de los poros en la pasta de cemento, determinan la disponibilidad de oxígeno y de humedad en la superficie del acero. Ellos también determinan la velocidad de penetración de los iones cloro y del CO_2, los cuales son los dos causantes más comunes de la corrosión del acero embebido (30).

El concreto de cemento Portland, debido a su contenido de $Ca(OH)_2$ que se origina durante la hidratación del cemento, aumenta la alcalinidad a valores de pH mayores a 12.5 (4), creando un medio excelente para proteger al acero de la corrosión, forma una película protectora de óxido de fierro en la superficie del metal, que es pasivadora. Cuando se alcanzan valores de pH por abajo de 11.0, se destruye la película pasiva protectora.

Solamente cuando se presentan sales, ó cuando el recubrimiento de concreto del refuerzo se carbonata, alcanzando valores de pH de cerca de 9.0, se despasiviza la película protectora, se facilita la conducción eléctrica y se hace vulnerable el acero a la corrosión.

Existen diversas formas para mantener pasiva la película protectora, a saber:

1. Aplicando membranas o selladores del concreto que impidan la penetración del CO_2, de la humedad, del oxígeno o de iones cloro en el concreto.
2. Dando un tratamiento químico a la superficie del acero (galvanizado) o aplicando pinturas epóxicas anticorrosivas.
3. Aumentando la impermeabilidad del concreto a los líquidos y gases, haciéndolo más denso.

4. Instalando una protección catódica.
5. Adicionando al concreto aditivos inhibidores de la corrosión, minerales u orgánicos.

Todos los métodos anteriores requieren de un análisis de eficiencia-economía para tomar una decisión en su uso si se desea aumentar la vida útil de una estructura de concreto.

Por lo que se refiere al uso de aditivos, diversos productos químicos minerales u orgánicos, han sido evaluados como potenciales inhibidores de la corrosión, los cuales incluyen, entre los minerales, benzoato de sodio, nitritos, floruros y otros, aunque recientemente se han encontrado resultados contradictorios en el uso de nitritos de calcio.

C. Andrade y co-autores (31), encontraron que una adición de 3% de nitrito de calcio en peso del cemento protegía al refuerzo de la corrosión en concreto carbonatado.

Berke y Rosemberg (32), realizaron estudios sobre la efectividad del nitrito de calcio en concretos con distintas cantidades de cloruros. La eficacia del nitrito para evitar la corrosión está en función de la cantidad de cloruros, ya que entre mayor sea esta cantidad, mayor será el consumo de nitrito de calcio para inhibir la corrosión.

Sin embargo, Collepardi y co-autores (33) en sus investigaciones sobre especímenes de concreto agrietados sumergidos en agua de mar, la presencia de nitritos de sodio, que supuestamente inhibiría la corrosión, realmente hizo la corrosión por cloruro más severa. Entre más angosta sea la grieta, mayor tiempo se requerirá para que se presenten los efectos corrosivos. Puesto que el nitrito de sodio no afecta la difusión del cloro en especímenes de concreto sin grietas, es de esperarse, que el efecto negativo esperado del nitrito de sodio pueda presentarse aún en áreas no agrietadas pero a largos períodos de inmersión.

Hope y co-autor (34) en sus estudios preliminares, encontraron que una combinación 1:11 de nitrito de calcio y molibdato de sodio, mostraba ser más efectiva en la protección a la corrosión que el nitrito de calcio solo.

En ausencia de cloruros, la velocidad de carbonatación del concreto está en función de la porosidad y de la permeabilidad del concreto. En la Fig. 8 se presentan resultados obtenidos por M. Collepardi (35), en concretos de cemento Portland I, en un ambiente enriquecido de CO_2 y una humedad relativa del 75%, donde se puede observar que la velocidad de carbonatación del concreto se ve disminuida al bajar la relación A/C y también se ve disminuida la permeabilidad al agua y al aire, ingredientes necesarios para producir la corrosión. De aquí que cualquier aditivo capaz de reducir la porosidad y permeabilidad del concreto puede considerarse como un aditivo reductor de la corrosión. Por lo tanto, los aditivos superfluidificantes, son realmente reductores de la corrosión del acero, cuando se utilizan como super reductores de agua.

Se debe poner especial atención al uso de un aditivo potencialmente inhibidor de corrosión del acero, para cada caso de exposición del concreto al CO_2, que en el agua en el aire puede carbonatar el hidróxido de calcio, a la acción de cloruros o al curado e autoclave de los elementos de concreto, ya que este procedimiento reduc considerablemente la alcalinidad del concreto.

ADITIVOS QUE REDUCEN LA REACCION ALCALI-AGREGADO

La reacción álcali-agregado es un proceso físico químico en el cual ciertos agregados reactivos reaccionan con los álcalis presentes o inmigrantes a la masa del concreto. L reacción es expansiva y puede producir agrietamientos. La reacción más común es la d los álcalis con agregados silicosos.

En la reacción álcali-sílica se produce un gel que tiene las propiedades de tomar agua expandirse y causar esfuerzos internos que originan agrietamientos y el deterior progresivo del concreto.

La reacción álcali-sílica puede producirse por:

1. Un cemento que contenga suficiente cantidad de sodio y de potasio que pueda elevar el pH cuando el cemento se hidrata.
2. Un agregado reactivo que contenga sílice en una forma que pueda ser disuelta por soluciones con pH más altas que las del concreto común de 12.5 pH.
3. Los álcalis pueden ingresar al concreto en soluciones de agua desde el exterio

Las soluciones actuales están orientadas a disminuir los ingredientes de la reacción Si los agregados son reactivos, utilizando cemento bajo en álcalis con no más de 0.6% Si el cemento es alto en álcalis se pueden utilizar cantidades apropiadas de suplemento cementantes como escoria granulada o puzolanas, éstas, por un lado disminuyen e porciento total de álcalis en el total del cementante y, tomando algo del hidróxido d calcio, bajan la alcalinidad. En cualquier caso, deberá verificarse la eficacia realizando la pruebas recomendadas por ASTM C 441.

Se han reportado importantes reducciones en los efectos de la reacción utilizand como aditivo sales solubles de litio, bario o sodio. Debido a la limitada información en e uso de estos aditivos, el Comité ACI 212-91, no tiene una práctica recomendada y sugier que cualquier usuario de estos aditivos deberá ensayarlos ampliamente antes de procede a su uso en el campo.

Si con la presencia de agregados reactivos y los álcalis se llega a producir la reacción en donde es necesaria la humedad, un aditivo super reductor de agua, podría considerarse como reductor de los efectos de la relación álcali-agregado, en la medida en que al disminuirse la relación A/CM aumenta la impermeabilidad e ingresa menor cantidad de agua en el concreto, que es un ingrediente de la reacción y además la disminución en la relación A/CM aumenta la resistencia a las fuerzas expansivas de la reacción.

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Raymundo Rivera Villarreal,
Jefe Depto. Investigación en Tecnología
del Concreto, Fac. de Ingeniería Civil,
Universidad Autónoma de Nuevo León, México.

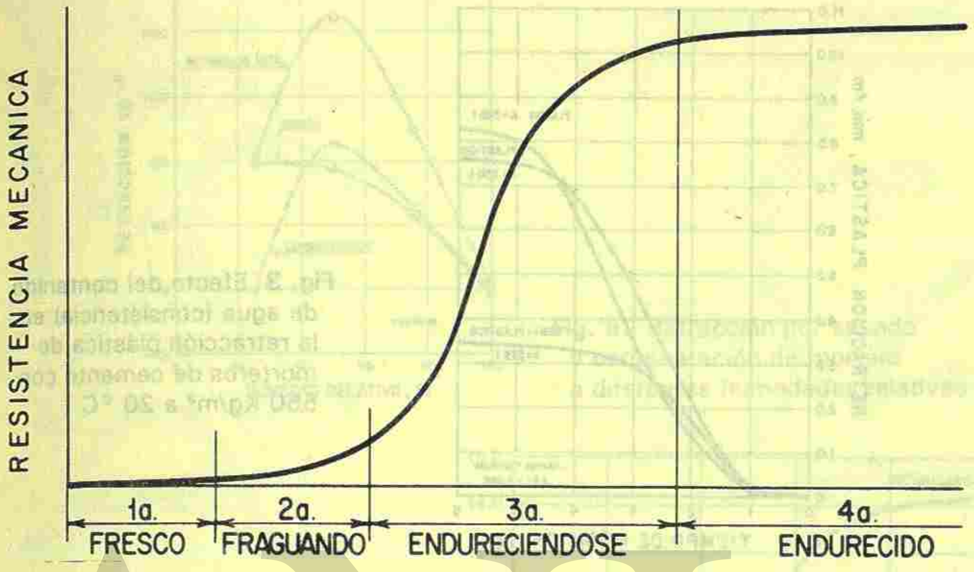

Fig. 1. Etapas de la vida del concreto

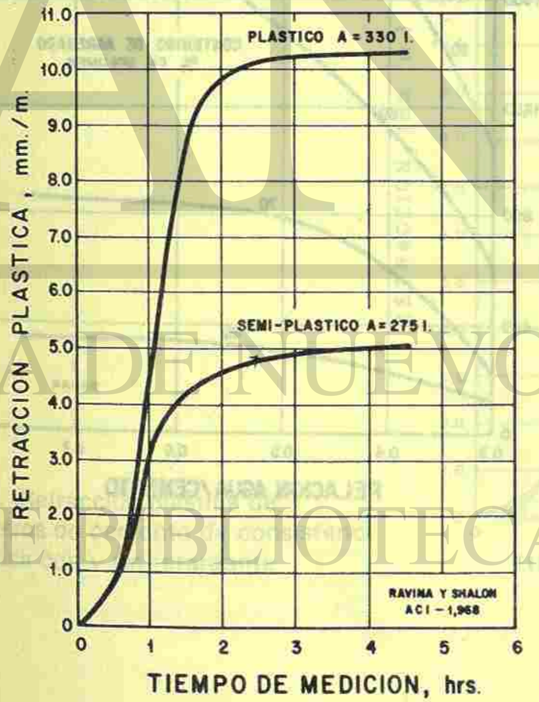

Fig. 2 Efecto del contenido de agua (consistencia) en la retracción plástica de morteros de cemento con 550 Kg/m³ a 30 °C.

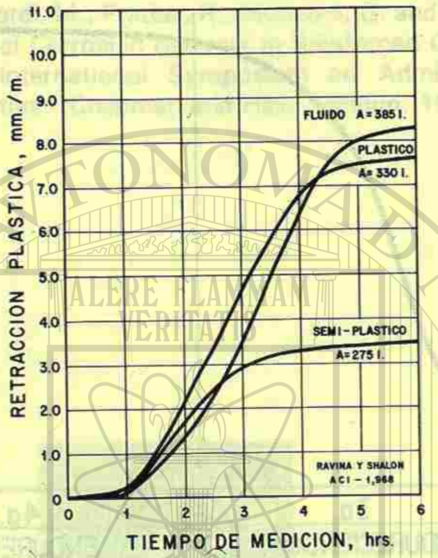

Fig. 3 Efecto del contenido de agua (consistencia) en la retracción plástica de morteros de cemento con 550 Kg/m³ a 20 °C

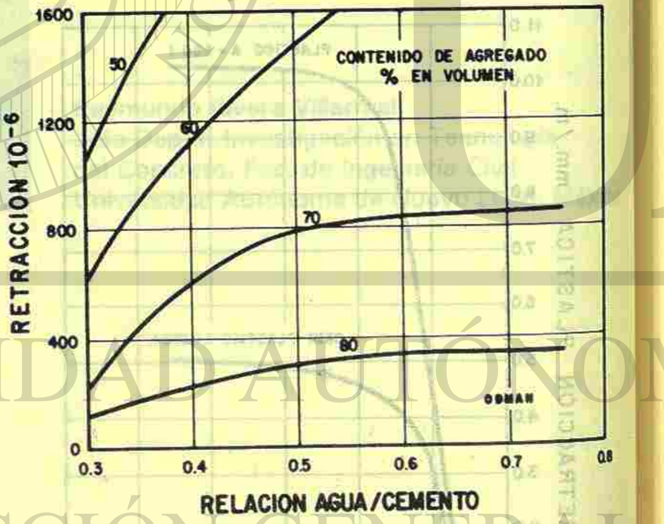

Fig. 4. Influencia de la Relación A/C y el contenido de agregado en la retracción

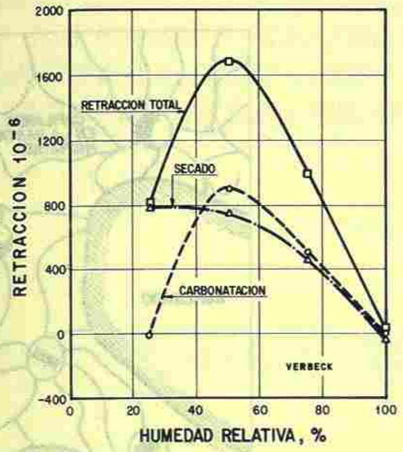

Fig. 5. Retracción por secado y carbonatación de mortero a diferentes humedades relativas

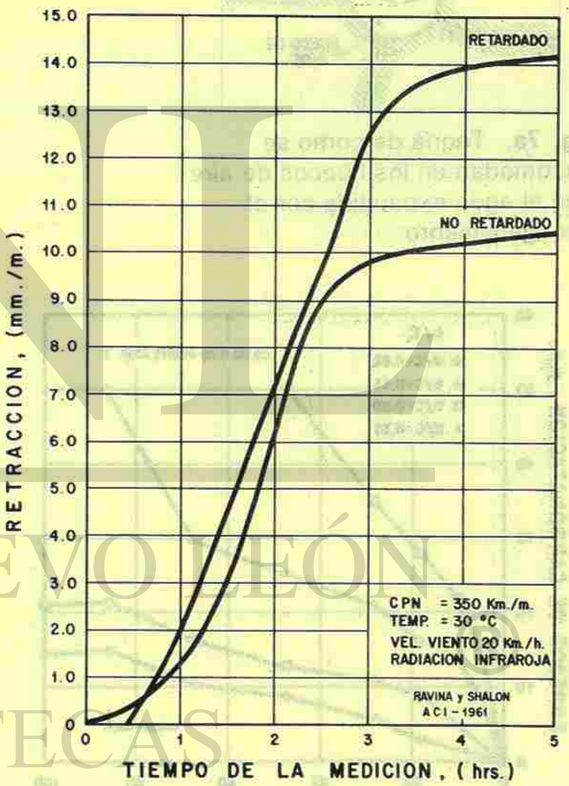

Fig. 6. Retracción plástica de morteros de cemento de consistenci plástica con y sin retardante

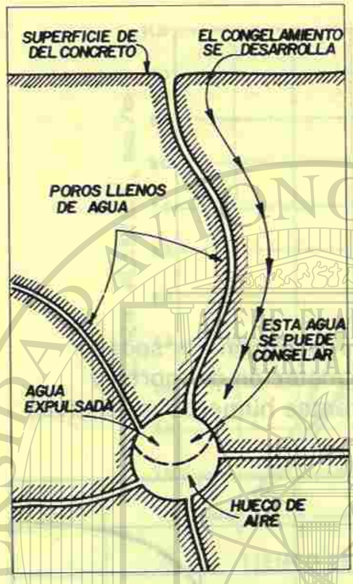

Fig. 7a. Teoría de como se acomodan en los huecos de aire en el agua expandida por el congelamiento

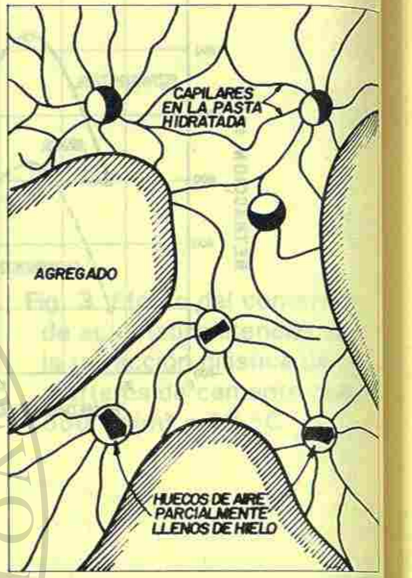

Fig. 7b Concreto congelado con inclusor de aire

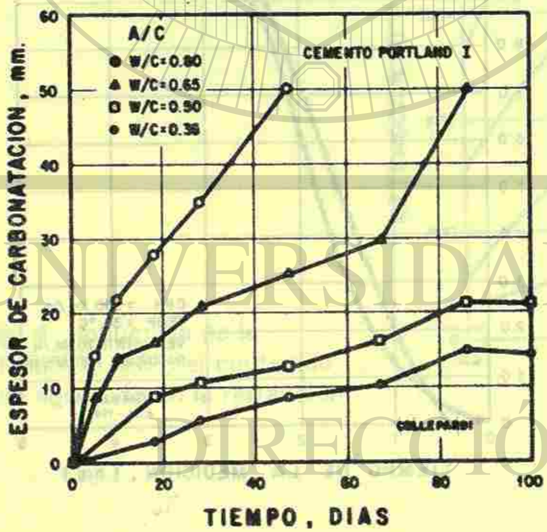

Fig. 8 Tiempo de exposición, contra espesor de carbonatación, ambiente enriquecido con CO_2 y H.R. = 75%, para distintas relaciones A/C

LA QUIMICA DE LOS SUPERFLUIDIFICANTES DEL CONCRETO: REOLOGIA Y CINETICA DE LA HIDRATACION DE LAS PASTAS DE CEMENTO PORTLAND CONTENIENDO MEZCLAS DE SUPERFLUIDIFICANTES A BASE DE NAFTALINA Y MELAMINA

por

Carmel Jolicoeur, François Perreault,
Marc-André Simard y André Nuyt

Sinopsis: Se midieron las áreas fluidas utilizando un mini cono de revenimientos y el calor de hidratación de las pastas de cemento conteniendo una relación agua/cemento de 0.35, en presencia de diversos superfluidificantes comerciales disponibles usándolos, ya sea en forma pura o en mezclas binarias de 50:50. Se ha estado investigando la influencia de los parámetros siguientes: variación en los compuestos del cemento, tipo de superfluidificante: sulfonato de polinaftalina de sodio y calcio (NaPNS y CaPNS) y sulfonato de polimelamina de sodio (NaPMS); y la concentración total de superfluidificante.

Los resultados de los experimentos reológicos (mini revenimiento) muestran que la fluidez de las pastas de cemento depende de las características de los superfluidificantes y del cemento usados. Se observó el efecto de saturación con diversas combinaciones de cemento-superfluidificante; por ejemplo: arriba de una concentración dada, un aumento de la concentración de superfluidificante tiene poco efecto sobre la fluidez. A bajas concentraciones de superfluidificante, las pastas de cemento fluidificado con mezclas binarias de 50:50 (o 1:1) de superfluidificantes, con frecuencia tenían fluideces intermedias de aquéllas conteniendo compuestos puros. A mayores concentraciones de superfluidificante, el efecto de saturación a menudo evita la diferenciación entre el efecto de los diferentes aditivos o mezclas de ellos. Los resultados de los experimentos calorimétricos muestran que la magnitud del efecto retardante depende del tipo de superfluidificante usado y que las propiedades medidas varían en forma lineal con la composición de la mezcla binaria de superfluidificante.

Palabras clave: Cemento, superfluidificante, sulfonato de naftalina, sulfonato de melamina, reología, calorimetría.

THE CHEMISTRY OF CONCRETE SUPERPLASTICIZERS: RHEOLOGY AND HYDRATION KINETICS OF PORTLAND CEMENT PASTES CONTAINING MIXTURES OF NAPHTHALENE- AND MELAMINE-BASED SUPERPLASTICIZERS

by

Carmel Jolicoeur, François Perreault, Marc-André Simard
and André Nuyt

Synopsis: The minislump spread areas and the heat of hydration of cement pastes having a water/ cement ratio of 0.35 were measured in the presence of several commercially available superplasticizers used, either in pure form, or in 50:50 binary mixtures. The influence of the following parameters have been investigated: variation in cement composition; type of superplasticizer: sodium and calcium polynaphthalene sulfonate (NaPNS and CaPNS) and sodium polymelamine sulfonate (NaPMS); and total superplasticizer concentration.

The results of rheological (minislump) experiments show that the fluidity of cement pastes depends on the characteristics of the superplasticizer and of the cement used. With various cement-superplasticizer combinations, a saturation effect is observed, i.e., above a given concentration, an increase of the superplasticizer concentration has little effect on the fluidity. At low superplasticizer concentrations, the cement pastes fluidified with 50:50 (or 1:1) binary mixtures of superplasticizers usually have intermediate fluidities between those of cement pastes containing the pure compounds. At higher superplasticizer concentrations, the saturation effect often precludes differentiation between the effects of the different additives or mixtures of these. The results of calorimetric experiments show that the magnitude of the retardation effect depends on the type of superplasticizer used and that the measured properties vary linearly with the composition of binary superplasticizer mixtures.

Keywords: Cement, superplasticizer, naphthalene sulfonate, melamine sulfonate, rheology, calorimetry.

C. Jolicoeur is Professor of Chemistry at the Université de Sherbrooke since 1971. H specializes in the physical chemistry of solutions and colloidal systems with application in several areas, for example: mineral processing, cement and concrete, and wate treatment. He is currently one of the principal investigators of the Canadian Network o Centres of Excellence on High Performance Concrete, and a member of the Centre d recherche interuniversitaire sur les bétons (CRIB).

F. Perreault and M.-A. Simard are Research Assistants in the laboratory of Professo Jolicoeur.

A. Nuyt is Professor of Chemistry at the Séminaire de Sherbrooke. Since 1989 he i also director general of this institution.

INTRODUCTION

The use of superplasticizers in the production of concrete leads to a major increase i the workability, and enables a reduction in either the water, or both the water and th cement quantities used depending on the application (1). In addition to their influence o rheological properties, the superplasticizer may affect other properties of fresh concre which are related to the kinetics of hydration, such as the lag period, early strength, etc In practice, these properties are not severely altered by high range water reducers, sinc standard specifications define limiting acceptable values (2).

Due to the wide range of cement and admixture types available, and to the inhere variability in their industrial production, compatibility problems may sometime arise wi particular cement-admixture combinations. For example, a superplasticizer may affect th rheological properties of various cements in different ways depending on the chemic composition of the cements, or on their physico-chemical characteristics (3,4). On th other hand, a cement may behave differently with various superplaticizers; this ma depend on factors such as their chemical type (naphthalene- or melamine-based (5)), the molar mass (6), their counterion (sodium or calcium), or the presence of by-products.

Blends of different superplasticizers may also be used for the production of concret For example, lignosulfonates are frequently mixed with synthetic superplasticizers f economical or technological reasons. Blends of naphthalene- and melamine-base superplasticizers have also been used occasionally as reported in the literature (4 However, systematic comparisons of the behavior of superplasticizer mixtures with th of pure components are not available.

In this paper, we initiated an investigation to determine how the rheological propert and the hydration kinetics of cement pastes are altered by binary mixtures o superplasticizer, as compared to the influence of the pure admixtures. In particular, w

aim to establish if there are any synergetic or antagonistic effects when superplasticizer blends are used.

EXPERIMENTAL

Materials

Three commercially available superplasticizers have been selected for this study: sodium poly-β-naphthalene sulfonate (NaPNS), calcium poly-β-naphthalene sulfonate (CaPN-S), and sodium polymelamine sulfonate (NaPMS). The poly-β-naphthalene sulfonates were supplied by Handy Chemicals as concentrated solutions, and the polymelamine sulfonate was obtained from SKW in a solid form. The superplasticizers were characterized by ionic chromatography (sulfate analysis), by ICP spectrometry (sodium and calcium analysis) and by ion pairing HPLC (molar mass distribution). Typical chromatograms are shown in Figure 1, for NaPNS and NaPMS. The percentage of high molar mass (HMM) chains in the polymers can be calculated from the area under the large peaks eluted at the longer retention times. This HMM fraction, together with other relevant superplasticizer features are summarized in Table 1. The results of ultrafiltration experiments on Na- or CaPNS have shown that the peak at $\approx$ 35 min (Fig. 1a) represents the polymers with molar mass ranging from 5 kD (Dalton = g/mol) to more than 100 kD (7). For PMS, ultrafiltration data showed that the peak at $\approx$ 30 min (Fig. 1b) represents mainly the 5-30 kD polymers, while that at 55 min is due to polymer with still higher molar mass (8).

Three Portland cements of different origin were used in this study. Their phase compositions, SO_3 and alkali contents, losses on ignition, and Blaine surface areas are presented in Table 2.

Methods

Mixing procedure -- The cement pastes were prepared at a water/cement ratio (W/C) of 0.35. The superplasticizer was first mixed with water and the cement was added to the solution. The paste was mixed manually for one minute and then with a high speed mixer for two minutes in order to obtain a well dispersed slurry. The mixing procedure was made under controlled temperature conditions to obtain a final temperature of 25 ±1°C. The superplasticizer concentration in the mix is expressed on a dry weight basis (weight of superplasticizer relative to the weight of cement). For binary mixtures of superplasticizers, the total concentration and the mass ratio of the superplasticizers are indicated.

Rheology -- The paste fluidity was determined as function of time following a method described by Kantro (9). In the latter, the cement paste is poured into a plexiglass cone having the same geometry as the Abraham cone for regular slump tests, but with reduced dimensions (height of 6 cm). The minicone is raised and the area of the spread area of the cement paste is measured.

Calorimetry -- Hydration thermograms of cement pastes have been recorded for a period of 24 hours with an isoperibol calorimeter coupled with a data acquisition system. The experimental procedure used in the present work is similar to that described by Simard *et al.* (10).

RESULTS AND DISCUSSION

Rheological studies

Minislump areas of cement pastes prepared with pure superplasticizers and with their 1:1 binary mixtures have been measured as function of time (10, 60 and 120 minutes) at four total superplasticizer concentrations (0.3, 0.5, 0.7 and 1 wt %).

Saturation point -- The minislump areas recorded at 10 minutes for pastes of cements A and B fluidified with the pure superplasticizers at different concentrations are shown in Figure 2. For the two cements, the efficiency of the superplasticizers to increase the paste fluidity varies in the order NaPNS > CaPNS > NaPMS. The behavior of each cement upon addition of superplasticizer is otherwise quite different: Cement A is easier to fluidify than cement B (i.e., at the same superplasticizer concentration, the minislump areas are much greater for cement A). Pastes of the latter cement also show a saturation effect at a superplasticizer concentration between 0.5 and 0.7% for NaPNS and CaPNS; beyond the saturation point, an increase in superplasticizer concentration has little effect on the fluidity. For cement A containing NaPMS, and for cement B with all superplasticizers, the saturation point would be at 1% or more. Similar results were also found for the other measurement times investigated (60, 120 min).

Superplasticizer mixtures -- As noted above, different superplasticizers can exhibit important differences in their effectiveness to fluidify cement pastes. In this section, we present the results of cement paste fluidification obtained with binary mixtures of superplasticizers.

Minislump areas at 10, 60 and 120 min after mixing are shown in Figure 3 for pastes of cement A containing, either pure, or 1:1 binary mixtures, of superplasticizers at four total concentrations (W/C = 0.35). At admixture concentrations of 0.3%, the initial fluidity is low and the loss of fluidity as function of time is important. For most pastes, measurements cannot be made at 120 minutes. However, at other measurement times, differences in efficiency can be observed between the three pure superplasticizers; this efficiency varies in the order NaPNS > CaPNS > NaPMS. The difference between NaPNS and CaPNS may be attributed mainly to their relative content in high molar mass polymers. For the 1:1 binary mixtures, the fluidities appear intermediate between those obtained with the pure superplasticizers.

At admixture concentrations of 0.5%, cement paste fluidities increase and the relative loss in fluidity as function of time is greatly reduced. Binary mixtures of superplasticizers

again show fluidity results intermediate between those of the pure admixtures, and this is observed at the three measuring times.

For admixture concentrations of 0.7 and 1%, a saturation point is reached with NaPNS and CaPNS as noted previously for pastes containing either pure superplasticizer or their mixture. At a total concentration of 0.7%, the NaPMS-NaPNS mixture yields fluidities similar to those obtained with pure NaPNS, while the 1:1 binary mixture of NaPMS and CaPNS, which has not yet reached the saturation point, gives fluidities intermediate between those of the pure admixtures at the three times of measurement. The behavior of the NaPMS/NaPNS mixture (0.7%) is a first indication of deviations from a simple proportionality mixing rule.

Similar results of minislump experiments with cement B containing pure or binary mixtures of superplasticizers are shown in Figure 4. As noted earlier, at the same admixture concentration, cement B is much less fluid than cement A. This is due in part to the higher Blaine area of cement B and, possibly, to differences in the chemical composition of the two cements. Despite the lower paste fluidities, the general trend of results as function of superplasticizer concentration or as function of admixture types, is similar to that observed for cement A. All 1:1 binary mixtures show results intermediate between those of the pure admixtures, except for the mixture CaPNS-NaPNS at a concentration of 0.7%, where the results are closer to those with NaPNS, and at 1%, where there is no variation of fluidity between NaPNS or CaPNS or their mixture. Again, slight deviation from a simple proportionality rule in the mixed superplasticizer are only evidenced at a concentration ≈ 0.7 wt %.

For cement C, minislump areas have only been measured at a superplasticizer concentration of 0.7%, the results are presented in Figure 5. With the 1:1 binary mixtures, CaPNS-NaPNS and NaPNS-NaPMS, fluidities are intermediate between those of the pure components. However, with the NaPMS-CaPNS mixture, a synergistic effect can be observed, the fluidities being higher than those with the pure components. This behavior is rather unique and would appear related to the particular composition of cement C (Table 2).

Calorimetric Studies

The hydration thermograms were recorded over a period of 24 hours for pastes of each of the three cements containing pure or binary mixture of superplasticizers at a total concentration of 0.7%. The temperature-versus-time curves (integral heat) and their derivatives (heat flux) are illustrated in Figure 6 for cements A, B, and C containing 0.7% of NaPNS.

As evidenced by these curves, the hydration kinetics of the three cements show significant differences. For cements A and B, which have comparable chemical compositions, the shape of the heat flux curves is similar, showing a two steps reaction. The lag period and the maximum heat flux found with these cements are however quite different: cement B has a longer lag period and lower maximum heat flux than cement A. Cement C, which is very low in C_3A, shows an extended lag phase and a heat flux curve with only one main peak. However, the hydration reactions beyond this peak are more exothermic than for the other cements, giving a higher temperature increase after 24 hours.

The times evolved to reached the maximum heat flux values are plotted in Figure 7 for the various cement-superplasticizer systems investigated. The shifts in the heat flux peak due to the change in superplasticizer type are of comparable magnitude for cements A and B, but appear more important for cement C. A comparison of the results for the three cements shows that the time to reach maximum heat flux is longest with CaPNS and shortest with NaPNS. For cements A and B, the shift in the heat flux peak with these two superplasticizers is about one hundred minutes, while for cement C, it is about two hundred minutes. With NaPMS, the shifts are approximately half-way between those with CaPNS and NaPNS.

For the 1:1 binary mixtures of superplasticizers, the times to reach maximum heat flux are generally intermediate between those for the pure superplasticizers. One exception to this behavior is the 1:1 mixture of NaPNS and NaPMS with cement B for which the time is much higher than for the two pure compounds.

CONCLUSIONS

The results of this exploratory investigation on the influence of several pure and mixed superplasticizers on the rheological properties and hydration behavior of different cements enable the following conclusions.

As frequently observed, each cement-superplasticizer combination exhibit some peculiarities in the rheological properties and hydration kinetics. With the types of systems (pastes at W/C = 0.35) and measurement techniques investigated here, the peculiarities due to the superplasticizer (for a given cement) are best evidenced at relative superplasticizer contents of approximately 0.7 wt %.

Mixture of superplasticizers (1:1) were generally found to exhibit behaviors intermediate (average) between those observed with the pure superplasticizers at the same total concentration. At a concentration of 0.7 wt %, significant deviations from average behavior were found, in the rheology of pastes with one cement (A) in the presence of NaPNS/NaPMS mixtures. In another case, a positive synergy is found in the fluidification of cement C (low C_3A) with mixtures of NaPMS and CaPNS (i.e., fluidity higher with the 1:1 mix than with the pure compounds).

The influence of superplasticizers on the hydration kinetics of cement pastes, as measured by their hydration thermograms, also reflect peculiarities of cements and cement-superplasticizer combinations. In this case, changes induced by the

superplasticizers in the measurable parameters (lag time, time to maximum heat flux) are relatively smaller than those observed in the paste rheology; with superplasticizer mixtures, however, the proportionality rule is generally observed (i.e., the influence of a 1:1 superplasticizer mixture is intermediate between those of the pure superplasticizer, except in one case, where a synergistic effect was apparent (cement B) with a NaPNS/NaPMS mixture).

It is noteworthy that superplasticizer mixtures which lead to deviations from the simple proportionality rule, involve both the melamine- and naphthalene-based superplasticizers; mixtures of Na and Ca naphthalene sulfonates were found to behave predictably in all cases.

ACKNOWLEDGEMENTS

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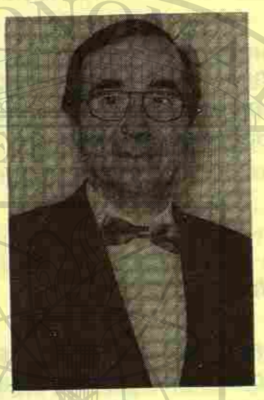

Prof. Carmel Jolicoeur,
Professor of Chemistry at the
Université de Sherbrooke,
Canada

Table 1: Main Characteristics of the Superplasticizers

	NaPNS	CaPNS	NaPMS
Base	Naphthalene	Naphthalene	Melamine
Na (%)'	11.7	0.8	10.0
SO_4 (%)	0.3	7.8	0.0
Ca (%)	2.9	0.0	2.0
HMM (%)	61.0	50.0	-----

All results are expressed on a dry weight basis.

Table 2: Cement Composition and Physical Properties

	Cement		
	A	B	C
C_3S (% (wt))	55.24	49.40	72.73
C_2S	18.01	21.85	11.10
C_3A	5.71	6.30	3.60
C_4AF	8.89	8.64	6.48
SO_3	3.45	2.95	2.37
Na_2O	0.16	0.48	0.18
K_2O	0.53	0.63	0.36
LOI	2.21	3.52	0.49
Blaine area (m^2/g)	362.00	389.00	300.00

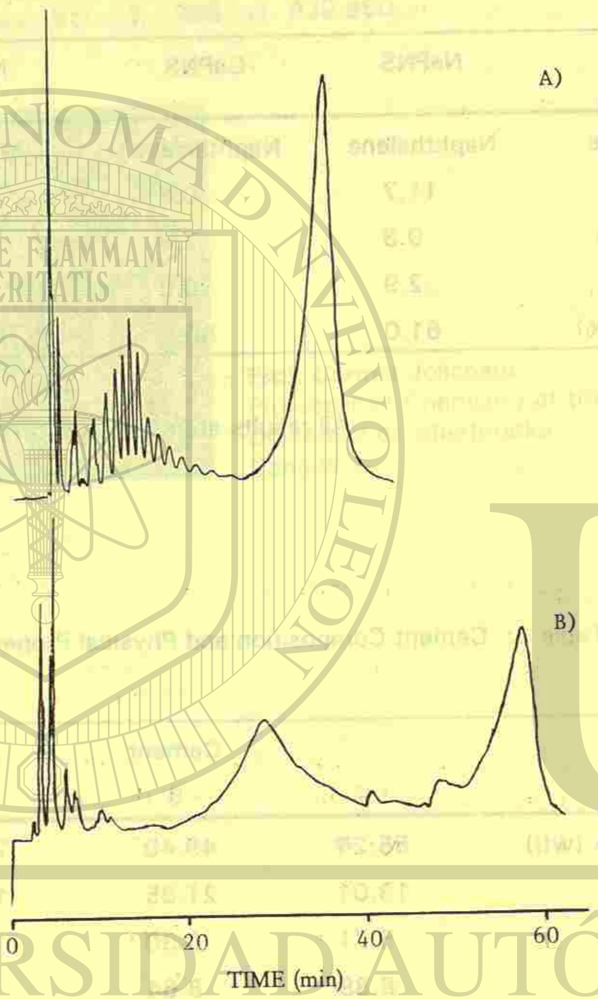

Fig. 1: HPLC chromatograms of the superplasticizers. a) NaPNS; b) NaPMS.

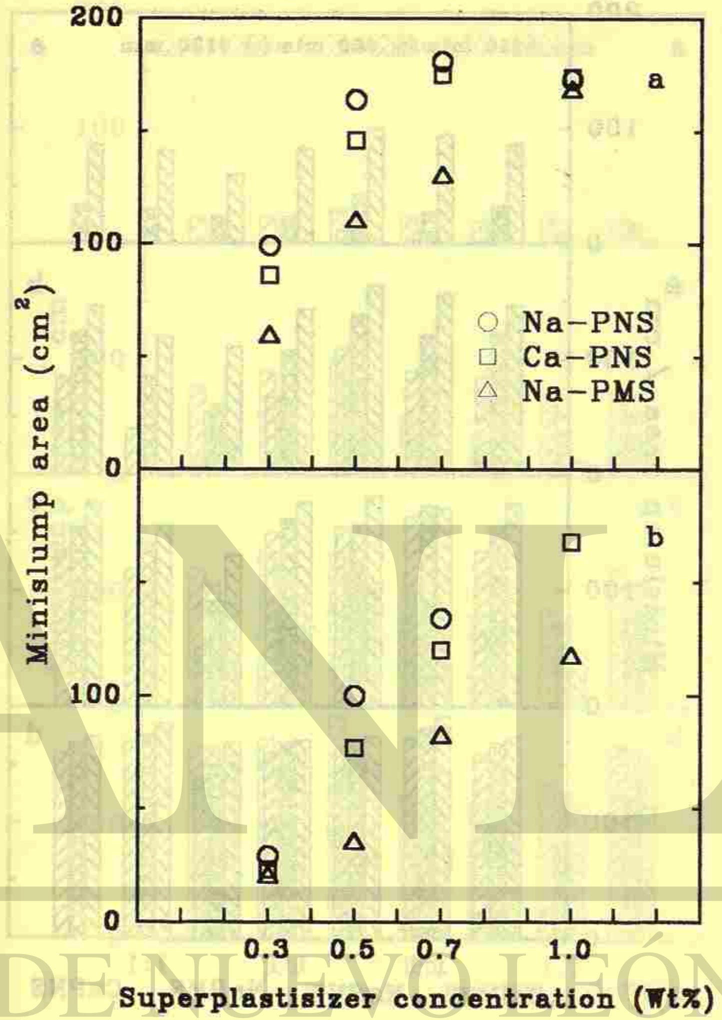

Fig. 2: Minislump area of cement pastes, W/C = 0.35 as function of superplasticizer concentration. a) cement A; b) cement B. ○: NaPNS; □: CaPNS; △: NaPMS.

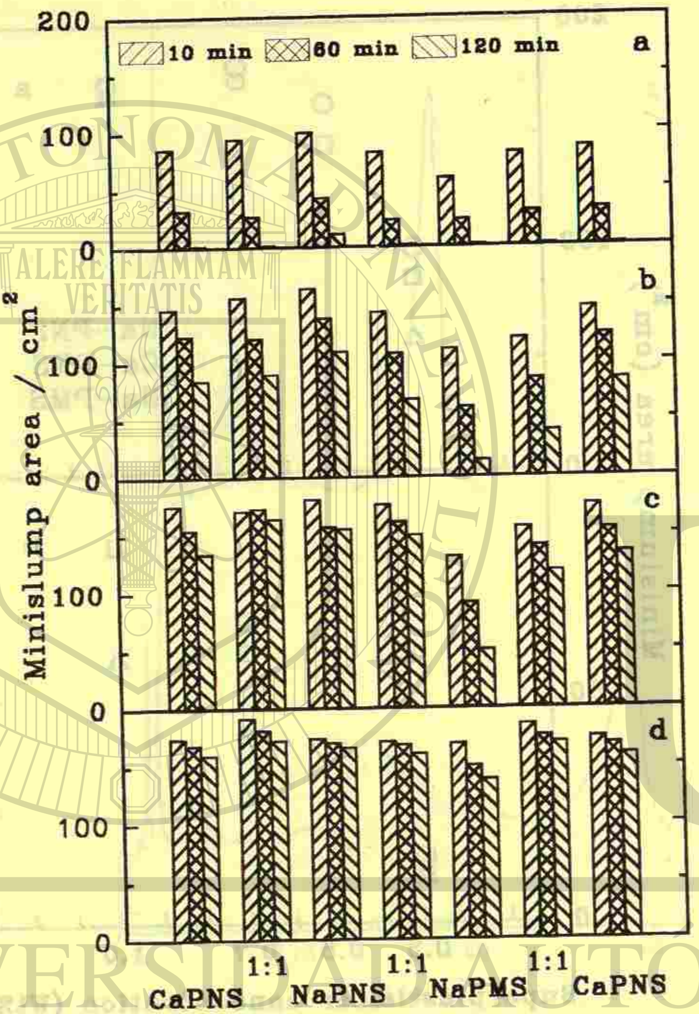

Fig. 3: Minislump areas of pastes of cement A, W/C = 0.35, with the various superplasticizers or their 1:1 binary mixtures. a) 0.3%; b) 0.5%; c): 0.7%; d) 1%.

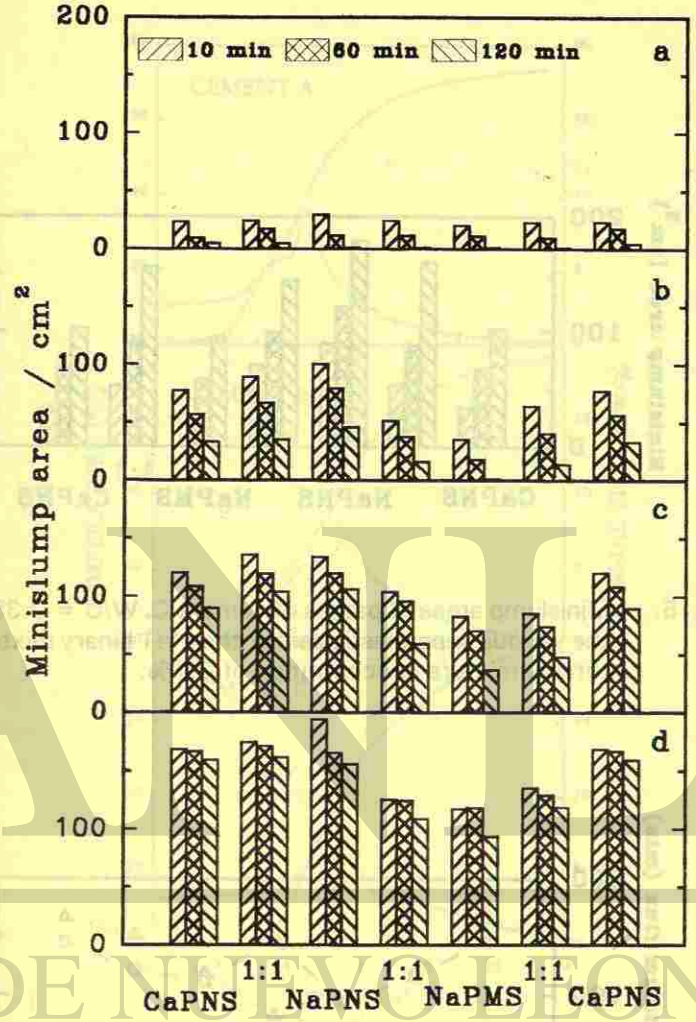

Fig. 4: Minislump areas of pastes of cement B, W/C = 0.35, with the various superplasticizers or their 1:1 binary mixtures. a) 0.3%; b) 0.5%; c): 0.7%; d) 1%.

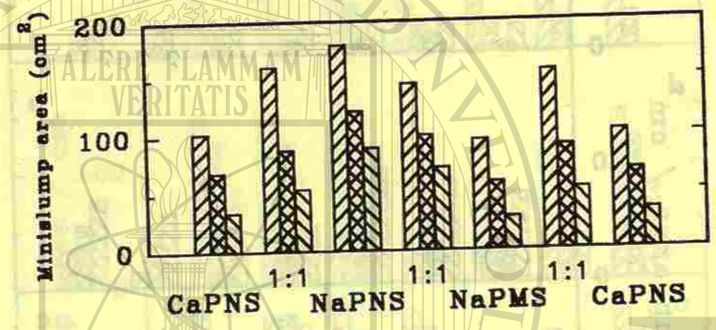

Fig. 5: Minislump areas of pastes of cement C, W/C = 0.35, with the various superplasticizers or their 1:1 binary mixtures at total admixture concentration of 0.7%.

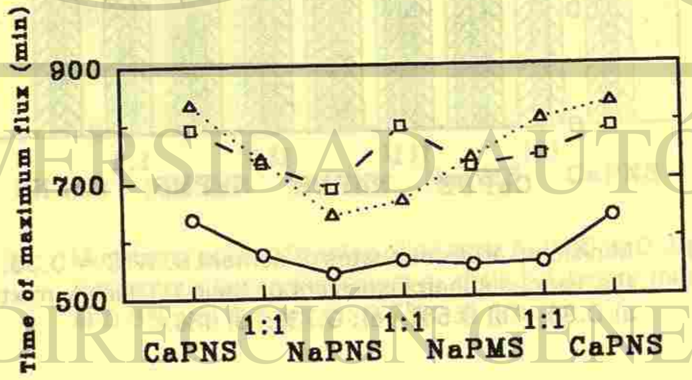

Fig. 7: Time elapsed to maximum heat flux of cement pastes, W/C = 0.35 containing 0.7% of the various superplasticizers or their 1:1 binary mixtures. ○: cement A; □: cement B; △: cement C.

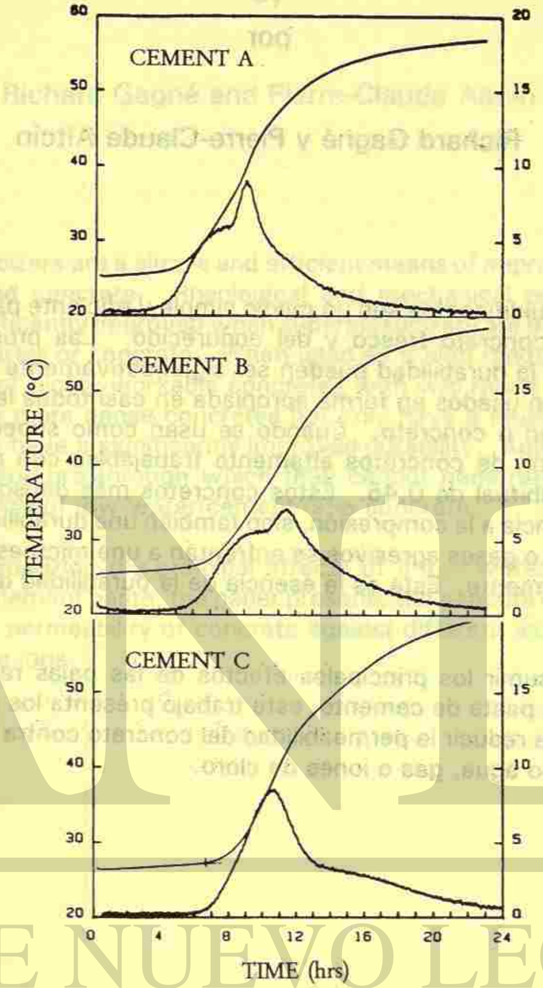

Fig. 6: Thermograms of the cement pastes, W/C = 0.35, containing 0.7% of NaPNS; both the temperature vs time curve and its time derivative (heat flux) are illustrated for each cement.

SUPERFLUIDIFICANTES PARA OBTENER CONCRETO DURABLE

por

Richard Gagné y Pierre-Claude Aïtcin

Sinopsis: Los superfluidificantes son un medio simple y eficiente para mejorar muchas de las propiedades del concreto fresco y del endurecido. Las propiedades reológicas y mecánicas, asi como la durabilidad pueden ser significativamente mejoradas cuando los superfluidificantes son usados en forma apropiada en casi todas las mezclas de mortero fluido, mortero comun o concreto. Cuando se usan como super reductores de agua, permiten la producción de concretos altamente trabajables con relaciones A/C mucho menores del valor habitual de 0.45. Estos concretos más densos no sólo poseen una mucho mayor resistencia a la compresión, sino también una durabilidad significativamente mejorada. Los iones o gases agresivos se enfrentan a una microestructura por la cual no pueden penetrar fácilmente. Esta es la esencia de la durabilidad de concretos con bajas relaciones A/C.

Después de resumir los principales efectos de las bajas relaciones A/C sobre la microestructura de la pasta de cemento, este trabajo presenta los beneficios de usar una baja relación A/C para reducir la permeabilidad del concreto contra los diferentes agentes exteriores, tales como agua, gas o iones de cloro.

Palabras clave: Relación agua-cemento, superfluidificante, durabilidad, permeabilidad al cloro, permeabilidad al aire, permeabilidad al agua, microsílica, durabilidad al congelamiento y descongelamiento, corrosión, carbonatación, ataque por sulfatos, reacción álcali-agregado

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SUPERPLASTICIZERS FOR DURABLE CONCRETE

by

Richard Gagné and Pierre-Claude Aïtcin

Synopsis: Superplasticizers are a simple and efficient means of improving many properties of fresh and hardened concrete. Rheological and mechanical properties, as well as durability, can be significantly improved when superplasticizers are properly used in almost all mixes of grout, mortar or concrete. When used as a high range water reducer, they allow the production of highly workable concretes with w/c ratios well below the usual value of 0.45. These more dense concretes not only have a much higher compressive strength but they also have a significantly improved durability. Aggressive ions or gases face a dense microstructure through which they cannot penetrate easily. This is the essence of the durability of low water/cement ratio concrete.

After briefly summarizing the principal effects of a low water/cement ratio on the microstructure of the cement paste, the paper presents the benefits of using a low water/cement to reduce the permeability of concrete against different exterior agents such as water, gas, or chloride ions.

Keywords: Water-Cement Ratio, Superplasticizer, Durability, Chloride Permeability, Air Permeability, Water permeability, Silica Fume, Freeze-Thaw Durability, Corrosion, Carbonation, Sulfate Attack, Alkali-Aggregate Reactivity.

Richard Gagné is associate professor at the University of Sherbrooke, Sherbrooke, Canada. His research interests are in the areas of durability of concrete and high strength concrete. He is one of the directors of the local chapter of ACI (Eastern Ontario and Quebec) and one of the researchers of the CRIB (University of Sherbrooke and Laval University joint research center on concrete).

Pierre Claude Aïtcin is a professor at the University of Sherbrooke, Sherbooke, Canada. He is currently the director of the Canadian Network of Centers of Excellence on High Strength Concrete. He is a member of ACI Committees 234 and 363, and RILEM TC155 on HSC. He is a Fellow of the American Concrete Institute and of the Canadian Society for Civil Engineering.

INTRODUCTION

Using superplasticizers is a simple and efficient method of improving many characteristics of fresh or hardened concrete. Rheological and mechanical characteristics, as well as durability, can be significantly improved when superplasticizers are properly used in almost all types of cement based mixes (grout, mortar or concrete).

The basic mechanism by which superplasticizers can affect many of the properties of fresh or hardened concrete is related to their deflocculating properties. In pure water, cement particles tend to flocculate because of the attraction forces induced by many unsaturated electric charges at their surface. The large flocs, formed by a group of cement particles, trap water and offer shear resistance during mixing and placing of concrete (Fig. 1a). When a superplasticizer is added to the mixing water, the absorption of polymer chains at the surface of the cement grains creates repulsive forces between particles (1, 2). The cement particles become completely deflocculated and form an almost homogeneous suspension (Fig. 1b). The benefit of using superplasticizers in a cement suspension is thus that, when particles are completely deflocculated, much less water is needed to achieve a given fluidity. From a practical standpoint, this means that by using a superplasticizer the water demand of a given concrete can be greatly reduced (2, 28).

Superplasticizers can therefore be used in two ways (Figure 2). The first way is to use them as a *fluidifying admixture* to increase the workability of concrete without changing the water content or the cement content. Of course, this has been the first use of superplasticizers in concrete. In this case, only the rheological properties are changed and the mechanical properties remain almost unchanged since the water/ cement ratio (w/c) is barely affected (see ***1*** in Fig 2). This way of using superplasticizers can also enhance the durability because a better workability ensures a better compaction of the fresh concrete even in heavily-congested reinforced areas.

The second way in which superplasticizers can be used is as *high range water reducing agents* to lower the w/c or the water content needed to achieve a given workability (see ***2*** on Fig. 2). This second approach is the one that has the most favorable consequences on the general quality of concrete. In that case, the reduction of the w/c will improve not only the mechanical properties but also other properties such as permeability and durability.

Of course, superplasticizers can be used partly as fluidifying agents and partly as high range water reducing agents (see ***3*** on Fig. 2). In that case, both workability and mechanical properties can be enhanced simultaneously.

THE USE OF SUPERPLASTICIZER TO IMPROVE THE QUALITY OF THE CEMENT PASTE

In normal concrete, the dosage of mixing water is mainly governed by workability considerations. In fact, the mass of water used in normal concrete is always greater than the mass of water that is strictly required to fully hydrate all the cement particles. According to Hansen (4) the w/c required for complete hydration of a cement paste continuously stored in water is approximately 0.36 *. For higher w/cs, which are common in normal concrete, the excess water that has not reacted with the cement will remain in the capillary pores. The number and dimensions of these pores affect most of the mechanical properties of hardened concrete as well as its permeability and durability. A simple way to increase the general quality of the cement paste is therefore to minimize this capillary porosity. This goal can be achieved by reducing the w/c ratio of the paste and by using curing conditions that are favorable to the hydration of Portland cement.

Figure 3 presents the theoretical physical composition of the cement paste as a function of w/c. These curves were computed by Hansen (4) using the assumptions proposed by Powers (3) and assuming a maturity factor of 80%, which is common for most concretes used in practice (5). For usual values of w/c (0.4 - 0.6), capillary pores can occupy about 15% to 35% of cement paste volume. The relative proportion of capillaries rapidly decreases with the w/c ratio and becomes negligible for a w/c of 0.30. It must be pointed out that, in practice, some capillary pores are detected even in cement pastes with a w/c as low as 0.25 because larger cement grains are often not fully hydrated owing to the formation of a dense shell of hydration products that prevents the transport of ions from and to the surface of the anhydrous grains (5).

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* Many values ranging from 0.20 to 0.42 can be found in the literature, depending on the basic assumptions and on the curing conditions. From a stoichiometrical point of view, it is generally accepted that the mass of water that can chemically react with 1 g of Portland cement is between 0.21 g and 0.25 g. An interesting discussion on this topic can be found in ref. 5.

The effect of the w/c ratio on the capillary porosity can be seen more precisely in Fig 4, which presents the mercury porosimetry measurements on paste specimens made with a w/c of 0.45 and 0.25. The reduction of the w/c from 0.45 to 0.25 produces two basic changes in the mercury intrusion curves. For the lower w/c, the total porosity, which corresponds to the cumulative porosity measured at the smallest equivalent pore size, is two times lower and the threshold diameter, which is the pore size through which most of the mercury penetrates the bulk of the sample, is about ten times lower. These results imply that low w/c pastes not only have a much lower volume of capillary pores but the average size of these pores is reduced by an order of magnitude.

Cement pastes with fewer and smaller pores have many practical advantages especially with respect to durability (6, 7, 8). A comparison between the physical composition and capillary porosity of two cement pastes made with a w/c of 0.55 and 0.30 is schematized in Fig. 5. The structure of the fresh cement paste is presented on the left hand side and the right hand side presents the structure of the hardened cement paste. It can be seen that in the fresh paste having a w/c of 0.55, cement grains are loosely packed and the gap between them is relatively large. In the hydrated cement paste, the larger voids between cement particles are not completely filled by hydration products and the excess water remains in the large capillary pores. Because of this relatively large amount of excess water, many large pores are very close together and, for the most part, connected to each other. When concrete is exposed to severe environmental conditions, this network of large capillary pores can offer a preferential path to the penetration of different types of potential aggressive agents.

In a low w/c paste (0.30), correctly dispersed with a superplasticizer, the cement grains are more closely packed in the initial suspension. There is much less space between the particles so there is a lower probability of having a large gap between a group of cement grains that would not be completely filled by hydration products. In that type of paste, even a relatively small volume of hydration products can completely fill the initial space between the cement particles. The total volume of the capillary porosity of the hardened cement paste is therefore significantly lower, the average dimension of the pores is smaller and the average distance between the pores is greater. Therefore, low w/c pastes have a more discontinuous network of pores that slow down the migration or penetration of most of the potential external aggressive agents such as liquids, gases or ions.

A lower w/c not only has a beneficial effect on the porosity of the cement paste matrix but also on the porosity of the paste-aggregate interface. In normal concrete, the microstructure of the paste near the aggregates is more porous, more open, and contain a higher proportion of ettringite crystals and massive hexagonal portlandite (9, 10 Generally, hydrates are loosely bonded to the aggregate face and this weak transition zone is less dense and more closely cracked (11). Many aspects of the durability of concrete can be linked to the characteristics of the paste-aggregate interface since this area of open porosity can offer a preferential path for the migration of aggressive agents.

It is now well established that the quality of the paste-aggregate interface is improved by using a lower w/c (1, 12). This is one of the reasons that explains the good mechanical properties of High-Performance Concrete (HPC) (12). In low w/c concrete, the bond between aggregates and cement paste is so strong that it results in a better transfer of stress across the paste aggregates interface. Amorphous-like dense calcium silicate hydrates are intimately bounded to aggregates so that there is no major difference between the porosity near the aggregate and the average porosity of the paste matrix. HPC is really a composite material in which the elastic and mechanical properties of the aggregate play a significant role (13).

It is clear that a low w/c increases the general quality of the paste and concrete. However, in normal concrete, when the w/c ratio becomes relatively low (< 0,40), workability problems arise and it may be difficult to place concrete. In fact, without a superplasticizer, the water content of concrete cannot be reduced very far, since this would results in an unworkable mix. Moreover, the cement content cannot be raised excessively because of the extra cost and the possibility of thermal problems. Superplasticizers can be used to overcome this problem.

SUPERPLASTICIZERS AND PERMEABILITY OF CONCRETE

In the previous sections, it has been shown that when a superplasticizer is used as a water reducing agent to lower the w/c, the resulting concrete is noticeably more dense and has more discontinuous network of pores. These changes in the structure of the cement paste have important consequences because permeability to different types of potential aggressive agents, such as water, gas or chloride-ions, is strongly reduced when the network of pores becomes discontinuous. This lower permeability is the essence of the better durability of low w/c superplasticized concretes.

The different types of aggressive actions that can arise in concrete do not always involve the same type of permeability. The water permeability is a major factor affecting the freezing and thawing resistance of concrete (14, 15). The deicer-salt scaling resistance is mainly governed by the water permeability and chloride-ions in the cement paste (16). Corrosion mechanisms largely involve the water permeability as well as the air and chloride-ions permeabilities (17). The water permeability is of major importance in the sulfates attack mechanisms (17). Efflorescence and carbonation are largely controlled by the availability of CO_2 which itself mainly depends on the air permeability (18).

Water permeability

Water permeability of concrete is closely linked to the characteristics of the pore structure of the cement paste and of the paste aggregate interface. As discussed previously, the more impervious pastes are those with a low total porosity composed of fine capillary pores (low w/c). A general relationship between the w/c and the water permeability of cement based materials is presented in Fig. 6 (19) which shows that the permeability of cement paste, mortar and concrete can be reduced by an order of magnitude, simply by lowering the w/c from 0.70 to 0.50. For a given w/c, mortars and concretes generally have a higher permeability because of the paste aggregate interface, which offers a preferential path for water movements in concretes and mortars.

Superplasticizers can be used to produce low w/c concretes or high-performance concretes, that generally have a very low water permeability. Figure 7, presents the water permeability of a group of superplasticized low w/c concretes made with ordinary or blended Portland cements (silica fume and fly ash). The detailed composition of these concretes can be obtained in the original paper (20). For the lowest water/binder ratios (w/b) (≤ 0.33), the permeabilities reach a very low value ($< 5 \times 10\text{-}14$ m/s) and, from a practical point of view, it is a general agreement that with that level of permeability, concrete can be considered as impervious to water (6). The results of Figure 7 also show that a very low w/b is not the only way to obtain a highly impervious concrete. When superplasticizers are used in combination with a mineral admixture, such as silica fume or fly ash, the permeability of concrete can be low ($< 10 \times 10^{-14}$ m/s) even for a usual w/b of 0.45. In Figure 7, concretes containing fly ash or silica fume have the lowest permeabilities obtained in the range of 35 MPa to 50 MPa.

A concrete with a low water permeability is generally a more durable concrete because many deterioration processes only take place when external water is available in the mass of concrete. For example, the formation of corrosion products around the reinforcing steels depends on the availability of OH ions which are formed by the electrochemical process between water, oxygen and the electrons liberated by the anodic reaction. The better protection against corrosion of concretes with a low water permeability can also be explained by their lower degree of saturation which increases the electrical resistivity of the electrolyte.

Other types of expansive products in cement paste such as silica gel formed by the alkali-aggregate reactions or ettringite formed by the sulfate-related reactions in concrete contain a very large number of water molecules and their rate of formation is a direct function of the availability of external water. Most of the destruction processes associated with freeze-thaw cycles, and particularly the salt scaling resistance, depend on the saturation level of the cement paste. Pastes with a low permeability usually have a lower saturation level near the surface and a better resistance to salt scaling (16,21). The penetration of aggressive ions such Cl is generally much slower in concretes with a low water permeability since these ions mainly penetrate the concrete through the liquid phase.

Air permeability

Figure 8 shows the relation between air permeability of concrete, the w/c and the length of the curing period (22). As for water permeability, air permeability of low w/c concretes is lower because of the finer and more discontinuous network of capillary pores of the hardened cement paste. The positive effect of a low w/c on the air permeability is particularly noticeable during the first 7 days of the initial moist curing. At 1d and 4d the air permeability is reduced by an order of magnitude when the w/c is lowered from 0.70 to 0.40. Obviously, air permeability is also a function of the length of the curing period. Moist conditions reduce air permeability by sustaining the formation of additional hydration products that fill and divide the capillary pores. It is mainly the first 7 days of curing that produce the most important reduction of the air permeability. It should be pointed out that air permeability is far less sensitive to the length of the curing period for low w/c concretes (w/c = 0.40). This has important practical consequences since air permeability of low w/c concretes should be less affected by unfavorable initial curing conditions.

Compressive strength can be a relatively good indicator of air permeability of concrete because both properties are closely linked to the quality of the paste matrix and of the paste-aggregate interface. Figure 9 presents the relationship between oxygen permeability and compressive strength of a fairly wide group of concretes made with or without complementary cementitious materials (silica fume, fly ash) and with w/b ranging from 0.26 to 0.80. The detailed composition of these concretes can be found in the original paper (23). From these results, concretes with a high compressive strength (or a low w/b) clearly have a much lower oxygen permeability. However, permeability becomes less affected and seems to reach a minimum value for w/b lower than approximately 0.30 or when the compressive strength is higher than about 60 MPa.

Air permeability is a major parameter in the durability of reinforced concrete because it controls the availability of both oxygen and CO_2 in the "covercrete" protecting the reinforcing bars. A low air permeability is beneficial because it should reduce the oxygen supply at the cathode where OH ions are produced and that eventually react to form expansive products. A low air permeability will also reduce the availability of CO_2 and the risk of carbonation, which can accelerate the corrosion process by destroying the protective passive oxide film of steel.

Chloride-ion permeability

Chloride permeability is an important property that governs many aspects of the durability of concrete structures (corrosion of reinforcing steel, deicing salt resistance). The AASHTO 277 procedure *Rapid determination of the chloride permeability of concrete* (24) is one of the most commonly used test method to assess the chloride permeability of concrete. The chloride permeability is estimated by measuring the total charge (in coulomb) passing through a concrete specimen (diam. 95 x 50 mm) maintained under a constant electrical tension of 60V during 6 hours. However, this relatively simple test procedure only gives an estimation of chloride permeability by using an indirect measurement mainly based on the conductivity of concrete. Despite this drawback, the AASHTO 277 test method can provide useful information on the protection of concrete against corrosion of reinforcing steel since it gives a relatively good assessment of the mobility of the chloride ions and of the conductivity of the cement paste (25).

As for the other types of concrete permeabilities, the w/c is a key factor controlling the rapid chloride permeability (Figure 10). This type of permeability measurement is particularly sensitive to low values of w/c (26). As shown in Figure 10, the total charge passed through the specimen is relatively less affected for w/c higher that 0.50, but for lower w/c, the total charge drops rapidly and becomes almost negligible for a w/c of approximately of 0.25. Even if it is always a good practice to maintain favorable curing conditions for the longest possible period to reduce the rapid chloride permeability, it must be pointed out that low w/c concretes (< 0.40) are far less affected by short or unfavorable curing conditions (Figure 10).

A general relationship between rapid chloride permeability and compressive strength is shown in Figure 11, which presents some results obtained with a relatively wide family of concretes made with w/b ranging from 0.25 to 0.45 and with various dosages of mineral admixtures (silica fume and fly ash) (20). The rapid chloride permeability of concretes with a compressive strength of less than 50 MPa is somewhat variable, depending on the w/b ratio and on the dosage of silica fume or fly ash. For that range of compressive strengths, concretes with fly ash or silica fume generally have a lower chloride permeability (20). For high-strength (f'c > 50 MPa), or for low w/b ratio concretes (< 0.33), the total charge after 6 h is generally under 1000 coulomb, which is very low according to the reference scale proposed by Whiting (27). From the results of Figure 11, it appears clearly that the use of a superplasticizer to reduce the w/b ratio will not only produce concrete with a higher compressive strength but will also produce concrete with a significantly lower chloride permeability.

While the rapid permeability test is not a direct measurement of the chloride-ion permeability, it can give valuable information on the corrosion protection of concrete, since the result is a direct function of the electrical conductivity of concrete (25). A concrete with a low rapid chloride permeability should offer a better protection against corrosion because of its low electrical conductivity that limits the corrosion current and the mobility of OH and Cl in cement paste.

CONCLUSIONS

Superplasticizers offer a simple and economical way of enhancing the general durability of many types of concrete structures. When used as high range water reducer, they allow the production of highly workable concretes with w/c ratios well below the usual value of 0.45. These more dense concretes not only have a much higher compressive strength but they also have a significantly improved durability. Literature data on to the durability of concrete show that, very often, this is the best and most inexpensive way to protect the concrete against any aggressive agents (6). If the strength of the resultant concrete exceeds the design strength, the designer will have to learn how to take advantage of the enhanced strength provided by these concretes of higher performance.

The essence of the better durability of low w/c superplasticizer concretes is their lower and finer capillary porosity. The more discontinuous network of pores produces a cement paste with a very low permeability against almost all types of potentially external aggressive agents (water, oxygen, CO_2, chloride ions). A quick review of the main destruction mechanisms and of the parameters that control the durability of concrete exposed to a severe environment reveal that a lower permeability generally reduces the extent of the destruction processes.

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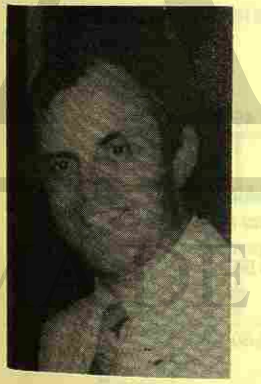
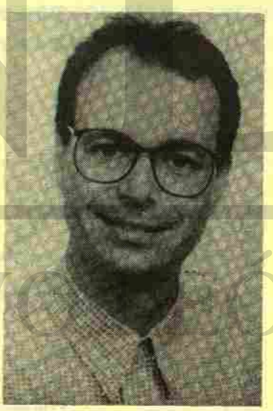


Pierre-Claude Aïtcin, professor
Department of Civil Engineering
University of Sherbrooke, Canada



Richard Gagné, associated professor
Department of Civil Engineering
University of Sherbrooke, Canada

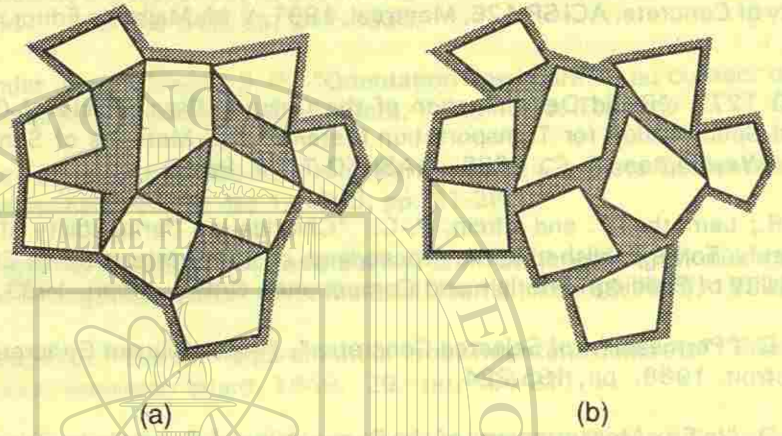


Fig. 1- Schematic representation of a suspension of cement grains in water. (a)-Flocculated state. (b)-Deflocculated state.

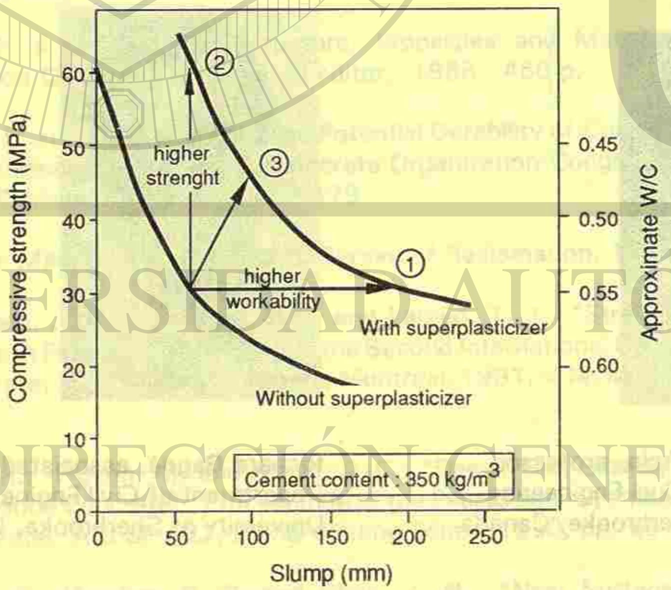


Fig. 2 - Typical effect of a superplasticizer addition on some properties of fresh and hardened concrete.

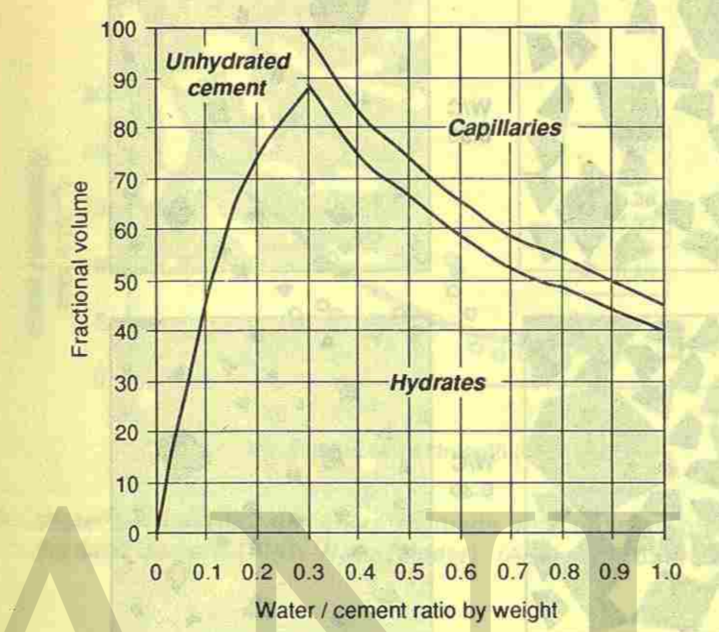


Fig. 3- Theoretical physical composition of a cement paste continuously stored in water (assuming a maturity facto of 8%).

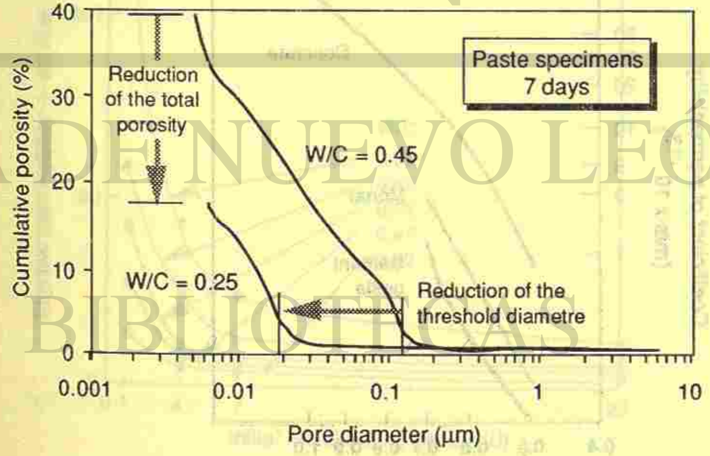


Fig. 4- Mercury intrusion curves of paste specimens after 7 days of curing in water.

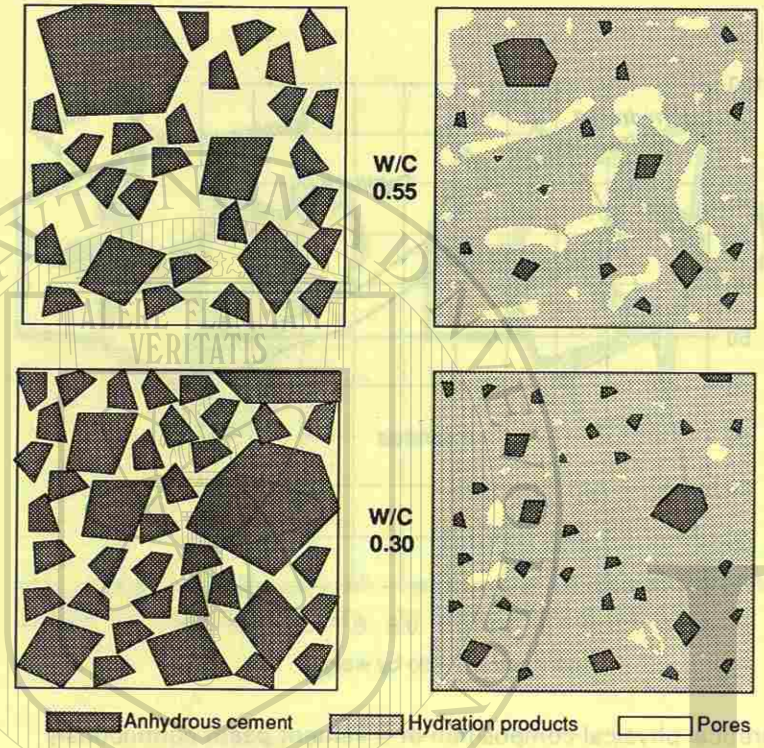


Fig. 5- Schematic representation of the physical composition of two cement pastes with a w/c of 0.55 and 0.30.

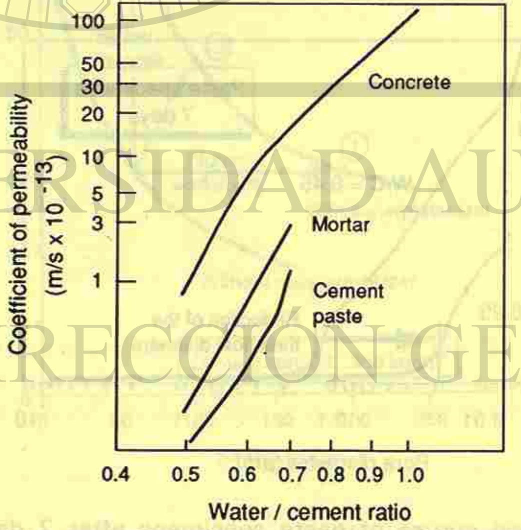


Fig. 6- Water permeability of cement paste, mortar and concrete as a function of w/c. (Adapted from ref. 19).

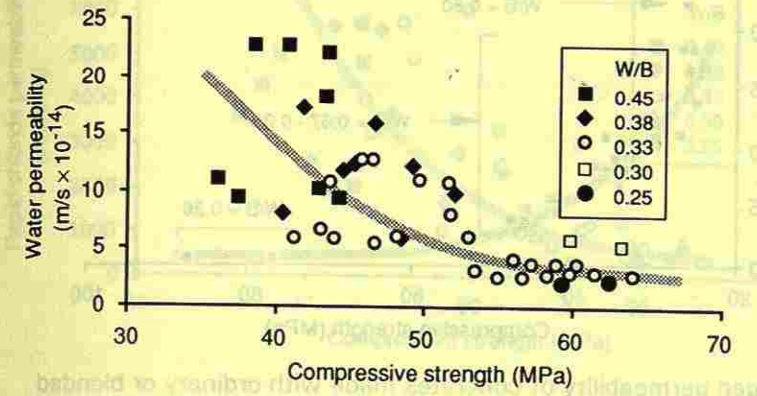


Fig. 7- Water permeability of concretes made with ordinary or blended Portland cement (W/B: Water/Binder). (Adapted from ref. 20).

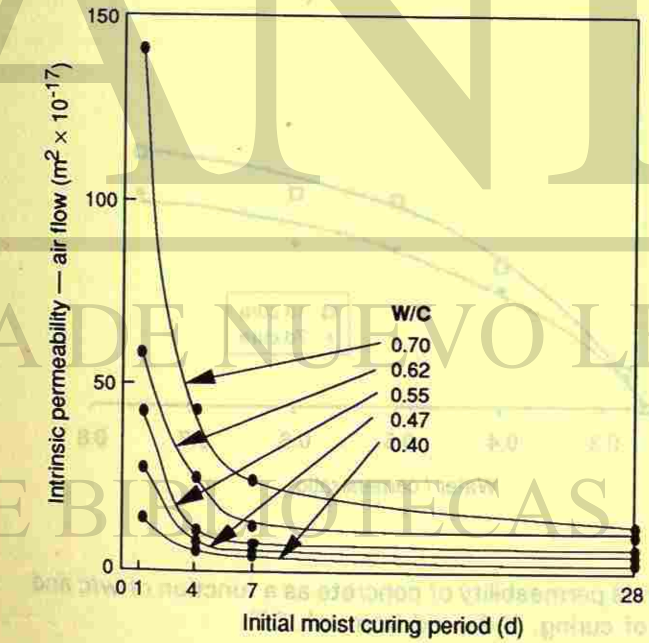


Fig. 8- Intrinsic air permeability of concrete as a function of w/c/ and the length of curing period. (Adapted from ref. 22)

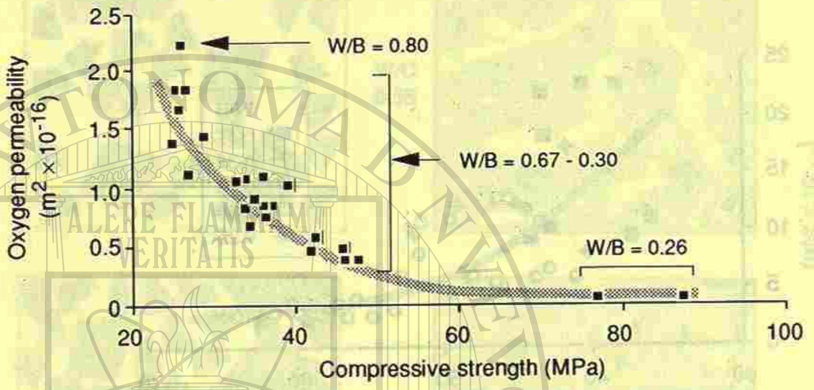


Fig. 9- Oxygen permeability of concretes made with ordinary or blended Portland Cement (W/B: Water/Binder). (Adapted from ref. 23).

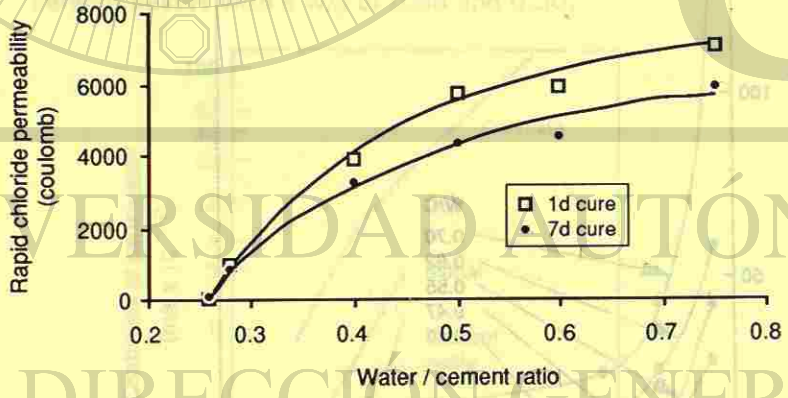


Fig. 10- Rapid chloride permeability of concrete as a function of w/c and the length of curing. (Adapted from ref. 26).

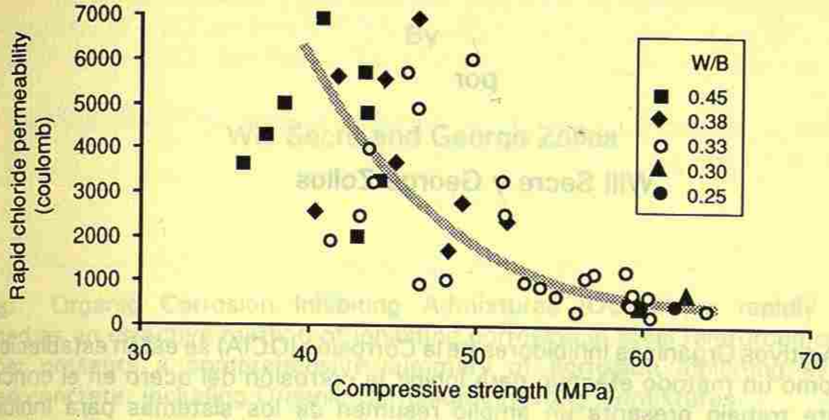


Fig. 11- Rapid chloride permeability of concretes made with ordinary or blended Portland cement (W/B: Water/Blinder). (Adapted from ref. 20).

NUEVAS TENDENCIAS EN EL SISTEMA DE ADITIVOS INHIBIDORES DE LA CORROSION PARA CONCRETO REFORZADO

por

Will Secre y George Zollos

Sinopsis: Los Aditivos Orgánicos Inhibidores de la Corrosión (OCIA) se están estableciendo rápidamente como un método efectivo para inhibir la corrosión del acero en el concreto reforzado. Este trabajo presenta un amplio resumen de los sistemas para inhibir la corrosión del concreto reforzado, incluyendo los Aditivos Orgánicos Inhibidores de la Corrosión.

Uno de tales OCIA funciona por medio de un mecanismo protector doble, el cual inhibe el ingreso de cloruros y humedad en el concreto y forma una delgada barrera protectora en el acero de refuerzo.

El trabajo detalla los procedimientos de ensaye y resultados de una prueba independiente del tiempo necesario para la corrosión llevado a cabo sobre este OCIA, en particular, el cual indica que su comportamiento es igual o mejor que 2, 4 y hasta 6 galones de nitrito de calcio, otro conocido inhibidor de la corrosión.

Palabras clave: Corrosión, inhibidor de la corrosión, sistema inhibidor de la corrosión, evaluación, aditivo orgánico inhibidor de la corrosión, OCIA, ensaye de corrosión a vigas preagrietadas, tiempo necesario para la corrosión, ensaye de corrosión a vigas no agrietadas.

NEW DEVELOPMENTS IN CORROSION INHIBITING ADMIXTURE SYSTEMS FOR REINFORCED CONCRETE

By

Will Secre and George Zollos

Synopsis: Organic Corrosion Inhibiting Admixtures (OCIA) are rapidly becoming established as an effective method of inhibiting corrosion in steel reinforced concrete. The paper presents a comprehensive summary of corrosion inhibiting systems for reinforced concrete, including Organic Corrosion Inhibiting Admixtures.

One such OCIA functions by way of a dual protective mechanism which inhibits the ingress of chlorides and moisture into the concrete, and forms a protective barrier film on the reinforcing steel.

The paper details the test procedures and results of independent time-to-corrosion tests performed on this particular OCIA, which indicate that it performs as well or better than 2, 4 and up to 6 gallons of calcium nitrite, another known corrosion inhibitor.

Keywords: corrosion; corrosion inhibitor; corrosion inhibiting system; Evaluation; organic corrosion inhibiting admixture; OCIA; Precracked Beam Corrosion test; time-to-corrosion; Uncracked Beam Corrosion test;

Will Secre is Marketing Manager for the Admixture Division of Master Builders, Inc. holds an M.S. degree in Chemistry, and has twelve years of experience in mater research and development, and marketing. He is currently a member of ACI 212, Chem Admixtures; NACE Committee T3K, Corrosion of Reinforcing Steel in Portland Ceme Concrete; and NRMCA, Research, Engineering and Standards Committee.

George Zollos is Product Manager for the Admixture Division of Master Builders, Inc. holds a Master's degree in Business Administration.

INTRODUCTION

"Our infrastructure is crumbling!" These words are being heard with increas regularity, and refer to the devastating effects of corrosion on the massive quantities concrete and steel that support our nation's buildings, bridges, and other structu Indeed, corrosion has become the number one enemy in our struggle to maximize useful life of these structures.

The cost of corrosion in terms of loss of use and cost of repairs is staggering. Approximately 42 percent of all bridges in the United States are classified as structu deficient or functionally obsolete by the Federal Highway Administration (1). More t 130,000 of the nations 575,600 bridges restrict the weight of trucks passing over the and about 5,000 have been closed (2).

The cost to repair the bridges alone is estimated at about $90 billion (2). Add to the cost of repairing the multitude of buildings, highways, parking garages, and ot structures damaged by corrosion and the estimate reaches $258 billion, almost 6% of national debt.[1]

In reinforced concrete, corrosion occurs when moisture, oxygen, and chlorides w their way through the concrete cover to react with the reinforcing steel. This reac causes the reinforcing steel to deteriorate, and causes ferric oxide to build up within concrete. As the volume of ferric oxide increases, it creates intense outward pressur the concrete causing it to crack and spall.

Once the reinforcing steel begins to corrode, the process accelerates. The cracking spilling allow even greater quantities of chlorides and moisture to reach the reinfor steel. In addition, the portions of reinforcing steel under the most stress corrode m rapidly, and stress concentrations increase as the reinforcing steel deteriorates. unrepaired, the load bearing capacities of the concrete and reinforcing steel will eventu be reduced, making them susceptible to further damage from the effects of structural and heavy traffic.

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[1] National Association of Corrosion Engineers, 1993.

CORROSION INHIBITING SYSTEMS

Due to the growing costs resulting from corrosion damage, more attention is being paid to the design of reinforced concrete structures exposed to moisture and chlorides, as well as to the materials being used to build them.

ACI 116R-85 defines a corrosion inhibitor as, "a chemical compound, either liquid or powder, that effectively decreases corrosion of steel reinforcement before being embedded in concrete, or in hardened concrete if introduced, usually in very small concentrations, as an admixture."

The National Association of Corrosion Engineers (NACE) further defines corrosion inhibitors as, "a substance which, when added to an environment, decreases the rate of attack by the environment on a metal (3)."

The reinforcing steel in portland cement concrete is protected by a passive iron oxide layer that forms because of the high pH of the concrete. As long as there are no breaks in this passive layer, corrosion will not occur. Thus, most corrosion inhibitors work by reinforcing the passive layer or preventing outside agents from reacting with it.

There are several systems currently used to combat corrosion. These systems work at three different levels: a) at the exposed surface of the concrete by closing the pores and prohibiting the penetration of moisture, oxygen and chlorides into the concrete; b) within the concrete by making the it denser, thus making it more difficult for the moisture and chloride to reach the reinforcing steel; c) at the reinforcing steel level by insulating the reinforcing steel from chlorides and moisture.

Membranes and sealers are surface methods commonly used to combat corrosion. These are applied to the concrete surface to seal the pores and prevent the entrance of moisture and chlorides into the concrete. Membranes, which may be urethanes, neoprenes, or epoxies, are usually built up in multiple layers. Membranes can be extremely effective, but must be protected from being damaged in service, and are typically quite expensive.

Sealers range from linseed oil to silanes and siloxanes. For penetrating sealers to be effective over the life of the structure, they must be reapplied periodically due to their exposure to abrasion and weathering.

Epoxy-coated reinforcing steel bars are a corrosion inhibiting system that works to insulate the reinforcing steel from corrosion causing agents. This system consists of precleaned steel bars protected with a coating of powdered epoxy that is fusion-bonded to the steel in an assembly line process. However, damage to the epoxy coating may occur during handling and placement (especially when the reinforcing steel is bent) resulting in an exposed area on the steel where the chloride ions can initiate the corrosion reaction. The effectiveness of epoxy coated steel is the subject of much debate, and is being increasingly questioned.

There are various methods available that work within the concrete to inhibit corrosion. These include low water/cement ration, increased cement content, no-slump concrete, silica fume, and corrosion inhibiting admixtures. These methods work in distinct ways and provide varying degrees of protection.

Cathodic protection can also be used to inhibit corrosion. However, due to the high cost of this system, it is typically used only in small areas of new construction or for preventing further corrosion of an existing structure in need of repair.

Good concrete practices including a low water-cement ratio, adequate cover over the reinforcing steel, sufficient curing, and the use of post-tensioning to minimize cracking are essential in any effort to minimize or eliminate corrosion. Unfortunately, while good concreting practices are essential, they alone will not prevent corrosion.

No-slump concrete is concrete proportioned with a very low water-cement ratio (usually 0.35 or below, containing about 800 lbs/ yd³ of cement) and is a low cost corrosion inhibition approach. No-slump concrete's effectiveness as a corrosion inhibitor is based upon its relative impermeability, and its performance is fair to good. However, it is extremely difficult to place and has limited applications.

Silica fume is an effective pozzolanic material that reacts with calcium hydroxide in hydrated portland cement to form additional cementitious material. Silica fume particles are many times smaller than those of cement. The silica fume particles fill the spaces between the cement particles making the concrete significantly denser and less permeable. Due to the increased density, special attention must be given to the curing and finishing stages of placement. While the cost of silica fume concrete is in the medium range, its performance is good to excellent in delaying or preventing the corrosion process.

Cathodic protection applies an external source of direct current to the reinforcing steel in the concrete, thus "short circuiting" the corrosion process and controlling corrosion. In cathodic protection, a metallic anode is embedded in the concrete and an electric current is applied. The introduction of the electric current forces the entire structure to become cathodic. Since corrosion takes place only at anodic locations, the corrosion process is stopped. Cathodic protection has high initial costs, but low to medium operating costs. It is viewed as a complicated process because of its preinstallation engineering and post installation monitoring. However, it provides excellent corrosion protection.

Until the introduction of Organic Corrosion Inhibiting Admixtures, calcium nitrite was the only corrosion inhibiting admixture available. Calcium nitrite is an inorganic corrosion inhibitor which reacts with ferrous ions from the reinforcing steel to reinforce the passive layer. In this reaction the nitrite ions compete with any chloride ions present to react with the free ferrous ions.

The relative amounts of chloride and nitrite available at the reinforcing steel determine whether the ferrous ions react with the chlorides or the nitrites. If there are more chlorides present than nitrite, the ferrous ions will react with the chloride and begin the corrosion process. If there are more nitrites than chlorides, the passive layer will be reinforced.

For calcium nitrite to be effective, the dosage of calcium nitrite must be determined based upon the anticipated chloride loading of the structure over its expected design life. Accurately determining lifetime chloride levels is difficult and subject to many variables.

ORGANIC CORROSION INHIBITING ADMIXTURES

Organic Corrosion Inhibiting Admixtures are the most recent development in the battle against corrosion. OCIA's have rapidly become established as an extremely effective and cost efficient method of inhibiting corrosion in steel reinforced concrete.

One such OCIA is a water-based organic admixture consisting of amines and esters. This particular OCIA functions through a dual protective mechanism to inhibit corrosion in steel reinforced concrete structures. The first protective mechanism functions by inhibiting the ingress of chloride ions into the concrete. This "screening" effect inhibits corrosion by reducing the amount of moisture and chlorides that are able to reach and react with the reinforcing steel.

The second mechanism through which this particular OCIA inhibits corrosion is the formation of a barrier film on the reinforcing steel. Components of the OCIA bond to the reinforcing steel by adsorption, and establish a barrier to the electrochemical attack of corrosion causing agents.

EVALUATIONS

The effectiveness of this OCIA, as well as its compatibility in concrete have been proven through independent testing. Accelerated corrosion tests and standard concrete compatibility tests were used in the evaluation.

An independent laboratory was commissioned to thoroughly evaluate the corrosion inhibiting properties of the OCIA. Three time-to-corrosion test procedures were performed; Uncracked Beam, Precracked Beam, and ASTM G-109. Concrete mixes containing one gal/ yd³ of the OCIA were tested against a control mix and mixes containing 2, 4, and 6 gallons/ yd³ of calcium nitrite.

Uncracked Beam Corrosion Tests- Procedure

The Uncracked Beam Corrosion test is an accelerated test that evaluates the corrosion of reinforcing steel in concrete over a 48 week period. The test simulates the exposure of a steel reinforced concrete structure to chlorides, in an accelerated time frame.

Concrete slabs measuring 12 x 12 x 7 in. are cast containing six (6), 1/2 in. (No.4) reinforcing steel bars. Two of the reinforcing bars are placed near the top of the slab with 1 inch of concrete cover. The remaining four bars are placed one inch from the bottom of the test slab. The top bars act as an anode and are where the corrosion takes place.

In order to accelerate the corrosion of the reinforcing steel, chloride ingress is facilitated by the use of moderate to low quality air entrained concrete, a w/c of 0.5, limited curing, and a nominal compressive strength of 5,000 psi at 28 days.

After the slabs are moist cured for 3 days, and air dried for 21, a Lucite dike is attached to the top and the slabs are coated with an epoxy sealer. A 10-ohm resistor is connected between the top and bottom bars.

The corrosion testing begins when the slabs are 28 days old. The slabs are ponded for 4 days with a 15 percent sodium chloride solution at 60 to 80 F. The solution is then vacuumed off and the slabs are air dried at 100 F for three days. This weekly ponding drying cycle continues for 48 weeks.

As the test progresses, moisture and chlorides from the salt solution eventually come in contact with the top reinforcing bars and the corrosion process begins. The corrosion of these bars is monitored by measuring electrical potentials at the top reinforcing bars, and the macro-cell corrosion currents generated between the top (anodic) and bottom (cathodic) reinforcing bars.

Corrosion currents are used to measure of the amount of corrosion occurring on the reinforcing steel. The greater the current, the greater the amount of corrosion that is occurring.

Electrical potentials are another measure of corrosion activity. Researchers for the Federal Highway Administration have determined that, "...corrosion activity starts at a half-cell potential of about -0.24 volts... Active corrosion occurred at half-cell potentials as low as -0.20 to -0.25 volts and that significant corrosion current developed in the "uncertain range," i.e. -0.25 to -0.35 volts, as discussed in ASTM C876 (6)."

In addition to measuring corrosion current and half-cell potentials, the chloride content of each slab is measured at the upper reinforcing steel level when corrosion is first initiated, and again after the 48 week test is completed.

Uncracked Beam Corrosion Tests- Results

Results from the Uncracked Beam Corrosion tests show that the OCIA most effectively inhibited corrosion as compared to the control mixes and those mixes containing up to 6 gal/ yd³ of calcium nitrite. The results of the Uncracked Beam Corrosion tests are shown in Table 1.

Figure 1 shows that on average specimens containing the OCIA were the last of the mixes to initiate corrosion (at about 19 weeks). The macro cell corrosion current was also the lowest in specimens containing the OCIA. This indicates that the OCIA effectively reduced the rate of corrosion once it was initiated.

The test specimens of control concrete as well as those containing calcium nitrite at 2 gal/ yd³ initiated corrosion between 7 and 12 weeks. Specimens containing 4 gal/ yd³ of calcium nitrite began corroding at 8 and 26 weeks. One slab containing calcium nitrite at 6 gal/ yd³ initiated corrosion in the 19th week of testing. In, one specimen containing the OCIA at 1 gal/ yd³, corrosion was not initiated until the 22nd week of the test. By the end of the 48 week test period, the companion slabs for the OCIA at 1 gal/ yd³ and calcium nitrite at 6 gal/ yd³ had not yet started to corrode.

The corrosion current generated by each specimen was monitored throughout the test. The average corrosion current of the control mixes and those containing calcium nitrite at 2 gal/ yd³ exceeded that of the calcium nitrite at 4 gal/ yd³ at approximately 10 weeks. The corrosion current generated by the single specimens treated with the OCIA at 1 gal/ yd³ and calcium nitrite at 6 gal/ yd³ remained very low throughout the test, and was significantly lower than the other specimens.

At the end of the 48 week test period, each specimen was vertically sawcut approximately 1 in. from the edge of the top bars to identify any corrosion induced delaminations. The control specimens showed delaminations 9 and 4 in. long. Slabs containing 2 gal/ yd³ of calcium nitrite both contained delaminations. However, one of the specimens was damaged before the delamination could be measured. The delamination on the other specimen was 4 in. long. Of the slabs containing 4 gal/ yd³, one contained a delamination of 6 in., while the other showed none. Finally, none of the OCIA slabs, nor the calcium nitrite slabs at 6 gal/ yd³ contained any delaminations.

After the initial vertical sawcut, the top concrete cover was removed to reveal the length of any corrosion that had taken place, and the overall condition of the reinforcing steel. The two top pieces of reinforcing steel were examined from each specimen, equating to four pieces of reinforcing steel per mix. The average corroded length of the four pieces of reinforcing steel for each mix is shown below.

Mix	Avg. corrosion length	Severity
1 gal OCIA	1.1 in.	Very light to minor
Control	5.6 in.	Severe
2 gal calcium nitrite	6.1 in.	Moderate to severe
4 gal calcium nitrite	2.1 in.	Very light to severe
6 gal calcium nitrite	1.1 in.	Very light to moderate

The reinforcing steel removed from the concrete specimens containing 1 gal/ yd³ of the OCIA were shown to contain only very light to minor scale- Evidence that only minor corrosion activity had occurred. In the control specimen, severe pitting was evidenced in all of the reinforcing steel. The length of corrosion in the control mix ranged from 3.25 in. to 9 in.. Moderate to severe pitting was also evident in all of the reinforcing steel taken from the specimens containing calcium nitrite at 2 gal/ yd³. The length of corrosion from these specimens ranged from 4 to 7.25 in.

In the slabs containing 4 gal/ yd³ of calcium nitrite, the reinforcing steel showed moderate to severe pitting in two pieces of reinforcing steel, and minor to light scale in the remaining pieces. Moderate pitting was observed in one reinforcing steel specimen from slabs containing calcium nitrite at 6 gal/ yd³ A second specimen contained moderate to severe scale, while the remaining two pieces of reinforcing steel were determined to have very light to minor scale. The length of corrosion and overall reinforcing steel condition are summarized in Table 2.

Precracked Beam Tests- Procedure

The precracked beam corrosion test measures the effectiveness of corrosion inhibitors in cracked concrete. While the specimens for this test were cast using the same concrete mix as was used in the standard Uncracked Beam Corrosion test, the Precracked Beam Corrosion test is more severe since chlorides and moisture have direct access to the reinforcing steel.

In this test method, 4x5x16 in. specimens were cast with one No.3 reinforcing steel bar placed near the top the specimen, and two No. 3 reinforcing bars placed near the bottom surface. The cover for the top and bottom reinforcing steel is 1 in.

The test slabs were moist cured for 7 days, then air dried for 14 days. The specimens were then stressed under center point flexure to induce a stress crack with a width at the top of the specimen of approximately 0.010 in. Steel plates were attached to the sides of the slabs to prevent the crack from closing. After the cracking procedure had been completed, the final surface crack width was verified to be between 0.008 to 0.010 in.

Once again, Lucite dikes were attached to the top of the test specimens, and a 10-ohm resistor was connected between the top and bottom reinforcing steel.

Corrosion testing began when the slabs were 28 days old. The specimens were ponded for 4 days with a 6 percent sodium chloride/ water solution at 60 to 80 F.
The solution was then vacuumed off, and the slabs dried for 3 days at 60 to 80 F.
This cycle continued for 23 weeks.

Precracked Beam Tests- Results

At the conclusion of the test period, only one specimen out of six containing the OCIA at 1 gal/ yd³ exceeded 100 uA of current. All of the specimens of the control mix, and of

calcium nitrite at 2 gal/ yd³ and 6 gal/ yd³ showed corrosion current in excess of 100 uA. Four of the six reinforcing steels from the specimens containing calcium nitrite at 4 gal/ yd³ exceeded 100 uA.

In analyzing the test results, the control specimens and specimens containing 2 gal/ yd³ of calcium nitrite had the highest average corrosion currents. They also had the most negative half-cell potentials. The test slabs containing the OCIA at 1 gal/ yd³ took the longest to initiate corrosion, and had the lowest corrosion currents during the test period. Corrosion currents from the specimens containing 4 and 6 gal/ yd³ of calcium nitrite were less than the control mix, but greater than those containing the OCIA. The average corrosion current in the specimens containing calcium nitrite at 6 gal/ yd³ increased significantly after 20 weeks, and was equivalent to those currents from the specimens containing 2 gal/ yd³ of calcium nitrite.

Results from the half-cell potential readings were similar. Specimens containing the OCIA and calcium nitrite at 6 gal/ yd³ had similar readings which were less negative than any of the other mixes. The control mix and the mix containing 2 gal/ yd³ of calcium nitrite had the most negative half-cell potential readings.

At the conclusion of the 23 week test period, the test slabs were sawcut and the reinforcing steel removed. Most all of the bars had corrosion close to the crack, and several had pitting at the point where the crack met the reinforcing steel. The length of corrosion was then measured, and the average corrosion length and the severity are shown below.

Mix	Avg. corrosion length	Severity
1 gal OCIA	2.8 in.	No corrosion to severe
Control	8.1 in.	Severe
2 gal calcium nitrite	11.3 in.	Severe
4 gal calcium nitrite	8.5 in.	Severe
6 gal calcium nitrite	6.8 in.	Moderate severe to very severe

The reinforcing steel removed from the concrete specimens containing 1 gal/ yd³ of the OCIA showed an average corrosion length approximately 65% less than the control specimens, and over 55% less than the specimens containing calcium nitrite at 6 gal/ yd³. The length of corrosion in the OCIA treated specimens ranged from 0.0 to 7.5 in..

In the control specimen, the length of corrosion ranged from 5 to 11 in.. Corrosion was severe on all specimens. Severe corrosion was also evident in all of the reinforcing steel taken from the specimens containing calcium nitrite at 2 gal/ yd³ The length of corrosion from these specimens ranged from 5.0 to 15.0 in.. In the slabs containing 4 gal/ yd³ of calcium nitrite, the reinforcing steel showed moderate severe to severe corrosion, and ranged in length from 2.0 to 15 in.. Moderate severe to very severe corrosion was observed in reinforcing steel specimens from slabs containing calcium nitrite at 6 gal/ yd³ The length of corrosion was from 3.0 to 4.25 in.. The length of corrosion and the overall severity of corrosion are summarized in Table 3.

A chloride analysis was performed on a total of nine test specimens (three each from the control mix, calcium nitrite at 4 gal/ yd³ and the OCIA at 1 gal/ yd³ The chloride analysis shows that chloride levels were highest in the specimens containing 4 gal/yd³ of calcium nitrite, and lowest in specimens containing the OCIA. The OCIA was found to, "significantly hinder the infiltration of chloride into the concrete and along the bar (7)."

ASTM G-109 Test

The ASTM G-109 test procedure is similar to the Time to Corrosion test described previously. The major differences in the G-109 test procedure is the use of smaller specimens (11 x 6 x 4.5 in.), a higher cement content (600 lbs), a 3% salt solution, and a monthly drying cycle at a temperature of 73 F.

The ASTM G-109 test was originally designed to evaluate the effectiveness of concrete admixtures in inhibiting corrosion. However, because of significant variability in test data from different labs, the test method is currently being reviewed.

Prior to these recent developments, the OCIA was tested by Wiss, Janney, Elstner Associates, Inc., a leading independent corrosion laboratory. More than a year after the test had started, none of the specimens (the OCIA nor the control mix) had initiated corrosion.

Effect of The OCIA on Plastic Properties

The OCIA has very little effect on the plastic properties of concrete. In the following sections, test data obtained from a field evaluation are presented on set time, slump, ai content, and temperature development of concrete containing the OCIA and those of a reference concrete. The mixture proportions used in this evaluation are presented in the following table. The OCIA was used at a dosage which is consistent with that recommended for corrosion inhibition.

Concrete Mixture Data (field evaluation)

		Reference	OCIA
Cement	(lbs/ yd³)	650	650
Fine aggregate	"	1249	1249
Coarse Aggregate	"	1800	1800
Water	"	260	260
Air Entrainer	(fl. oz/cwt)	1.4	1.4
HRWRA		10.0	12.0
Organic Corrosion Inhibitor		-----	.20
(% by wt. of cement)*			.40
w/c ratio		.40	6.5
Slump (in.)		7.0	5.7
Air (%)		6.2	147.5
Unit Weight	(lb./cu. ft)	145.1	72
Concrete Temperature (°F)		73	

* 1 oz./cwt. is equivalent to 0.01% by wt. of cement

Set Time

The effect of The OCIA on the setting time of concrete was evaluated at ambient temperatures of 50, 72, and 90 °F. Both initial and final set times were obtained at each temperature. The test data are shown graphically in Figures 2 and 3. The data indicates that The OCIA did not have a significant effect on initial set time at ambient temperatures of 72 and 90 degrees Fahrenheit. At an ambient temperature of 50 degrees Fahrenheit, The OCIA retarded the concrete slightly. However, the amount of retardation is less than that specified in ASTM C-494 for classification as a Type B, Retarding admixture.

Slump

The OCIA has no effect on concrete slump as shown in Figure 4.

Air Content

The addition of OCIA to a concrete mixture may require extended mixing or an increase in the amount of air-entraining admixture used, to achieve a given air content.

Temperature Development Profile

As shown in Figure 5, the OCIA will not affect the peak exotherm of temperature development profile of concrete. The data shown in this figure were obtained from thermocouples placed in the middle of insulated 1.0 cu. ft. wooden boxes filled wit concrete. The boxes were insulated to simulate a mass concrete placement. Th temperature development profiles are consistent with the set time data discussed above

EFFECT OF OCIA ON HARDENED PROPERTIES

Air-void System Parameters

The effect of the OCIA on the air-void system parameters of concrete were extensive evaluated using truck-mixed and central-mixed concretes. Air-entraining admixtures wer used in these evaluations at dosages ranging from 1.5 to 3.0 fl. oz/cwt The results of th air-void system analysis are summarized in the Table 4 for the truck-mixed and centra mixed concretes. The data show excellent correlation between the plastic and hardene air contents, indicating that the OCIA will not affect the stability of the air voids durin the hardening process. The calculated spacing factors are less than or equal to th maximum value of 0.008 in. recommended by ACI or adequate freeze-thaw durability o concrete.

Abrasion Resistance

The effect of the OCIA on the abrasion resistance of concrete was determined i accordance with ASTM C 779, Procedure A. The test data, shown in Figure 5 indica equal depths of wear for the reference and the OCIA treated concretes after 60 minute of exposure to abrasion. This implies that the OCIA will not affect the resistance o concrete subjected to severe abrasion.

Freezing and Thawing Resistance

The effect of the OCIA on the freezing and thawing resistance of concrete wa evaluated in accordance with ASTM C 666, Procedure A. After 300 freezing and thawi cycles, a durability factor of 96 percent was obtained for the concrete containing the OC relative to the reference. This excellent relative durability factor indicates that the OC will not affect the freezing and thawing resistance of concrete.

Compressive Strength

When used in combination with certain types of natural, rounded aggregate, th addition of The OCIA may result in a decrease of compressive strength. By reducing th w/c ratio, the compressive strength can be returned to the original level.

Sulfate Resistance

The OCIA will increase the sulfate resistance of concrete. Data obtained from an accelerated sulfate resistance test developed by the Bureau of Reclamation Figure 7 show that the expansion of test specimens for concrete containing the OCIA is only slightly less than that for the reference concrete, after 15 weeks of testing. However, after 48 weeks of testing, the OCIA-treated concrete showed significantly less expansion compared to the reference concrete. This can be attributed to a slower ingress of the sulfate solution into the OCIA-treated concrete matrix.

Concrete-Steel Bond Strength

The bond strength developed between concrete and steel is not affected by the OCIA. Test data obtained from an independent evaluation conducted at the university of Kansas and presented in Figure 8 show that, on the basis of equal compressive strength, there is no difference in bond strength between the reference and treated concretes, even at a dosage rate which is twice that recommended for corrosion inhibition.

Modulus of Elasticity

The effect of the OCIA on the modulus of elasticity of concrete was evaluated in accordance with ASTM C 469 using a computerized data acquisition system and a pair of extensometers. This test setup enabled the determination of the strain at peak load and the toughness of the concrete. The data obtained are presented in Figures 10 and 11. As shown in Figure 9, the modulus of elasticity of the concrete mixture treated with the OCIA was marginally higher than that of the reference mixture. The slight increase in elastic Modulus is, however, considered insignificant. Figures 10 and 11 indicate that strain at peak load and toughness of concrete are not affected by the use of the OCIA.

CONCLUSION

Organic Corrosion Inhibiting Admixtures (OCIA's) represent the latest generation in corrosion inhibitors. Building on existing technologies, organic corrosion inhibitors offer an effective, cost efficient method of controlling corrosion in steel reinforced concrete.

Results of extensive independent testing have shown this particular OCIA to be an extremely effective corrosion inhibiting system, particularly in delaying the onset of corrosion in cracked concrete where the aggressive agents have direct access to the reinforcing steel.

In evaluating the performance of this particular OCIA in uncracked and precracked time-to-corrosion tests, Wiss, Janney, Elstner Associates, Inc. concluded, "Concrete containing OCIA at 1 gal/ yd³, performed as well or better than concrete containing 2, 4, and 6 gal of (calcium nitrite) in standard uncracked time-to-corrosion slab tests and precracked be tests (7)."

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Will Secre, Marketing Manag for the Admixtures Division o Master Builders, Inc.

TABLE 1. Uncracked Beam Corrosion Test

Sample		Corrosion Current (uA)			Resistance (Ohms)		Half-cell Potential (-v)		
No.	Description	1 wk	23 wk	48 wk	1 wk	48 wk	1 wk	23 wk	48 wk
1A	Control	0	147	174	180	510	0.192	0.536	0.589
1B	Control	0	179	164	180	610	0.186	0.511	0.553
2A	2 gal CNI*	0	82	54	140	520	0.190	0.483	0.459
2B	2 gal CNI	0	56	253	150	420	0.176	0.400	0.589
3A	4 gal CNI	0	63	160	150	460	0.166	0.438	0.555
3B	4 gal CNI	0	1	6	150	530	0.186	0.180	0.308
4A	6 gal CNI	0	36	34	100	350	0.189	0.398	0.423
4B	6 gal CNI	0	0	0	120	460	0.194	0.219	0.165
5A	1 gal OCIA	0	9	6	200	930	0.187	0.314	0.330
5B	1 gal OCIA	0	0	0	200	840	0.199	0.161	0.163

* Calcium nitrite

TABLE 2. Uncracked Beam Corrosion Test- Rebar Condition

Sample No.	Length of Corrosion (in.)	Top/Bottom/ Circumference	Description
Control			
1A	9.0	T	Moderate to severe-deep pitting and flaking.
1A	5.0	T	Moderate to severe- deep pitting and flaking.
1B	3.25	T	Severe pitting
1B	5.0	T	Very severe, loss of rebar deformation
Calcium nitrite- 2 Gal/ yd³			
2A	4.0	T	Very severe pitting
2A	7.25	T	Moderate to severe
2B	6.25	T some on bottom	Moderate to severe
2B	6.75	T some on bottom	Moderate to severe
Calcium nitrite- 4 Gal/ yd³			
3A	4.75	T	Moderate to severe and very severe
3A	3.5	T	Moderate to severe
3B	0.25	T	Minor scale
3B	0.0	--	Very light scale
Calcium nitrite- 6 Gal/ yd³			
4A	2.5	T	Moderate to severe scale
4A	1.5	T	Moderate pitting
4B	0.25	T	Minor scale
4B	0.0	--	Very light scale
OCIA- 1 Gal/ yd³			
5A	4.0	T	Minor scale
5A	0.0	--	Very light scale
5B	0.0	--	Surface scale
5B	0.25	T	Very light scale

TABLE 3. Condition of Precracked Beam Specimens (Conclusion of Testing)

Specimen No.	Top Width of crack (in.)	Depth of crack (in.)	Maximum length of corrosion (in.)	Severity of corrosion	Location of corrosion (sides)
Control					
1A	0.005	3.50	7.00	Severe	Both
1B	0.009	3.00	8.00	Severe	Both
1C	0.009	2.75	8.00	Severe	Both
1D	0.009	3.50	10.00	Severe	Both
1E	0.007	3.25	11.00	Severe	Both
1F	0.009	3.50	9.00	Severe	Mostly one
1G	0.009	3.25	7.00	Severe	Both
1H	0.007	3.25	5.00	Severe	Mostly one
Calcium nitrite- 2 Gal/ yd³					
2A	0.010	2.75	14.00	Severe	Both
2B	0.005	3.00	14.00	Severe	Both
2C	0.005	3.25	14.50	Severe	Both
2D	0.007	3.00	8.00	Severe	One
2E	0.007	3.50	15.00	Severe	One
2F	0.009	3.50	15.00	Severe	One
2G	0.007	3.00	5.00	Severe	Mostly one
2H	0.005	3.25	5.00	Severe	Both
Calcium nitrite- 4 Gal/ yd³					
3A	0.010	3.25	14.00	Severe	Both
3B	0.011	3.50	14.50	Severe	One
3C	0.007	3.25	15.00	Severe	One
3D	0.009	3.00	11.00	Severe	One
3E	0.007	3.00	5.00	Severe	One
3F	0.007	3.75	3.50	Moderate severe	One
3G	0.007	3.50	3.00	Severe	Mostly one
3H	0.005	2.75	2.00	Severe	One
Calcium nitrite- 6 Gal/ yd³					
4A	0.007	3.00	11.00	Severe	Mostly one
4B	0.009	2.50	7.00	Severe	Both
4C	0.010	3.25	7.00	Very severe	Mostly one
4D	0.005	2.50	9.00	Severe	One
4E	0.005	3.00	4.00	Severe	One
4F	0.007	3.50	3.00	Severe	One

TABLE 3. (Continued)

Specimen No.	Top Width of crack (in.)	Depth of crack (in.)	Maximum length of corrosion (in.)	Severity of corrosion	Location corrosi (sides
			OCIA- 1 Gal/ yd^3		
5A	0.009	3.50	1.50	Severe	Both
5B	0.009	3.75	7.50	Light severe	Mostly
5C	0.070	3.75	2.50	Moderate severe	Both
5D	0.013	4.25	6.00	Severe	Both
5E	0.007	3.75	1.50	Light severe	Both
5F	0.009	3.00	2.00	--	Both
5G	0.007	3.00	None	None	None
5H	0.007	3.25	1.75	Severe	Both

TABLE 4. Air Void System Analysis Data

Evaluation No.	AEA */Dosage (0z./cwt.)	Air Content		Specific Surface (in. -1)	Spacing Factor
		Fresh	Hardened		
Truck-Mixed					
1	AEA/3.5	8.1	8.23	741	0.00
2	AEA/3.5	11.3	13.70	571	0.00
3	AEA/1.5	9.0	5.95	667	0.00
4	AEA/1.5	7.5	7.12	645	0.00
5**	AEA/2.0	7.9	7.14	678	0.00
6**	AEA/2.0	6.8	6.12	571	0.00
7	AEA/1.5	5.7	5.54	571	0.00
Central-Mixed					
8	AEA/2.25	7.7	8.70	615	0.00
9	AEA/2.25	7.7	6.92	615	0.00
10	AEA/2.25	7.1	7.23	727	0.00
11**	AEA/3.0	6.8	7.89	548	0.00
12**	AEA/3.0	6.8	6.59	615	0.00
13	AEA/2.25	5.3	5.76	625	0.00
14**	AEA/2.5	5.7	5.70	769	0.00
15	AEA/2.0	7.8	8.94	580	0.00

* Air-Entraining Admixture

** Silica fume concrete mixture; 8% silica fume addition by weight of cement.

Note: All concrete mixtures contained 650 lbs./yd^3 of cement, The OCIA at .020% by weight of cement, and Superplasticizer high-range water reducing admixture, for workability.

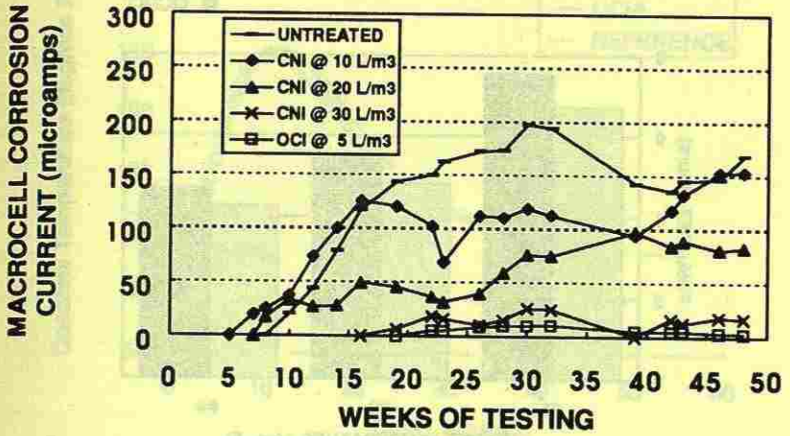

FIGURE 1- Time To Corrosion Test Results

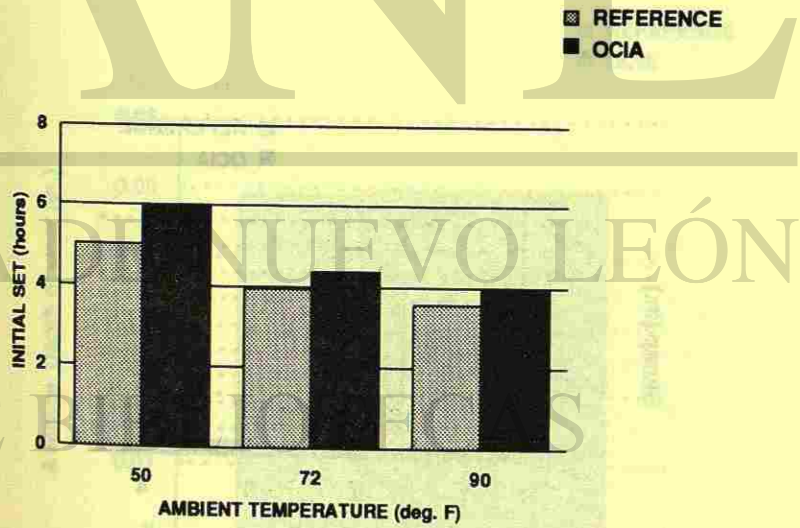

FIGURE 2- Effect Of OCIA On Initial Set Time

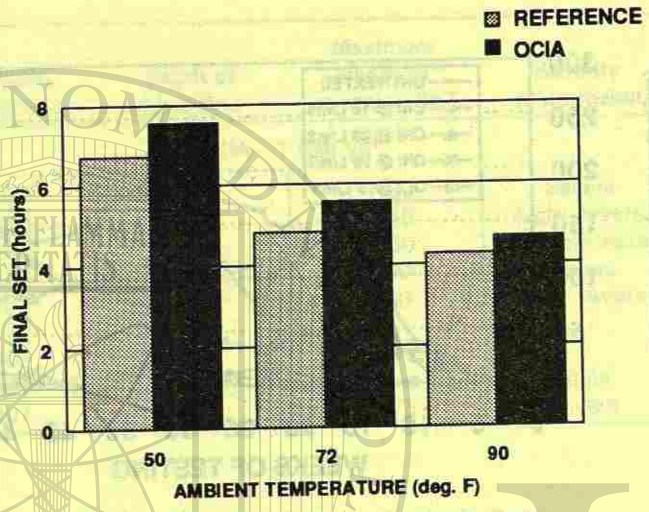

FIGURE 3- Effect Of OCIA On Final Set

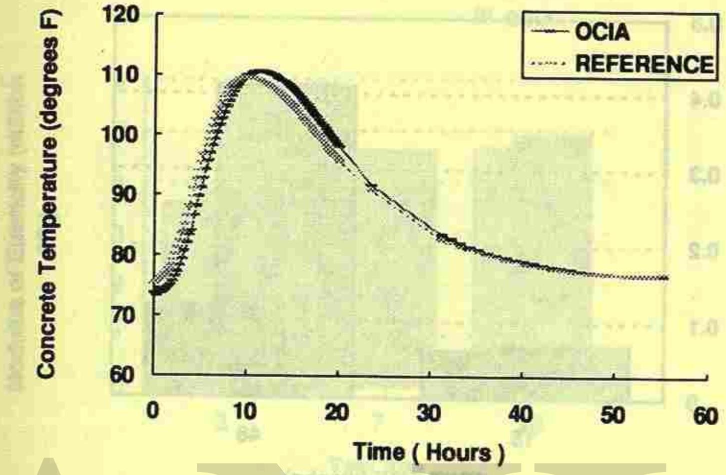

FIGURE 5- Effect of OCIA On The Temperature Of Concrete

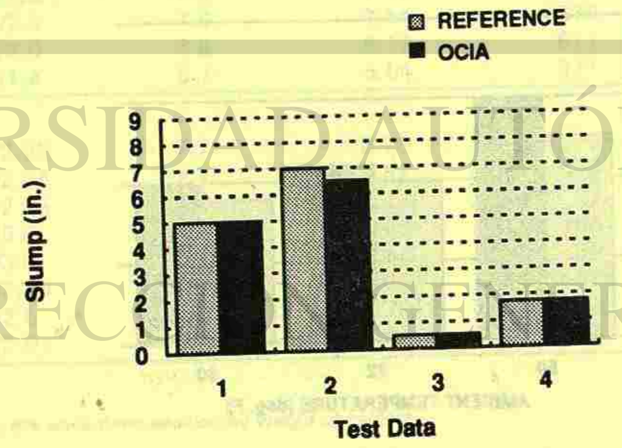

FIGURE 4- Effect Of OCIA On Slump

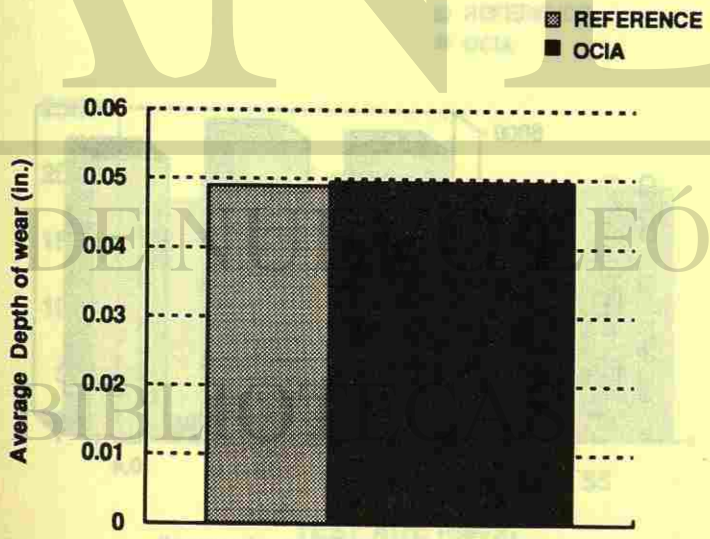

FIGURE 6- OCIA Abrasion Resistance

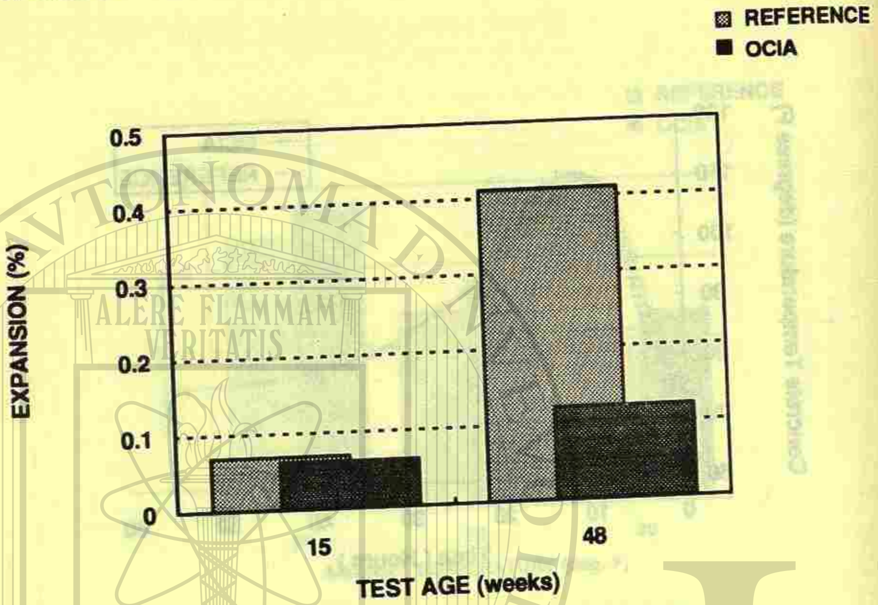

FIGURE 7- OCIA Sulfate Resistance

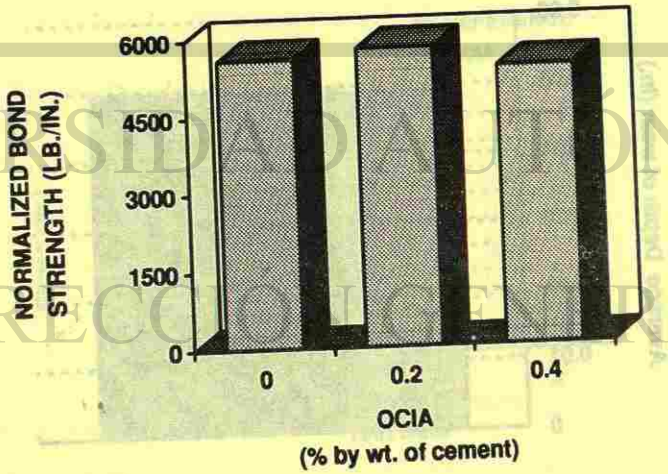

FIGURE 8- OCIA Effect On Concrete/Steel Bond Strength

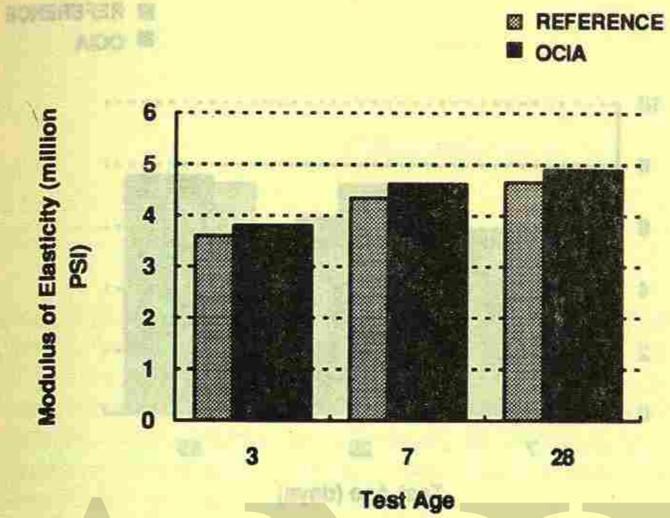

FIGURE 9- OCIA Effect On The Modulus Of Elasticity Of Concrete

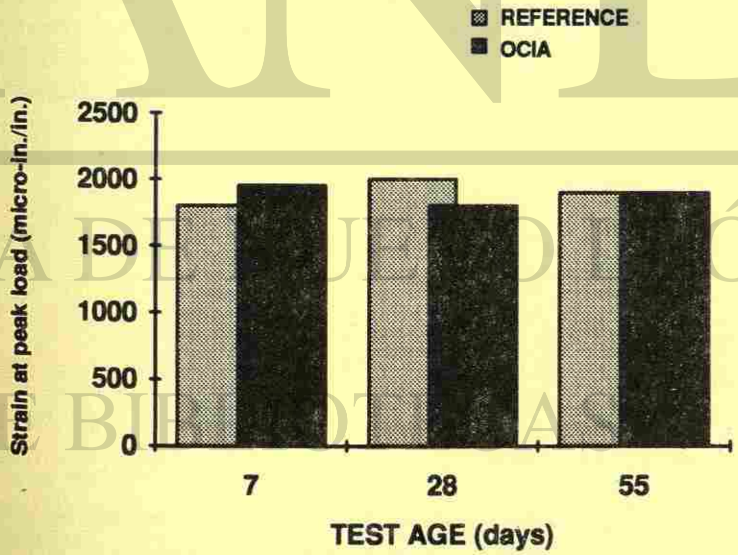

FIGURE 10- OCIA Effect On Strength At Peak Load

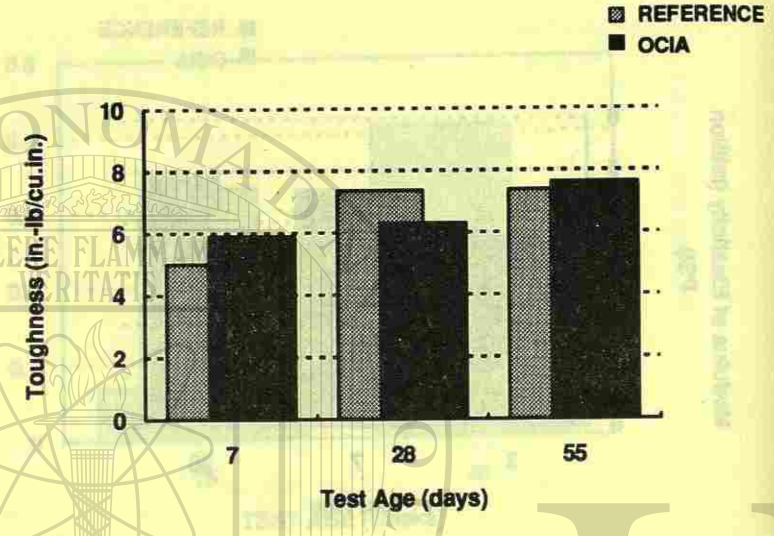

FIGURE 11- Effect of OCIA On Toughness Of Concrete

CONCRETO CON ALTO CONTENIDO DE CENIZA VOLANTE: PROPIEDADES MECÁNICAS Y ASPECTOS SOBRE LA DURABILIDAD

por

V.M. Malhotra

Sinopsis. El Centro Canadiense para la Tecnología de Minerales y Energía (CANMET) posee un programa de investigación continua que está enfocado al uso de materiales cementantes suplementarios en el concreto. Como parte de este programa CANMET ha desarrollado concretos en los cuales se incorporan altos volúmenes de ceniza volante de bajo contenido de calcio y se obtienen revenimientos mayores de 150 mm mediante el uso de grandes dosis de superfluidificantes. Comúnmente, en concretos con alto contenido de ceniza volante, el contenido de cemento se mantiene alrededor de 150 kg/m³, la relación agua-materiales cementantes es de aproximadamente 0.30 y el contenido de ceniza volante es de entre 55 y 60 por ciento en peso del total del material cementante.

Este trabajo discute las propiedades, aplicaciones y limitaciones de los concretos con alto contenido de ceniza volante. Las propiedades discutidas incluyen los proporcionamientos de las mezclas, propiedades mecánicas, diversos aspectos de durabilidad y la permeabilidad. Se concluye que el concreto con alto contenido de ceniza volante posee un excelente desarrollo de resistencia y características de durabilidad, y exhibe muy poco aumento de temperatura. Estas propiedades lo hacen ideal para usarse en grandes losas de cimentación, elementos estructurales gruesos y otras aplicaciones similares.

Palabras Clave: Concreto, durabilidad, ceniza volante, calor de hidratación, mecanismo de hidratación, permeabilidad, resistencia, superfluidificantes, propiedades térmicas.

CONCRETO CON ALTO CONTENIDO DE CENIZA VOLANTE: PROPIEDADES MECÁNICAS Y ASPECTOS SOBRE LA DURABILIDAD

por

V.M. Malhotra

Sinopsis: El Centro Canadience para la Tecnología de Minerales y Energía (CANMET) posee un programa de investigación continua que está enfocado el uso de materiales cementantes suplementarios en el concreto. Como parte de este programa CANMET ha desarrollado concretos en los cuales se incorporan altos volúmenes de ceniza volante con bajo contenido de calcio y se obtienen revenimientos mayores de 150 mm utilizando grandes dósis de superfluidificantes. Comunmente, en concretos con alto contenido de ceniza volante, el contenido de cemento se mantiene en cerca de 150 Kg/m³, la relación agua-materiales cementantes es de aproximadamente 0.30 y el contenido de ceniza volante es de entre 56 y 60 por ciento en peso del total del material cementante.

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Palabras Clave: Concreto, durabilidad, ceniza volante, calor de hidratación, mecanismos de hidratación, permeabilidad, resistencia, superfluidificantes, propiedades térmicas.

HIGH-VOLUME FLY ASH CONCRETE: MECHANICAL PROPERTIES AND DURABILITY ASPECTS

by

V.M. Malhotra

Synopsis: The Canada Centre for Mineral and Energy Technology (CANMET) has an ongoing research program dealing with the use of supplementary cementing materials in concrete. As a part of this program, CANMET has developed concretes in which high volumes of low-calcium fly ash are incorporated and slumps in excess of 150 mm are obtained by the use of large dosages of superplasticizers. Typically, in high-volume fly ash concrete, cement content is kept at about 150 kg/m³, the water-to-cementitious materials ratio is about 0.30, and fly ash content is between 56 and 60 per cent by weight of the total cementitious material.

This paper discusses the properties, applications and limitations of the high-volume fly ash concrete. The properties discussed include mixture proportions, mechanical properties, various aspects of durability, and permeability. It is concluded that, in general, high-volume fly ash concrete has excellent strength development and durability characteristics, and exhibits very low temperature rise. These properties make it ideal for use in large mat foundations, thick-structural elements and other similar applications.

Keywords: Concrete, Durability, Fly ash, Heat of hydration, Mechanism of hydration, Permeability, Strength, Superplasticizers, Thermal properties.

ACI Honorary Member **V.M. Malhotra** is the Program Principal for the Advanced Concret Technology Program, Canada Centre for Mineral and Energy Technology (CANMET), Dept of Natural Resources, Ottawa, Canada. He is a prolific author, editor, and researcher, an has received many awards and honours from ACI and other institutions throughout th world.

INTRODUCTION

Low-calcium fly ash (ASTM Class F) has been incorporated into concrete a replacement for portland cement for the construction of mass gravity dams since 1935 (1 The primary aim in the use of fly ash was, and has been to reduce the heat of hydration The low early-age strengths obtained by the use of fly ash were not critical as th compressive strength requirements were generally of the order of 20 MPa at 90 days o one year. In earlier years, the percentage of fly ash used was limited to about 30 per cen In 1965, Mather (2) reported an investigation in which he studied the effects of 60% soli volume replacement of cement with fly ash. For water-to-(cement + fly ash) rati (W/C+F) of 0.50 and 0.80, he reported three-day strengths of 4 and 1.5 MPa respectively. In recent years, large volumes of fly ash have been incorporated into th construction of roller-compacted concretes but, once again, the low-early age strengt are not of concern (3).

In 1985, CANMET initiated studies on structural concrete incorporating high volum (>50%) of low-calcium fly ash. The purpose of this research was to develop concre with adequate early-age strength and workability, low temperature rise and high later-ag strength. This was successfully achieved (4-19).

This paper discusses the mechanical properties and durability aspects of high-volum fly ash concrete and outlines its limitations.

MIXTURE PROPORTIONS

Like normal portland cement concrete, the mixture proportions of high-volume fly a concrete will depend upon the strength level required. In general, the cement conte varies from 100 to 180 kg/m³ with W/C+F ranging from about 0.30 to 0.40. The wa and fly ash contents are kept at about 115 and 220 kg/m³, respectively, and slumps excess of 150 mm are achieved with high dosages of superplasticizers. In studies CANMET, the maximum size of coarse aggregate has been kept at 19 mm, but hig volume fly ash concretes have been made using 25 and 39 mm maximum size aggrega The high percentage of fly ash in the concrete mixture does not pose any serious proble for the entrainment of 5 to 6 per cent air in concrete except that large dosages of the a entraining admixtures are required. However, this has no adverse effect on the proper of fresh and hardened concrete (6). Nevertheless, exploratory investigations should be performed to ensure that air-entraining admixtures used are compatible with the type of cement and fly ash incorporated into the concrete mixture. Typical mixture proportions for the high-volume fly ash concrete are shown below:

	Batch Quantities
ASTM Type I Cement:	150 kg/m³
ASTM Class F fly ash:	210 kg/m³
Water:	115 kg/m³
Coarse aggregate (19-mm max):	1275 kg/m³
Fine aggregate:	620 kg/m³
Air-entraining admixture:	720 mL/m³
Superplasticizer:	4.0 L/m³

The physical properties and chemical analysis of the various fly ashes used are given in Table 1.

PROPERTIES OF FRESH CONCRETE

Bleeding

Bleeding tests performed on high-volume fly ash concrete have shown that this concrete does not bleed (Table 2). This is due to the very low water content (≈115 kg/m³) used in this type of concrete (11). This would necessitate additional curing in hot weather to avoid plastic shrinkage cracks.

Entrainment of Air in Fresh Concrete

No difficulty has been encountered in the entrainment of air in high-volume fly ash concrete, though the dosage required is considerably more than that for portland cement concrete without fly ash. However, it should be pointed out that the fly ashes used in the high-volume fly ash concrete systems have relatively low carbon content. It may be difficult to entrain high percentages of air in the above system if the fly ash has carbon content >6%, but the judicious selection of an air-entraining admixture can overcome this problem (14).

Density of Fresh Concrete

Investigations at CANMET have shown that the density of high-volume fly ash concrete is of the order of 2400 kg/m³, and is comparable with the density of portland cement concrete without fly ash. Considering that the specific gravity of fly ash is generally lower than that of portland cement (2.6 versus 3.1), the density of high-volume fly ash concrete is considered high, and this is due to the reduced water content due to the use of superplasticizers.

Dosage Requirement of Superplasticizer

Because of the very low water-to-cementitious materials ratio (≈0.30) of the high-volume fly ash concrete, the use of superplasticizers becomes mandatory. The dosage required will depend upon the slump to be achieved; for flow concrete, the dosage is of the order of 1.5 per cent of the total cementitious material (≈5 kg/m³). There is also a question of compatibility between superplasticizers and portland cements in the presence of fly ashes. Exploratory investigations should always be performed to ensure that there is no undue delay in the setting time of concrete when using the high-volume fly ash concrete system (11,14).

PROPERTIES OF HARDENED CONCRETE

Temperature Rise

Because of the very low cement content, the temperature rise in the high-volume fly ash concrete during the first few days after placement is very low. For example, the peak temperature measured at mid-depth of a high volume fly ash concrete block (1.5 x 1.5 x 1.5 m) at 2 days was 31.3°C when the ambient and placing temperatures were 24° and 12°C, respectively (12).

Strength Properties

The high-volume fly ash concrete exhibits adequate strength development characteristics both at early and later ages. For example, the one-day compressive strength of high-volume fly ash concrete can vary from 5 to 9 MPa depending upon the type of cement. These strength values are more than adequate for formwork removal at normal temperatures, and are comparable to the strength developed by portland cement concrete with about 250 kg/m³ of cement. The later-age compressive strength in some instances have been shown to reach about 60 MPa at one year. The 28-day flexural and splitting-tensile strengths of the high-volume fly ash concrete are about 15 and 10 per cent of the corresponding 28-day compressive strength, and these are, once again, comparable to the values for the normal portland cement concrete. Fig. 1 shows typical compressive strength development data for high-volume fly ash concrete.

The compressive strength development of air-cured high-volume fly ash concrete preceded by 7 days of initial moist curing as compared with the continuously moist-cured specimens follows the same trends as the control concrete i.e. the strength of the former concrete (air cured) is somewhat lower than the latter concrete (moist cured) (Fig. 2).

Young's Modulus of Elasticity

The Young's modulus of elasticity of high-volume fly ash concrete made with limestone aggregate generally exceeds 30 GPa, and is thus somewhat higher than the modulus for comparable strengths of portland cement concrete. The higher modulus is probably due to large percentage of aggregates, and in addition, due to the unhydrated fly ash particles acting as a fine filler material in the concrete.

Creep Characteristics

The creep characteristics of high-volume fly ash concrete are shown in Fig. 3. The creep strain at one year ranges from 150 to 400 x 10^{-6} which is comparable to or lower than that of the portland cement concrete of comparable strength.

Drying Shrinkage

The drying shrinkage strains of high-volume fly ash concrete and normal portland cement concrete, determined in accordance with ASTM C 157, are comparable (11). Fig. 4 shows such data on drying shrinkage strains on fly ash and control concrete prisms which had been cured in lime-saturated water for either 7 or 91 days and then air dried at 23 ± 1.7° and 50 ± 4 per cent relative humidity for periods up to about 500 days.

Water Permeability

Water permeability tests using a CANMET uniaxial flow apparatus were performed on the fly ash concrete incorporating 150 kg/m³ of ASTM Type I cement and 190 kg/m³ of fly ash. Briefly, the test method consists of measuring the uniaxial water flow through concrete cylinders, 125 mm high with a diameter of 150 mm, under a pressure of 3.5 MPa. The test specimens were maintained in the permeability cells for more than 6 months, and it was found that no water had passed through the specimens during this period. Based upon data previously published by CANMET, this indicates that the concrete tested has extremely low permeability, although no numerical values can be assigned (20).

DURABILITY ASPECTS OF HIGH-VOLUME FLY ASH CONCRETE

Resistance to the Penetration of Chloride Ions

Several investigations at CANMET have indicated that high-volume fly ash concrete has very high resistance to the penetration of chloride ions (6,7). The AASHTO T277-83 test method, "Rapid Determination of the Chloride Permeability of Concrete" is the most commonly accepted test in North America, and was therefore, adopted as the preferred technique. Briefly, the above method consists of monitoring the amount of electrica current passed through a 102-mm diameter by 51-mm thick concrete specimen when potential difference of 60V dc is maintained across the specimen for a period of six hours. Chloride ions are forced to migrate out of a NaCl solution subjected to a negative charg through the concrete into a NaOH solution maintained at a positive potential.

The conditioning of the concrete disc specimens for the test procedure consists of on hour of air drying, three hours of vacuum (pressure <1 mm Hg), one hour of additiona vacuum with specimens under de-aerated water, followed by 18 hours of soaking in water. The total charge passed, in coulombs, is used as an indicator of the resistance of the concrete to the passage of chloride ions.

The test results encompassing a number of investigations involving the use of severa different fly ashes have indicated that the resistance of high-volume fly ash concrete t the penetration of chloride ions, as measured by the charge in coulombs, is very high. The values of the charge ranged from 150 to 973 coulombs for concretes tested at ages ranging from 28 to 91 days. It is generally agreed that for low permeability concretes, the value of the charge in coulombs passed through the specimens should not exceed 1000 and for very low permeability concretes this value should preferably be less than 600. I silica fume concrete incorporating 400 to 500 kg/m³ of portland cement and 10 per cen silica fume, chloride-ion penetration tests yield charge in coulombs generally less than 800.

Carbonation

Limited data on carbonation tests performed on broken portions of cores drilled afte five years from a large block of high volume fly ash concrete have been obtained. Th block was cast in December 1985 and moist cured for 28 days. Following that the bloc was left in a room with limited ventilation and at a temperature of about 23°C and i relative humidity of 40 to 50 per cent. The average carbonation depth was less than 10 mm after 5 years.

Control of Expansion Due to Alkali-Aggregate Reaction

The undesirable expansion of concrete due to reaction between the cement alkalies and certain types of silica in aggregates is a universal problem. Research at CANMET an elsewhere has shown that the above alkali-silica reactions in concrete can be controlle by incorporating good quality fly ash as a partial replacement for cement. The general recommended levels of cement replacement by fly ash are between 25 and 40 per cen

As described earlier, high-volume fly ash concrete incorporates about 56 per cent o fly ash as a percentage of total binder, together with large dosages of a superplasticize It was, therefore, considered prudent to perform investigations to ensure that th contribution of alkalies, both from the fly ash and the superplasticizers, will not adverse

affect the ability of high-volume fly ash concrete to control or considerably reduce the above expansive reaction when reactive aggregates are used.

In order to determine the role of high-volume fly ash concrete in controlling expansion due to alkali-aggregate reaction, two concrete mixtures were made, one control and one incorporating ASTM Class F fly ash (13). The coarse aggregate used was 19-mm crushed limestone; this limestone contains a highly reactive silica phase and has a known history of expansive reaction in concrete.

A number of 75 x 75 x 305-mm prisms were cast and subjected to the following seven test regimes to determine the expansion due to the alkali-silica reaction.

Test regime 1:	Continuous curing of the prisms in a moist-curing room maintained at 38°C.
Test regime 2,3:	Continuous curing of the prisms in 5% NaCL solution maintained at 38 and 80°C after an initial moist curing for 24 h.
Test regime 4,5:	Continuous curing of prisms in 1 normal NaOH solution maintained at 38 and 80°C after an initial moist curing for 24 h.
Test regime 6,7:	Continuous curing of prisms in 1 normal KOH solution maintained at 38 and 80°C after an initial moist curing for 24 h.

The prisms were exposed to the above regimes for a period of 275 days.

The test results which have been published elsewhere indicated that, regardless of the test procedure used, the test prisms cast from the high-volume fly ash concrete did not show any expansion in spite of the very reactive coarse aggregate used in the concrete (13). The above tests confirm the previously published data that fly ash can play an effective role in controlling the alkali-silica reactions in concrete.

LIMITATIONS

The major limitations in the utilization of high-volume fly ash concrete is the availability of good quality fly ash within economic haulage of the construction site and adequate silo capacity. The other limitations include the compatibility between fly ash/cement and superplasticizer. This needs to be investigated as each source of fly ash is unique. In some instances the initial and final setting of the high-volume fly ash concrete may be retarded by several hours. This problem can likely be resolved by changing the type and brand of the superplasticizers used.

The performance of high-volume fly ash concrete subjected to de-icing salts is relatively poor and further research is needed before the use of this type of concrete can be recommended for this application involving this kind of exposure.

ACKNOWLEDGEMENT

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Table 1- Physical Properties and Chemical Analysis of Various Fly Ashes Used in High-Volume Fly Ash Concrete

	Fly Ash A	Fly Ash B	Fly Ash C
Physical Properties			
Fineness - passing 45 µm, %	78.8	82.7	80.6
- Blaine, m²/kg	--	289	326
Specific Gravity	2.38	2.53	2.05
Chemical Analysis			
Insoluble residue	--	--	--
Silicon dioxide (SiO_2)	45.1	47.1	55.6
Aluminum oxide (Al_2O_3)	22.2	23.0	23.1
Ferric oxide (Fe_2O_3)	15.7	20.4	3.48
Calcium oxide (CaO), total	3.77	1.21	12.3
Magnesium oxide (MgO)	0.91	1.17	1.21
Sulphur trioxide (SO_3)	1.40	0.67	0.30
Sodium oxide (Na_2O)	0.58	0.54	1.67
Potassium oxide (K_2O)	1.52	3.16	0.50
Loss on ignition	0.32	2.88	0.29

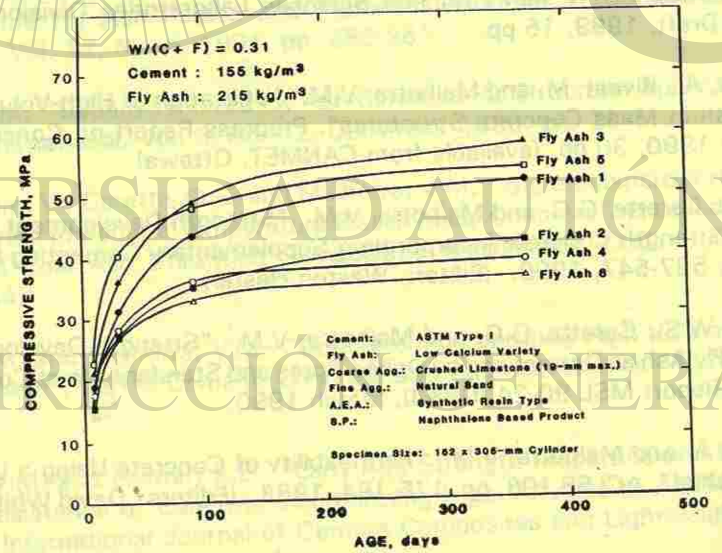


Fig. 1- Compressive strength development of high-volume fly ash concrete w
155 kg/m³ of cement.
From reference 14.

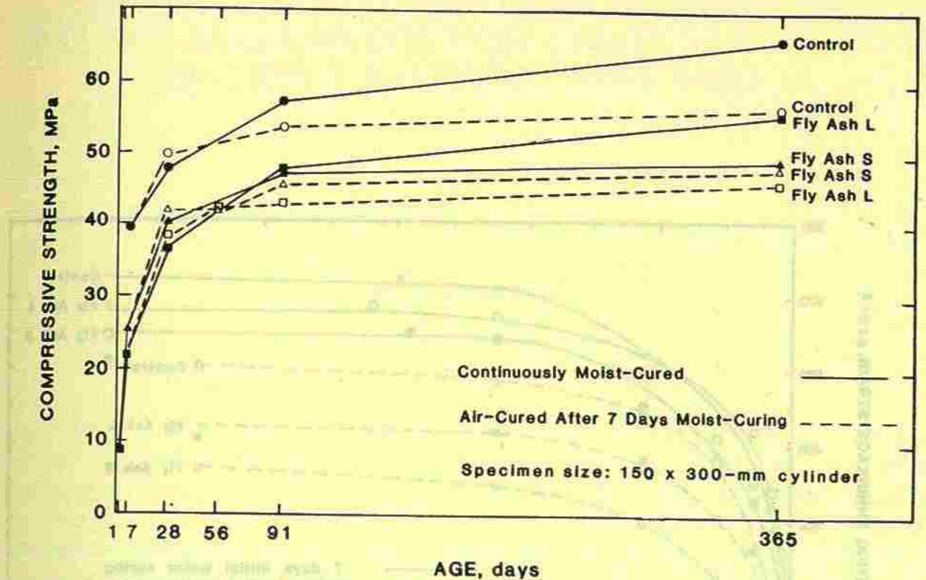


Fig. 2 - Compressive strength development of test cylinders under moist-cured and air-cured conditions.

From reference 12.

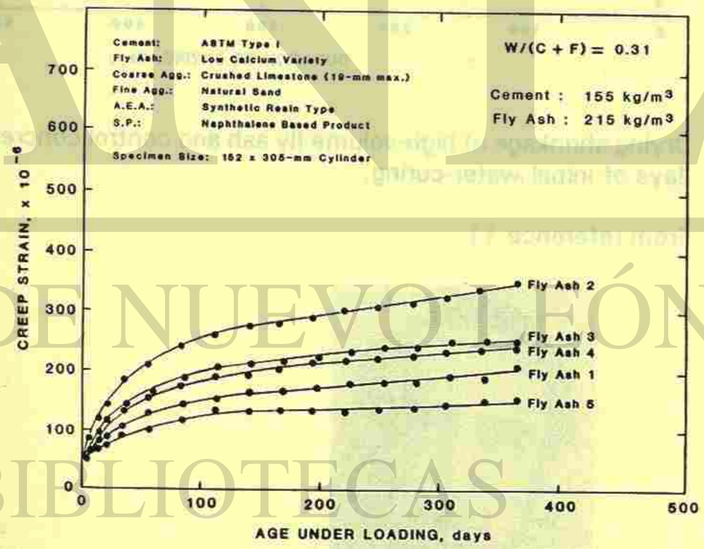


Fig. 3 - Creep strains for high-volume fly ash concrete.

From reference 14.

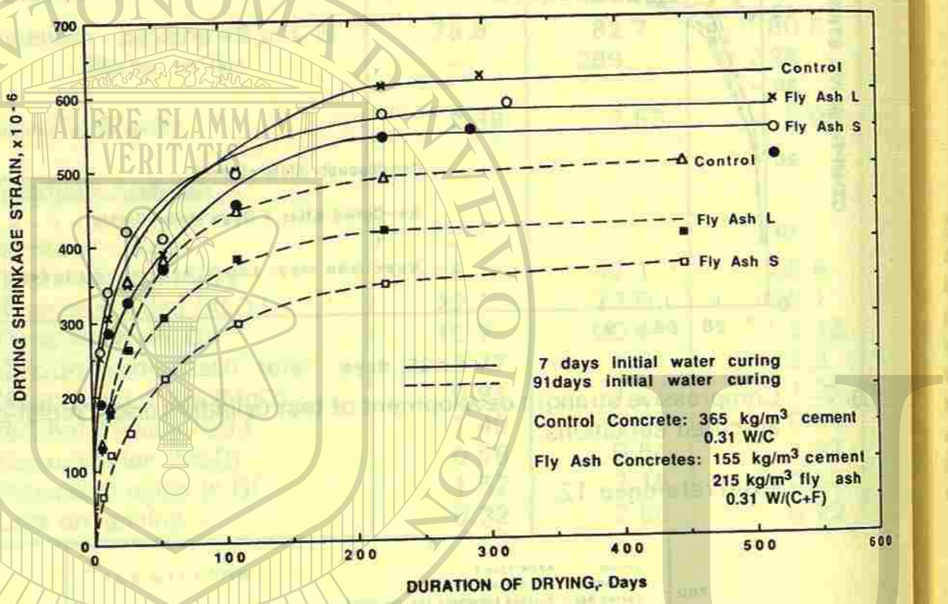
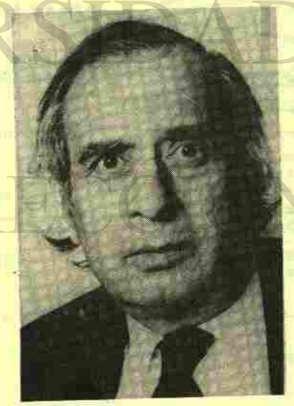


Fig. 4 - Drying shrinkage of high-volume fly ash and control concrete after 7 and 91 days of initial water-curing.

From reference 11.



V. Mohan Malhotra, ACI Honorary Member
Program Principal for the Advance
Concrete Technology Program, CANMET,
Dept. of Natural Resources, Canada.

DURABILIDAD A LARGO PLAZO DEL CONCRETO ESTRUCTURAL CON MICROSILICA, CONCRETO LANZADO, MORTERO DE AGREGADO FINO, LOSA PARA CUBIERTAS Y PARCHES

por

John Wolsiefer y Ken Clear

Sinopsis: La microsílica previene la corrosión del acero de refuerzo en el concret mediante la disminución de la permeabilidad al ion cloro y el aumento de la resistividad eléctrica. Los programas de ensaye de campo a largo plazo fueron llevados a cabo pa concreto estructural, concreto lanzado, mortero de agregado fino, losa para cubiertas y parches de concreto. Los resultados de los ensayes muestran que el concreto co microsílica, concretos lanzados y morteros de agregado finos exhibían un aumento en l resistencia, muy baja permeabilidad, muy alta resistividad eléctrica, excelente resistenci a la adhesión y durabilidad al congelamiento y descongelamiento. Los parámetro positivos de durabilidad aumentan con el tiempo, dando como resultado un concret extremadamente durable en ambientes agresivos con ion cloro.

Palabras clave: Durabilidad, microsílica, rápida permeabilidad al ion cloro, resistivida eléctrica, concreto estructural, concreto ligero, concreto lanzado, mortero de agrega fino, parchado del concréto, cubiertas de concreto.

LONG TERM DURABILITY OF SILICA FUME STRUCTURAL CONCRETE, SHOTCRETE, GROUT, SLAB OVERLAYS AND PATCHES

by

John Wolsiefer and Ken Clear

Synopsis: Silica fume prevents reinforcing steel corrosion in concrete by decreasing chloride permeability and increasing electrical resistency. Long term field test programs were conducted for structural concrete, shotcrete, grout, slab overlays and concrete patches. The test results show that silica fume concrete, shotcretes, and grouts exhibits increased strength, very low permeability very high electrical resistivity, excellent bond strength and freeze/thaw durability. The positive durability parameters increases with time, giving an extremely durable concrete in aggressive chloride environments.

Keywords: Durability, silica fume, rapid chloride permeability, electrical resistivity, structural concrete, lightweight concrete, shotcrete, grout, concrete patching, concrete overlays.

JOHN WOLSIEFER, SR. is President of Norcem Concrete Products, a silica fume manufacturer. He has 19 years experience, including past CEO positions, in consulting, ready mix and construction. He has a B.S. Applied Physics, a M.S. Managemen Engineering, and is a member of ACI, ASTM, and committees: ACI 363, High Strength; ACI 234, Silica Fume; and ASTM F12.8, Protective Containment Structures. He has given numerous presentations, authored papers on silica fume, and made a significan contribution to the development of silica fume concrete technology.

KENNETH C. CLEAR obtained a B.S. and M.S. in Civil Engineering at Pennsylvania State University. He spent 11 years heavily involved in the Federal Highway Administration's programs aimed at the elimination of premature bridge deck deterioration and his "Time-to- Corrosion" research efforts provided valuable information on the effect of concrete mix design, construction techniques and special treatments on salt penetration into concrete and subsequent corrosion of the reinforcing steel. Mr. Clear is a member of ASTM, NACE, ASCE, TRB, an ACI Fellow and received the 1987 Corrosion Award at the Mathe International Conference on Concrete Durability.

INTRODUCTION

Since the mid 70's, silica fume has been used in the United States as a means o enhancing the durability of concretes and grouts. The initial uses were in the chemica industry, where increased resistance to acid was required. Beginning in about 1980, th authors began to consider silica fume (dry, uncompacted, in concert with superplasticize as a means of significantly decreasing the chloride permeability of concrete and increasin its electrical resistivity. The need for improved concrete properties, is related to corrosio of the reinforcement, as a result of exposure to deicing salt or marine environment Conventional concretes and grouts exhibit relatively high permeability and low resistivi The chloride, once present at the reinforcing level at a concentration of about 1.3 lbs/c $(0.77 kg/m^3)$ or 0.035% by weight of concrete, destroys the passive film on the steel an acts as a catalyst, in a cancerous process, which results in concrete cracking, delaminatio (undersurface fractures at the level of the steel) and spalling. In some structures, th metal loss caused by corrosion can be significant and lead to structural performan problems. Many millions of dollars have been expended in the last two decades to rehabi tate existing bridge and parking decks and to provide protective systems for ne structures. This paper summarizes our findings in select laboratory, outdoor exposure an field efforts in the last 13 years.

Initial Laboratory Test Programs

In the early 1980's, the authors initiated laboratory test programs to evaluate th potential of silica fume concrete to prevent chloride induced reinforcing steel corrosio

concrete. One of the first tests performed was that of the Federal Highway Administration (FHWA) 90 day chloride ponding test, which measured chloride ion penetration by ponding 3% NaCl onto concrete. This work showed that silica fume concrete mixes (20% by weight of cement) could reduce chloride penetration, by 98% compared to the control.[1] Another early test was the National Cooperative Highway Research Program (NCHRP) cube test, which measured water absorption and chloride penetration, under air dry, chloride soak cycles. The test data showed that silica fume mixes (20% by weight of cement) reduced the percent absorbed chloride to 16% of the control.[1] The time period to conduct these test procedures proved too long and AASHTO T277-83, "The Standard Method of Test for Rapid Determination of Chloride Permeability of Concrete" was then adopted to measure the performance potential, of silica fume concrete. Briefly, the test involves (1) trimming a 4-inch diameter core or cylinder to a 2-inch length, (2) vacuum saturation and overnight soak, (3) the sealing of the core in a permeability cell, with electrodes and specific solutions on each side, and (4) the application of 60 volts DC for 6 hours. The coulombs (amp-sec) of total charge passed is determined and has been correlated with chloride permeability. The lower the charge passed the lower the permeability. Resistivity, another important material property, which affects corrosion rate, can also be estimated, in this test, by measuring the AC resistance between the two electrodes prior to test start and converting to resistivity, using an experimentally defined cell constant. The resistivity, when measured in this way at 70°F is referred to as the wet resistivity because of the vacuum saturation and soak. Conventional concrete typically yields a charge passed in excess of 2000 coulombs and a wet resistivity less than 10,000 ohm-cm. The relatively high permeability allows chloride, from the environment, to readily penetrate, and the relatively low resistivity allows corrosion to continue (in the presence of oxygen) at a high rate. Thus, for maximum corrosion control, a concrete should exhibit very low permeability, but very high resistivity.

Initial Field Test Program:

During this time period, the primary use of silica fume in the United States, was the protection of concrete from chemical attack. The concrete deterioration was two fold, (1) cement paste failure due to acid and (2) reinforcing steel corrosion and concrete delamination due to salt penetration. As a silica fume admixture (dry silica fume and super at 20% by weight of cement) was performing well, in the chemical environment, an existing project with four years of service, was chosen for concrete field evaluation. Cores were extracted from a bulk storage fertilizer warehouse floor, (prilled ammonium nitrate fertilizer with a pH of 5.8 and a concentration of about 98 percent). The warehouse silica fume field concrete had a water/cement + silica fume (W/C + S) ratio of 0.35, 550 pounds of portland cement per cubic yard $(325 kg/m^3)$ and 110 pounds $(0.65 kg/m^3)$ of silica fume per cubic yard. The concrete was air-entrained and had a placement slump of 5 to 7 inches (127 to 178 mm). The RPT tests, performed in 1983, yielded results varying from 154 to 333 coulombs and averaging 226 coulombs. The wet concrete resistivities ranged from 52,000 to 107,000 ohm-cm. This resistivity range is the area, in which, most agree that corrosion problems will be minimal. Thus, this and other field testing confirmed that the dry silica fume/superplasticizer combination, that was performing well in the chemical environment, had the potential to provide a field concrete of very low permeability and high resistivity, suitable for concrete in chloride environments.

Long Term Structural Concrete Test Program:

A laboratory and outdoor exposure test program was initiated in 1982 to study the characteristics of silica fume concrete and continues today. Task 1 of the program involved the fabrication and testing of specimens using the two high strength silica fume concrete mix designs shown below:

SF Mix 1: W/C+S = 0.28
750 lbs cement/cy (441 kg/m³)
150 lbs silica fume admixture (88.5 kg/m³)

SF Mix 2: W/C+S = 0.22
1000 lbs. cement/cy (590 kg/m³)
200 lbs. silica fume admixture (118 kg/m³)

Normal weight aggregates were used in this part of the program. A conventional concrete (W/C = 0.50 and 615 lbs cement/cy [363 kg/m³]) was included for comparison. The average moist cured concrete compressive strengths were:

Concrete	Compressive Strength, psi (Mpa)		
	7 Day	28 Day	90 Day
Conventional	2,944 (20.3)	4,257 (29.4)	5,911 (40.7)
SF Mix 1	7,162 (40.4)	11,110 (76.6)	12,449 (85.9)
SF Mix 2	8,912 (61.5)	13,240 (91.3)	14,282 (98.5)

Concrete slabs were cast from each mix and cured in a fog room. Rapid permeability tests were run on portions of the cores from two slabs representing each mix (6 cores per variable) between 60 and 90 days of age. Average results ere as follows:

Concrete	Charged Passed coulombs	Wet Resistivity ohm-cm
Conventional	6,480	6,850
SF Mix 1	55	372,000
SF Mix 2	43	417,000

The silica fume structural concretes exhibited negligible permeabilities and extremely high resistivities, even when water saturated. At these levels of permeability and resistivity, it is doubtful that corrosion could be a problem in the long term. To confirm this, concrete slabs with two mats of reinforcing steel (1-inch [25.4 mm] cover) and the above concrete mix designs were cast, cured and subjected to 24 weeks of NCHRP 244 southern exposure testing (Figure 1). Each weekly cycle consists of 4 days of ponding with 15 percent sodium chloride solution at 70°F, and three days unponded at 100°F. Figure 2 shows the macrocell corrosion current densities measured during this test series. Corrosion of the reinforcing steel in the conventional concrete was initiated after only 4 weekly cycles, but no corrosion occurred on the steel in any of the slabs containing silica fume. After the 23 weekly cycles, the resistivity (70°F) of the concrete between the two rebar mats in each slab was determined. The average values for each variable were:

Concrete	Mat to Mat Resistivity, ohm-cm
Conventional	19,500
SF Mix 1	440,000
SF Mix 2	673,000

These data show that the silica fume concretes exhibited resistivities which were much more higher (22 to 34 times higher), than that of the conventional concrete. All these resistivities are higher than those given above in the case of the rapid permeability tests (RPTs). Such occurs because the RPT test resistivity is a wet resistivity, whereas, the concrete in the test slabs is not saturated with water. Chloride analyses results after the 24 southern exposure cycles are shown in Figure 3. The conventional concrete underwent significant chloride penetration, whereas, the silica fume concretes did not. The conventional concrete slab testing was discontinued because rust staining and concrete cracking had begun. Autopsy showed severe corrosion of the top mat reinforcing. No rust staining, cracking, corrosive half cell potentials or macrocell current was present on any of the silica fume concrete specimens. Because of the good performance, the silica fume concrete slabs were subjected to another 24 weekly cycles of southern exposure. During these additional cycles, no corrosion occurred on any of the silica fume concrete slabs (see Figure 2). The slabs were then transferred to above ground racks at the KCC INC. outdoor exposure facility, in Virginia, where they have been subject to natural weathering, including freezing and thawing for about 8 years. No surface scaling, cracking, rust staining or other deterioration has occurred. Corrosion performance has been monitored using half cell potentials (ASTM C876), macrocell corrosion measurements and mat to mat resistance measurements. Also, in 1992, cores were extracted from one slab representing each variable and analyzed for chloride, permeability and resistivity. The resistivity of the SF Mix 1 concrete slabs has averaged 613,000 ohm-cm during outdoor exposure, and that for the SF Mix 2 slabs has averaged 1,050,000 ohm-cm. No significant decreases in resistivity with time have occurred on any of the silica fume concrete slabs; indicating that no deterioration of the corrosion resisting properties has occurred during the 8+ years of exposure. All half cell potentials have been more positive than -200 mV CSE, indicating a very high probability of no steel corrosion. Mat to mat macrocell corrosion currents have been zeroon all slabs throughout the eight years of outdoor exposure. Examination of the reinforcing steel extracted from the cores and the concrete traces showed conclusively that no corrosion had occurred. Chloride analyses in March 1992, indicated that very little chloride had penetrated the concretes (about 1.3 lbs/cy [0.77 kg/m³] or 0.035% by weight of concrete, is required to induce corrosion), as follows:

Sampling Depth, inches (mm)	Chloride, % by Weight of Concrete	
	SF Mix 1	SF Mix 2
1/16 (1.6) to 0.5 (12.7)	0.0051	0.0032
0.5 (12.7) to 1 (25.4)	0.0017	0.0013
1 (25.4) to 1.5 (38.1)	0.0005	0.0008

Obviously, very, very little chloride penetrated these silica fume concretes during the 48 weeks of southern exposure (ponding 4 days of each week with 15 percent NaCl solution); and what little did penetrate, did not migrate deeply into the concrete during the subsequent 8 years of outdoor exposure. Carbonation depths are less than 1/16-inch and the material remains high pH throughout its thickness. Comparing these results to those in Figure 2 for the conventional concrete, after only 24 weeks of southern exposure highlights the great superiority of the silica fume concretes. The results of the rapid permeability tests on cores from the slabs (9 years of age) were as follows:

Concrete	Charge Passed coulombs	Wet Resistivity ohm-cm
SF Mix 1	48	374,000
SF Mix 2	3	1,539,000

These values represent the lowest permeabilities and the highest wet resistivities o any concrete containing portland cement studied in the KCC INC laboratories in 10 years Obviously, there has been no deterioration of these concretes, as a result of this tes program. The slab surfaces show no signs of freeze-thaw deterioration or cracking. The reinforcing steel extracted from cores showed no corrosion.

In 1986, silica fume concrete test specimens were prepared with a varying silica fum admixture dosage that was increased in discrete steps (zero to 22% by weight of cement After 42 days of moist laboratory cure, rapid chloride permeability and wet resistivi measurements were conducted. The resultant data showed that the rapid chlori permeability decreased, while the wet electrical resistivity increased, when the silica fum admixture dosage was increased. These tests were rerun April 1992, on previous untested 2-inch (50.8 mm) slices from the same specimens, after 7 years outdo exposure, in the KCC INC test yard. The specimens had been exposed to hundreds o freeze-thaw, wetting and drying, and heating and cooling cycles during their outdo exposure. the rapid chloride permeabilities (RCP) and wet resistivities are presented below

Silica Fume dosage %	RCP coulombs 1986	RCP coulombs 1992	Wet Resistivity ohm-cm 1986	Wet Resistivity ohm-cm 1992
12	100	40	197,400	449,400
15	97	36	205,800	483,000
22	50	13	352,800	1,554,000

The specimens showed no evidence of cracking or freeze-thaw damage. The data presented shows that the rapid chloride permeability has decreased in the range of 26 to 40%, while the resistivity has increased 2.3 to 4.4 times. This improvement in concrete quality is dramatic, considering the small specimen size and the large number of freeze-thaw cycles experienced yearly, in this Virginia test yard location. The field data certainly reinforces previous test results reported for freeze-thaw durability (according to ASTM C666, procedure A), at higher silica fume admixture dosages (20%)³.

Task 2 of the program involved the use of silica fume in lightweight concrete. In 1983, three lightweight concrete mixes were studied, each with 1000 lbs of portland cement per cubic yard (590 kg/m³) and superplasticizer. Slump was maintained in the range of 4 to 6 inches (102 to 152 mm) and the maximum lightweight aggregate size was 3/4 inch (19 mm). LtWt Mix 1 was the non-air entrained lightweight structural concrete control. SF LtWt Mix 2 had 20 percent silica fume admixture (by weight of cement), and SF LtWt Mix 3 had 30 percent silica fume admixture (by weight of cement) and air-entrainment. The 90 day compressive strengths, rapid permeabilities and resistivities are presented below:

Concrete	Comp. Str, psi (Mpa)	Charge Passed, coulombs	Resistivity, ohm-cm
LtWt, non-AE	9,230 (63.7)	3,667	5,850
SF LtWt, AE	9,071 (62.6)	357	54,100
SF LtWT, non-AE	10,880 (75.0)	277	41,400

These data indicate that the addition of 20 percent dry silica fume by weight of cement will result in a lightweight structural concrete of much lower permeability and much higher resistivity. Another lightweight aggregate concrete study was performed in 1986 using a shrinkage compensating cement and dry silica fume. Trial mixes were made with lightweight aggregate, superplasticizer and 0, 10 and 20 percent silica fume by weight of cement (658 pcy [388 kg/m³] of shrinkage compensating cement). Slumps ranged from 5 to 8 inches (127 to 203 mm) and all concretes were air entrained. The properties of these concretes (strengths at 60 days, other properties are 42 days of age) are summarized below:

Concrete	Comp. Str, psi (Mpa)	Charge Passed, coulombs	Wet Resistivity ohm-cm
LtWt, KC-AE	5,490 (37.8)	3,780	7,500
10% SF LtWt, KC-AE	7,480 (51.6)	542	33,600
20% SF LtWT, KC-AE	7,878 (54.5)	364	58,800

These data show that both 10 and 20 percent silica fume additions by weight of cement greatly improve both the permeability and the resistivity properties of a shrinkage compensating cement and lightweight aggregate concrete.

Project Field Testing:

Task 3 of the structural concrete test program involved quality control monitoring and outdoor exposure of concrete cylinders cast on field projects in which dry silica fume and superplasticizer were incorporated in parking deck concretes and marine loading docks throughout the United States. Approximately 15 projects were monitored over a five year period. Select examples are presented below.

A marine transfer terminal in New York was reconstructed in late 1986, using silica fume concrete. This project encompassed the encapsulation and repair of pile caps underwater, by tremie method, overlay of a ramp roadway and tipping platform subjected to truck traffic and the construction of a new ramp bridge with piles, beams and full depth roadway. The silica fume concrete mix design had a 750 lb/yd³ (44 kg/m³) cement factor with a silica fume admixture weighing 165 lbs/yd³ (90 kg/m³ The compressive strength averaged 11,430 (78.8 Mpa) psi at 56 days with 6% a entrainment. The 42 day charge passed on field cylinders cast in February 198 (laboratory moist cure) averaged 183 coulombs and the wet resistivity average 77,175 ohm-cm. Other portions of these specimens were boil cured to provide a indication of the long term permeability, of this concrete, after additional curing time The average charge passed was 56 coulombs and the average wet resistivity was 358,000 ohm-cm. Examination of this structure in 1990 showed that both the rehabilitated and new silica fume concrete was performing well.

A prestressed parking deck was constructed in New Jersey in 1988. Eleven percent silica fume was utilized, with a cement content of 658 lb/yd³ (388 kg/m³) and the W/C+S ratio was 0.34. The charge passed, on boil cured field cylinders, average 517 coulombs and the wet resistivity averaged 46,700 ohm-cm.

A prestressed Missouri airport terminal parking garage, constructed in 1988, utilized 10.5 percent dry, compacted silica fume. Superplasticizer was used and the W/C+ ratio was 0.32. The average charge passed for 32 ninety day field cores was 30 coulombs. The wet resistivities were in excess of 45,000 ohm-cm.

Silica fume admixture in concrete was used in 1983 in construction of a one mile long channel at a New York State power station. The 14 foot (4.3m) wide channel encircled a coal storage pile (on plastic liner), and was utilized to channel sulfuric acid run-off to a neutralization pond. The silica fume concrete mix design had a 750 lb/yd³ (441 kg/m³) cement factor, and a 165/yd³ (97.4 kg/m³) silica fume admixture (SF 20% by weight of cement). Cores were extracted from the completed structure and tested in RPT. The average charge passed for two cores was 252 coulombs and the average wet resistivity was 72,000 ohm-cm. Conversations with a plant supervisor, in 1991, indicated the channel was in good condition.

Silica Fume Grout:

Silica fume was first considered, by the authors as a means of decreasing the permeability and increasing the resistivity of portland cement grout in 1982. A Maryland parking deck rehabilitation required the installation and grouting of steel collars, on select beam/column connections, because stirrup steel had not been properly used during construction. Through the garage's 10 years of service life, the prestressed beams had shortened and were becoming separated from the columns. Severe chloride contamination had occurred at the beam/column connections as a result of improper calcium chloride additions during construction, and/or deicing salt leakage. The structural detail desired was a steel collar, surrounding both the beam end and the top portion of the column (see Figure 4), which was bolted in place to the existing concrete. Because of the high concentration of steel in the concrete, preventing electrical contact between the collar and the reinforcing was impractical. Thus, once the grout was placed, it was feared that the inside steel surface of the collar would be cathodic to the reinforcing and prestressing steels, in the structure and greatly increase their corrosion rates. The only viable means of avoiding this was to increase the grout resistivity to an extremely high value. The cost of doing so with polymer mortar was prohibitive. A commercially available non-shrink grout (sand/cement grout) was tested and found to exhibit low wet resistivity. Therefore, a program of modifying the grout with dry, uncompacted silica fume was undertaken. A twenty percent dry silica fume admixture (silica fume and superplasticizer) was added by weight of cement to the grout. The water content of the mix was adjusted for equal grout flow and tests were conducted to determine placement time (workability loss with time), strength, permeability and resistivity. A mini-slump cone with measurements of slump and spread was used to define workability with time. The mix water for equal flow and pumpability was reduced by 25 percent, when the silica fume admixture was added. The working time (considered as the time until 50% loss in slump or spread) was increased, from 30 minutes for the commercial grout, to 45 minutes for the silica fume modified grout. The 28 day moist cured cylinder strength was 6,197 psi (42.7 Mpa) for the commercial grout and 8,300 psi (57.2 Mpa), for the silica fume modified grout. The permeability and resistivity findings (averages, 60 day water cure) were as follows:

Grout	Charge Passed coulombs	Wet Resistivity ohm-cm
Commercial	14,390	3,400
SF Modified	116	195,000

Obviously, the silica fume admixture greatly improved the corrosion resisting properties of the grout. The permeability was reduced by 99 percent and the resistivity was increased 5,700 percent (i.e. 57 times). The modified grout was successfully used to grout all the collars on the parking garage in 1983. The material was prepackaged such that one bag of commercial sand-cement grout, one bag of silica fume admixture and 1 gallon (3.8 L) of water was mixed in a mortar mixer. The mixed grout was placed in the pump hopper and pumped with normal grouting equipment. Quality control in the field was accomplished using "resistivity cups" (100ml plastic beakers fitted with four platinized wires). A grout sample was made from each batch, covered with plastic for one day and then placed in limewater. By monitoring the AC resistance with time and converting this to resistivity using the experimentally determined cell constant, it was determined that the field grout provided a cured resistivity in excess of 75,000 ohm-cm in all cases. The beam/column connections were inspected in 1990, seven years after installation. All collars remained in place with no signs of grout or surrounding concrete deterioration.

A second silica fume grout project was undertaken in 1986 for the purpose of grouting prestressed tieback anchors (in acid soil), for a highway retaining wall, located in Virginia. Both portland cement and cement-sand grout were studied and modified with 20 percent silica fume and powdered superplasticizer. Approximately 87 tieback anchors, 9 inches (22.9 cm) in diameter and 50 feet (15m) long were involved in this project. Grout consistency was defined using a flow cone time (ASTM C939) in the range of 15 to 30 seconds. Because of the short time span for project implementation, it was necessary to use autogenous curing to accelerate all tests. This was accomplished in accordance with ASTM C684. The silica fume grout utilized significant superplasticizer and had a W/C+S ratio of 0.33, while the control portland cement grout had a W/C ratio of 0.40. The average permeabilities and resistivities of the various grouts are given below:

Grout	Charge Passed coulombs	Wet Resistivity ohm-cm
Portland Cement	32,097	960
20% SF + Cement	260	58,000
20% SF, Sand/Cement	134	147,000

The portland cement grout had very high permeability and very low resistivity. A limewater cured sample at 21 days of age exhibited a charge passed of 20,600 coulombs and a resistivity of 2,900 ohm-cm. Although slightly better, these values will provide little protection against ingress of adverse ions and corrosion. The high shrinkage of the portland cement grout is one cause of the high permeability. The best grout was obviously the 20% silica fume, Sand/Cement Grout (2.9 sand/cement ratio by weight); probably because of reduced shrinkage. Seven day compressive strengths of the silica fume grouts ranged from 6,500 (44.9) to 10,000 psi (69 Mpa). A 20% silica fume + cement grout (with superplasticizer) was used on the field project. The 3 day accelerated cure compressive strength of a sample obtained from the first trial field batch was 10,250 psi (70.7 Mpa); and the 21 day limewater cured permeability averaged 121 coulombs. The estimated resistivity averaged 163,000 ohm-cm at 21 days. The water for use in this trial batch was determined manually. Quality control on this project also involved the use of accelerated cured resistivity specimens indicated that a high quality material was not being made. The problem was traced to a faulty water meter, which was inputting 6.2 gallons (24 L) of water but only registering 3 gallons (11.5 L) which was then corrected. This experience certainly emphasizes the value of using resistivity and accelerated curing in quality control programs.

Silica Fume Shotcrete:

Shotcrete modified silica fume admixture (20 percent by weight of cement) was used to repair beam and column damage as a result of corrosion, induced by magnesium chloride manufacturing, in a Texas chemical plant. In 1983, cores were obtained from the completed project and the permeability and resistivity were defined. The average charge passed (3 specimens) was 128 coulombs, and the average wet resistivity was 184,000 ohm-cm. Conventional shotcrete tested in another program exhibited an average charge passed of 6,800 coulombs and an average wet resistivity of 2,750 ohm-cm.[2] Obviously, shotcrete durability properties, in adverse chloride environments, are improved greatly when 20% silica fume is used. A check with the company engineer in 1991, showed that the column and beam members, repaired with silica fume shotcrete, have performed well (over 8 years), with no additional repair required. A large shotcrete test program was completed in April 1991, which evaluated different silica fume product forms, for both wet and dry shotcrete.[2] Parameters measured were; shooting characteristics, plastic properties, strength, shrinkage, absorption, permeability and resistivity.

Test Program for Silica Fume Concrete for Patching:

Because of the high resistivity of silica fume concrete, its use was considered in parking garage patching in the mid 1980's. One major problem when patching a salt-contaminated structure, is the effect of the patched area on corrosion of the steel in the remaining salty concrete. The steel corrosion rate is very dependant upon oxygen reduction at the cathode. The cathode is the non-corroding side of the corrosion battery where the electrons released at the corroding site (anode) are consumed. When deteriorated concrete is removed, the rebar is cleaned and a normal concrete patch is placed, a new cathode is created. This tends to concentrate corrosion in the unpatched concrete directly around the patch and often results in premature failure there; and is the cause of the so called "ring effect" or "spall around a spall" phenomenon. Silica fume concrete had the potential to minimize this effect because its very high resistivity prevents steel in the patched area from coupling electrically with the steel in the original concrete by minimizing ion flow between the steel in the two areas. To confirm this hypothesis, concrete slabs with four corroding rebars in the top mat, were rehabilitated such that the concrete surrounding bars 1 and 4 (i.e. the outside bars) was removed (see Figure 5). The area with bar 1 was patched with silica fume concrete containing 20 percent fume by weight of cement, and the area around bar 4 was patched using a latex modified concrete (styrene-butadiene, 15% latex solids by weight of cement). Conventional concrete patches were not studied, since it was known that they would perform poorly (the resistivity of conventional concrete is less than that of latex modified concrete). Since the four top mat bars were not continuous within the concrete, they were rewired such that the actual electrons of corrosion could be measured via an external resistor. Portions of two slabs were patched with silica fume concrete and other portions were patched with latex modified concrete. Bars 1 and 2 (i.e. latex patch to original salty concrete) were so interconnected, as were bars 3 and 4 (salty original concrete to silica fume patch). Upon completion of the patches and wiring and three days of moist curing the slabs were exposed on above-ground racks at the KCC INC outdoor exposure facility in Virginia. Monthly, the corrosion of each patch-original concrete cell was monitored using macrocell corrosion measurements, half-cell potential measurements and AC resistance measurements. Monitoring began in October 1983 and continued thru April 1992 (i.e. 8.5 years). The average macrocell corrosion currents and the concrete resistance between the two test bars for each cell and patch material during the 8.5 years of testing are presented below.

Patch Concrete	Avg. Macrocell Corrosion Current microAmps	Avg. Concrete Resistance ohms
Latex Modified	58	760
20% Silica Fume	23	3,685

Thus, this 8.5 year test program shows that the best concrete for use in minimizing the aggravation of corrosion during patching is a silica fume concrete. For 20 percent silica fume patching, the corrosion current will be about 3 times less than if latex modified concrete is used. This will extend the time until corrosion induced cracking of the steel in the remaining salty concrete, but will not, of course, completely stop it; because, self corrosion of that steel and other macrocells (ex. bottom mat steel cathodes) continue to plaque the structure. In this test program, it was not possible to directly (i.e. visually) compare the times to cracking because the bars in salty concrete had undergone an unknown, but undoubtedly differing amount of corrosion prior to patching. Such was not necessary however, since it is known that the time to corrosion induced distress is proportional to macrocell current and will also increase the time to distress. This is shown in Figure 6 in which the cumulative macrocell current-time is plotted, for each patch material. The "cumulative macrocell current" is the area under a plot of corrosion current versus time (expressed as mA-yrs/sq ft), and is the measure which directly relates to corrosion, until concrete cracking. It is actually a current density-time relation and is expressed

per square foot of macroanode to allow comparison of data, for different specimen configurations. Other studies at KCC INC have shown that severe cracking of conventional concrete occurs at a cumulative macrocell current density-time of about 2 mA-yrs/sq ft. The latex modified concrete patch reached this value (on the average) after about 4 years. The silica fume concrete patch did not reach 2 mA-yrs/sq ft, during the 8.5 years of testing. Projecting forward, about 12 years would be required. This 8 years of increased life is quite significant. The silica fume patch material used in the above studies had 20 percent silica fume by weight of cement, 750 lbs (441 Kg/m^3) of cement per cubic yard, and a W/C+S ratio of 0.28. A prepackaged patch material was produced in 1986 and it was tested in the KCC INC laboratories. The mix contained dry uncompacted silica fume, superplasticizer and air entraining agent, plus dry sand, stone and portland cement. Only water was added when the material was mixed. The slump was 8 to 10 inches, (19.6 to 25.4 mm), the unit weight was 134.4 pcf and the air content was 5.7 percent. Rapid permeability and resistivity tests were performed on cylinders subjected to accelerated curing (autogenous container), to a limewater cure of 6 days and to a limewater cure of 28 days followed by 18 days of outdoor exposure (46 days of age). the average permeabilities and resistivities are presented below:

Cure	Charge Passed coulombs	Wet Resistivity ohm-cm
Accelerated	66	340,000
6 Day Limewater	938	29,400
45 Day (LW + Air)	354	64,000

The untested cylinders were placed under natural weathering exposure at the KCC INC outdoor facility in Virginia. After 5 years, they were retested for permeability and resistivity. The average charge passed was 59 coulombs and the average wet resistivity was 394,800 ohm-cm, after exposure to hundreds of freeze-thaw, wetting and drying, and heating and cooling cycles.

Field Projects; Silica Fume Concrete Patching:

Field projects involving patching of parking decks and other structures with silica fume concrete began in 1985 with work in Ohio, Illinois and Missouri. In all cases except the first, twenty percent silica fume admixture (dry silica fume and superplasticizer) was added by the bag to ready mix concrete on site. The mix design was typically 750 lbs/yd^3 (441 kg/m^3) of cement with 165 lbs/yd^3 (97 kg/m^3) of silica fume admixture. The findings from the quality assurance programs on select project are discussed below.

The Illinois project involved prepackaged patching material for use in night-shift patching of a high-rise office building (the lightweight concrete was placed with calcium chloride and thick sections covered with tile were badly delaminated, over the reinforcing steel). On one night shift, the floor covering and the deteriorated concrete were removed and the area was covered with plywood for use the next day. The next night the plywood was removed, the concrete prepared by wetting, the prepackaged material was combined with one gallon (3.8 L) of water in a small "wheelbarrow" rotary drum mixer, mixed and then placed, consolidated, finished and recovered with plastic and plywood for use of the area the following day. After 3 days of curing, the plywood was removed and carpet tiles were installed (again at night). Thus, the patching was completed without loss of space use to the tenants. A freight elevator was used as the staging area and for material and equipment storage during the day. Resistivity specimens made from prepackaged material and subjected to a similar curing sequence, exhibited a 21 day resistivity which averaged 313,500 ohm-cm. This project started in 1985 and is still an ongoing application. After 7 years, the owners are still using the material and report satisfactory performance from the patched concrete.

This 1986 project in Ohio involved ready-mix silica fume concrete for patching a parking deck. The 40 day (28 day moist and 12 day lab air cure) field cylinders exhibited an average charge passed of 164 coulombs, with a range of 87 to 242 coulombs. The average wet resistivity was 86,829 ohm-cm, with a range of 56,700 to 147,000 ohm-cm. One cylinder was made prior to the addition of silica fume to a mix. This specimen showed a charged passed of 1,696 coulombs and a wet resistivity of 11,550 ohm-cm. The patches have now been in service on this structure for about 5.5 years (a membrane was installed to prevent additional chloride ingress into the unpatched areas). The project engineering consultant, THP Limited, reports excellent performance with no additional repair required to date. They also report similar satisfactory performance on four other silica fume concrete parking garage patching efforts.

Silica fume parking garage patching was specified in Missouri by Structural Engineering Associates (SEA) in 1986 as a substitute for latex modified concrete. The procedures and mixes were similar to those used in Ohio and these decks were also covered with waterproof membranes. SEA engineers indicated in 1992 that they were pleased with the performance of the silica fume patches and believed they performed better than latex modified concrete in that less additional delamination developed with time, when silica fume patching was used. Such, of course, agrees with the findings of the long term outdoor exposure study described above.

FHWA Silica Fume Concrete Test Program:

Silica fume concrete overlays were included in the United States Federal Highwa Administration (FHWA) Time-to-Corrosion Rehabilitation studies in 1984.(4-6) The origin 20 sq ft (1.86 m²) Time-to-Corrosion slabs were cast, in 1971, as part of the FHW research program to combat premature bridge deck deterioration. The slabs were salte daily, with 3 percent sodium chloride solution, for several years and then monitore throughout the 1970's. At nine years of age, the 0.50 water-cement ratio slabs, with o inch (25.4 mm) cover, all exhibited corrosion damage in the form of rust staining, crackin delamination and spalling. The slabs originally contained only a top mat of reinforcing steel.

To facilitate use in rehabilitation studies, a bottom mat of steel in salt free concrete (W/C = 0.50) was added, as well as, select instrumentation (resistivity monitoring and rate of corrosion probes). All delaminations and spalls were patched with 0.50 W/C concrete or mortar and then, various rehabilitation overlays, membranes and sealers were placed on all slabs, except for select controls. The effect of the rehabilitations, on continued corrosion in the originally sound, but salty concrete, has been monitored. The measurements include macrocell corrosion current, half cell potential and resistance and temperature, as well as visual and delamination surveys. In 1980, the original rehabilitation series was initiated. This included latex modified concrete (cracked and uncracked), low W/C (0.32) conventional concrete, 0.45 W/C conventional concrete and internally sealed concrete overlays. Silica fume concrete overlays were added in 1984.

The silica fume (twenty percent by weight of cement) concrete was made using dry, uncompacted silica fume and superplasticizer. The cement content was 750 lbs/cy (441 kg/m) and a vinsol resin air entraining agent was used to provide a six percent entrained air. The coarse aggregate was Riverton limestone, 1/2 inch (12.7 mm) maximum size meeting AASHTO M43 size no. 7. The overlays were two inches (50.8 mm) thick and were placed, onto the damp concrete surface, after placement of a cement slurry bonding agent. The overlay concrete was consolidated using a vibrating screed, wood floated, broomed and cured for 3 days, using wet burlap and plastic. Two Time-to-Corrosion slabs were overlaid. Rapid permeability tests, at 35 days of age on cores (four total), from field cured companion slabs, averaged 520 coulombs and the resistivity averaged 41,000 ohm-cm. After 2.5 years of exposure at the FHWA outdoor test facility, in McLean, Virginia, the overlays were evaluated for soundness, permeability, resistivity and bond strength. Three cores containing overlay and base concrete were obtained from each slab. No delaminations, or overlay debonding had occurred, on either of the silica fume concrete slabs. No freeze-thaw or other damage was seen, except for very small, short shrinkage cracks which had formed during curing. The cracks had not increased in size or severity since the curing was removed 2.5 years earlier. The charge passed, averaged 92 coulombs with 5 of the 6 specimens yielding values of less than 100 coulombs. The average wet resistivity was 22,000 ohm-cm. The overlay bond was evaluated in shear. Shear breaks were preformed at the bond line, in the original base concrete, and in the silica fume concrete. The findings are summarized below.

Shear Plane	Shear Strength, psi (Mpa)	
	Average	Range
Base Concrete	1,075 (7.4)	942 to 1,250 (6.5-8.6)
Bond line	1,238 (8.5)	1,159 to 1,377 (8.0-9.5)
Silica Fume concrete	1,214 (8.4)	942 to 1,540 (6.4-10.6)

These data indicate that the overlay bond strength was excellent and greater than the shear strength of the underlying concrete.

Visual examinations, at 3.5 and 7.5 years, indicated that the overlays were in excellent condition. The small shrinkage cracks had not extended or enlarged and no freeze-thaw damage was present. Interestingly, during both surveys, the overlays on many of the other slabs (latex and conventional concrete) showed full length cracks over the top mat rebars in the original salty concrete, probably the result of continued corrosion; while the silica fume overlay slabs did not. Thus, although the silica fume concrete overlay does not halt corrosion of the steel in the underlying concrete, it does seem to physically resist the forces of continued corrosion for a longer time than that provided by the other overlay concretes which were tested.

CONCLUSIONS

1. Concretes made with 10 to 22 percent silica fume by weight of portland cement exhibit:

Very high strength

Negligible or very low permeability

Very high electrical resistivity

Excellent bond strength

Excellent freeze-thaw durability, and

provide excellent protection to embedded reinforcing steel in adverse salt environments. As these materials age they become even stronger, lower in permeability and higher in resistivity.

2. The use of concrete with 20 percent silica fume for patching structures where sound but salty concrete will remain in place, minimizes aggravation of the corrosion process and will result in longer term performance than the use of conventional or latex modified concretes.

3. Concretes (normal and lightweight), grouts, and shotcrete, which contain from 10 to 22 percent silica fume (by weight of cement) have been successfully used in both new construction and in rehabilitation efforts, for over 10 years in the United States. The resultant materials provide the best cementitious product available for adverse environments. Although some special construction precautions are required because of the very small amount of bleed water and the overall cohesiveness of the mix, these have not prevented implementations, from either technological or cost standpoints.

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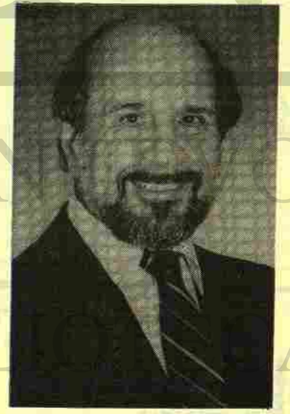

John Wolsiefer, Sr. is President of Norcem Concrete Products, Hauppauge, N.Y. U.S.A.

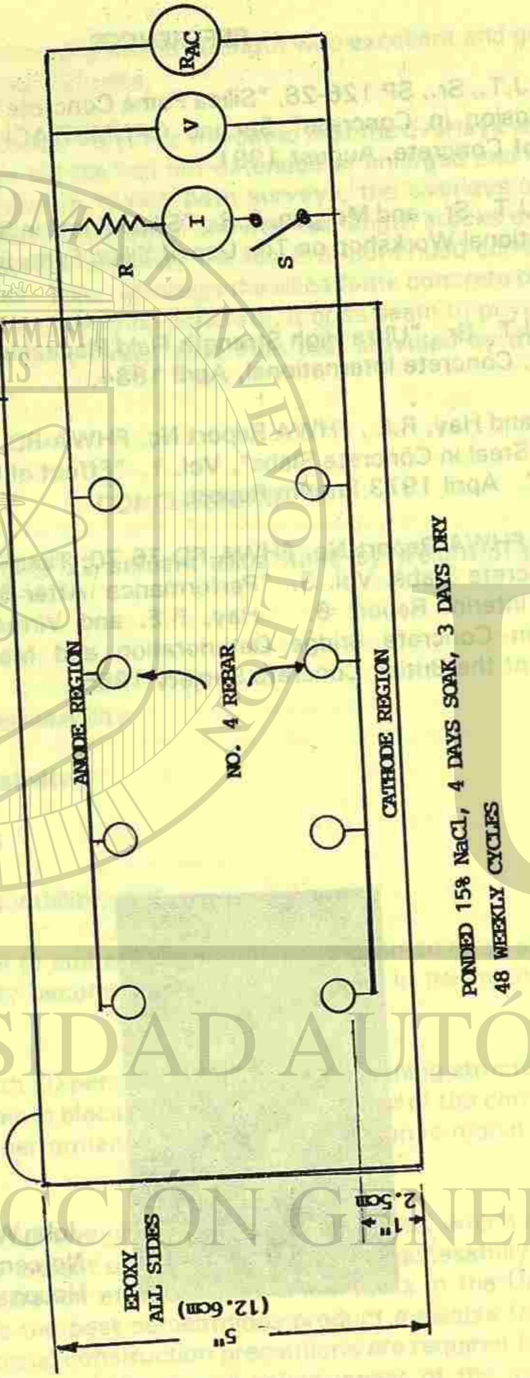

Fig. 1. Time-to-Corrosion NCHRP slab test southern exposure

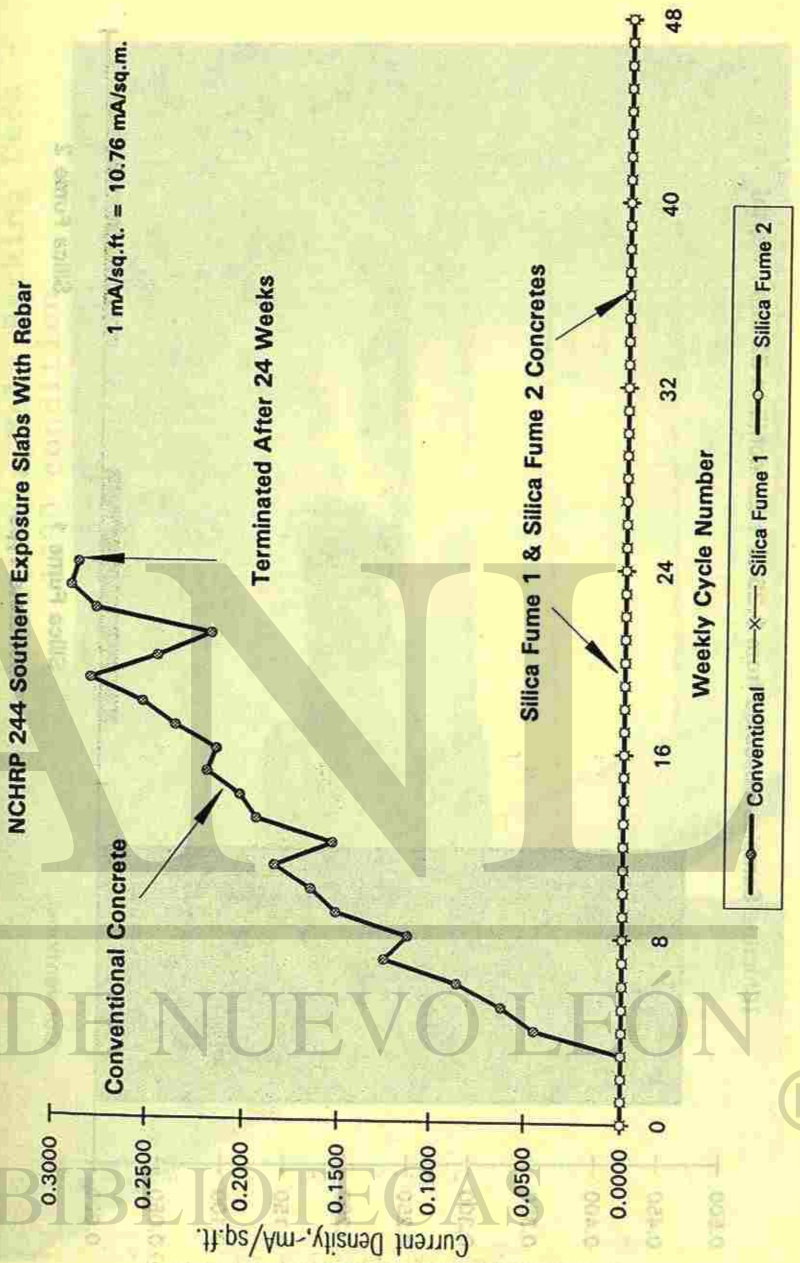

Fig 2. Macrocell Corrosion Current Density vs. Time

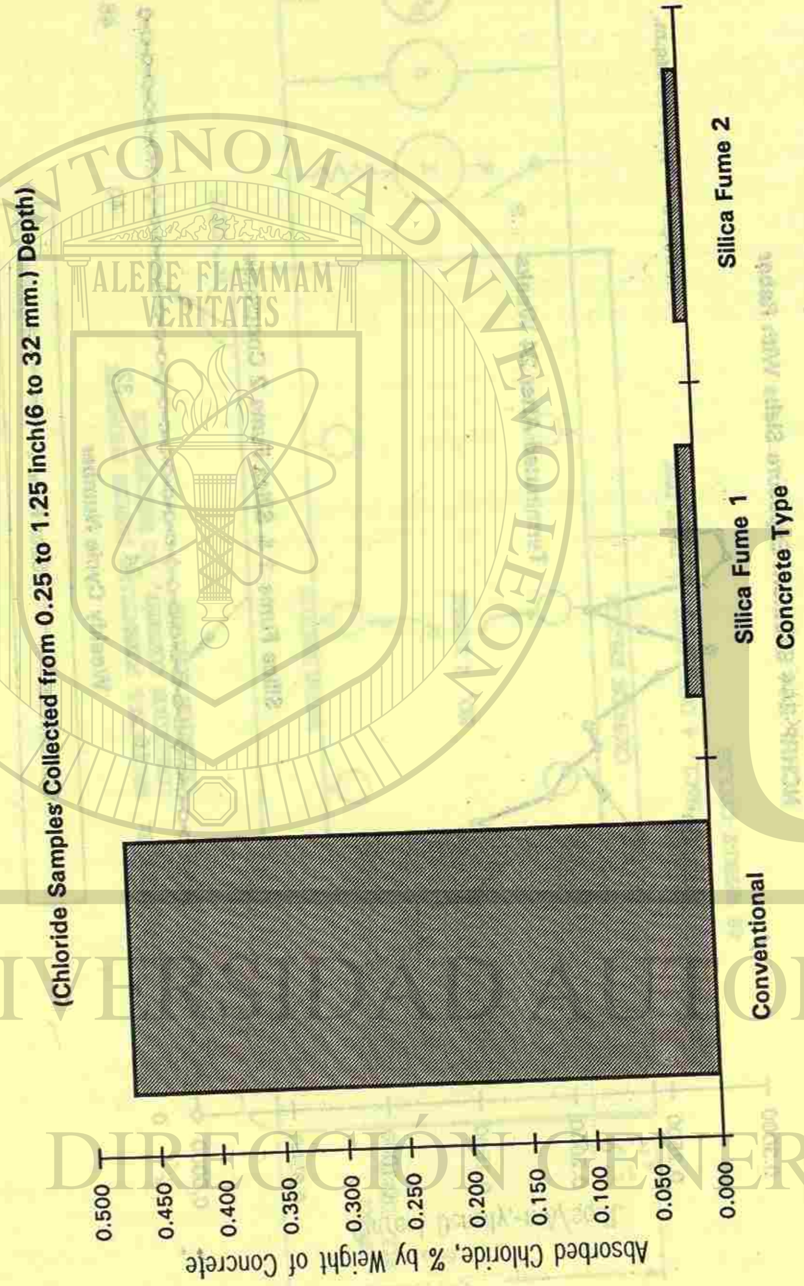

Fig. 3. Average Chloride Intrusion After 24 SE Cycles

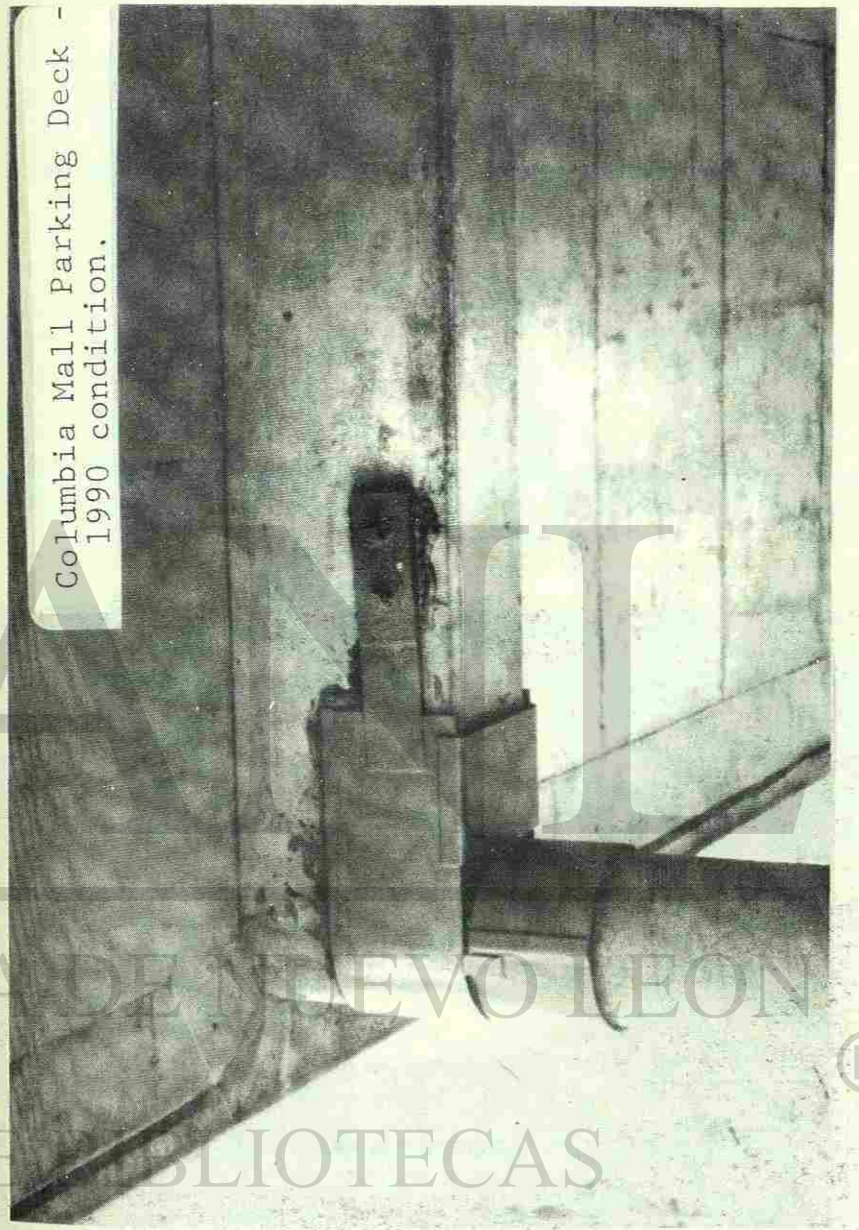
Columbia Mall Parking Deck - 1990 condition.

Fig. 4. Columbia Mall Beam/Column, Support Collar.

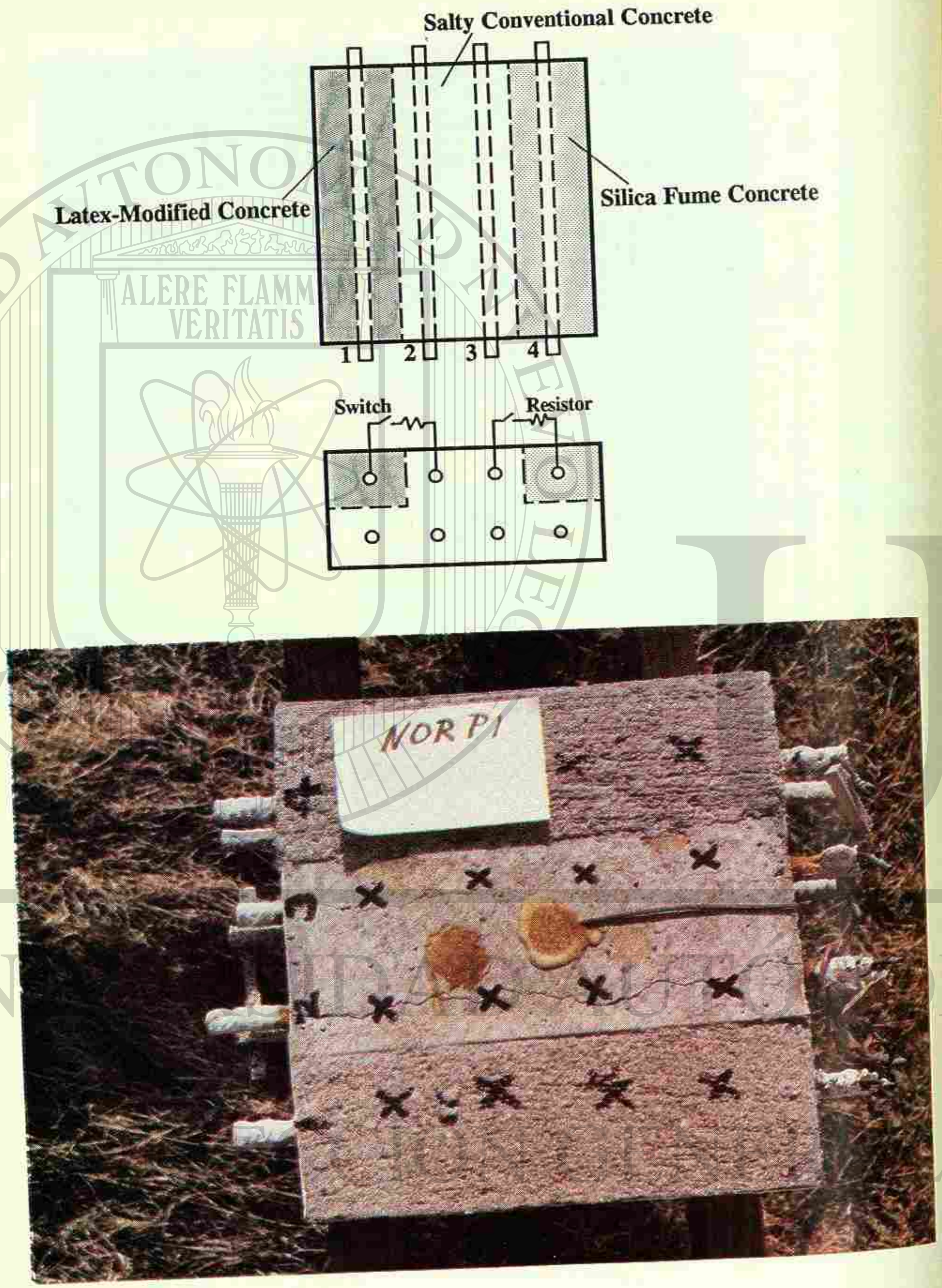

Fig. 5. Patching Study Specimen.

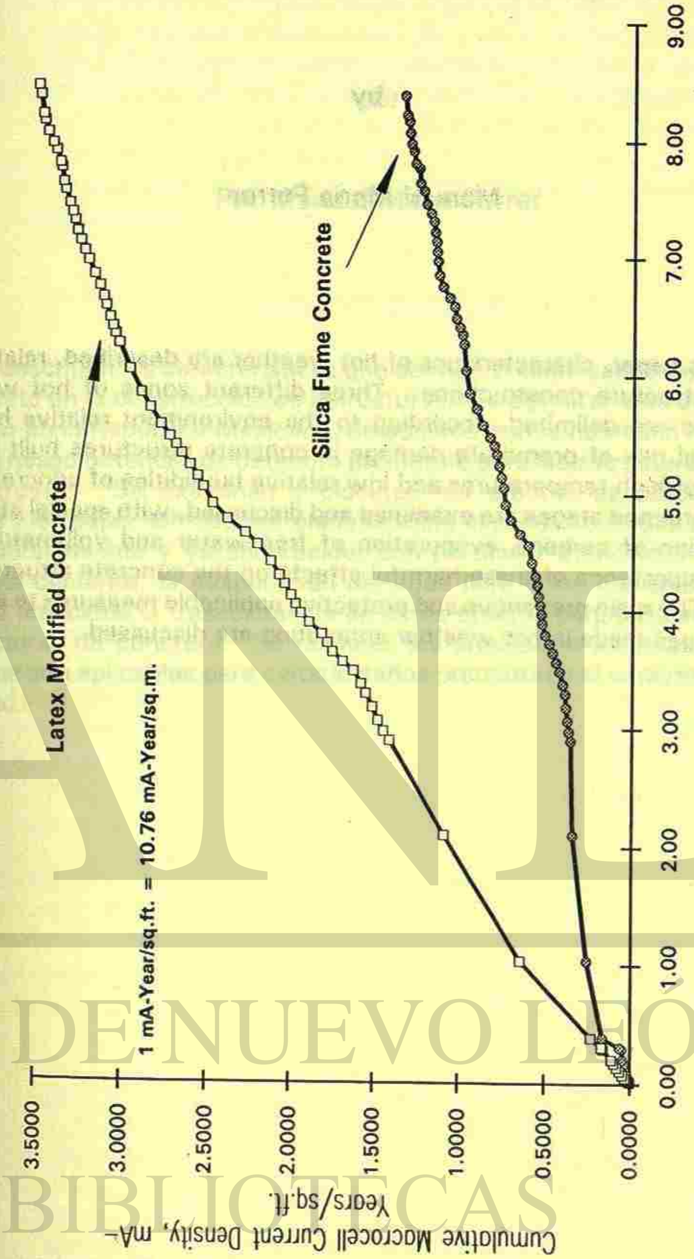

Fig. 6. Patching Material Test
Cumulative Macrocell Current Density vs. Time

CONCRETE IN HOT WEATHER: DURABILITY ASPECTS

by

Manuel Mena Ferrer

Synopsis: In this paper, characteristics of hot weather are described, related to the use of concrete in structure constructions. Three different zones of hot weather in the Mexican Republic are delimited, according to the environment relative humidity, with different potential risk of premature damage in concrete structures built in each zone. Harmful effects of high temperatures and low relative humidities of concrete in its fresh, hardening and hardened stages are examined and discussed, with special attention on the effect of hydration of cement, evaporation of free water and volumetric changes of concrete. The importance of these harmful effects on the concrete structures durability are mentioned. The main preventive and protective applicable measures to avoid concrete premature damages made in hot weather concreting are discussed.

Keywords: Hot weather, temperature, relative humidity, durability, hydration, evaporation, volumetric changes, curing, craking, maintenance.

CONCRETO EN CLIMA CALUROSO: ASPECTOS SOBRE LA DURABILIDAD

por

Por Manuel Mena Ferrer

Sinopsis: Se describen las características que definen el clima caluroso, en relación con el uso del concreto en la construcción de estructuras. Se delimitan tres diferentes zonas de clima caluroso en la República Mexicana, de acuerdo con la humedad relativa ambiental, con diferente riesgo potencial de deterioro prematuro para las estructuras de concreto que ahí se construyen. Se examinan y discuten los efectos perjudiciales de las altas temperaturas y las bajas humedades relativas sobre el concreto en sus estados fresco, en curso de endurecimiento y ya endurecido; con particular atención a los efectos en la hidratación del cemento, la evaporación del agua libre y los cambios volumétricos del concreto. Se menciona la trascendencia de estos efectos perjudiciales en la durabilidad de las estructuras de concreto. Se resumen las principales medidas preventivas y de protección que son aplicables para evitarle daños prematuros al concreto que se cuela en clima caluroso.

Palabras Clave: Clima caluroso, temperatura, humedad relativa, durabilidad, hidratación, evaporación, cambios volumétricos, curado, agrietamiento, mantenimiento.

Manuel Mena Ferrer ejerce actualmente como ingeniero consultor en tecnología del concreto, en la ciudad de México, D.F. Con anterioridad fue jefe de la Oficina de Materiales del Departamento de Estudios Experimentales de la Comisión Federal de Electricidad, e investigador en la Sección de Estructuras del Instituto de Ingeniería de la Universidad Nacional Autónoma de México.

ASPECTO GENERAL

Los primeros usos del concreto reforzado se remontan a las décadas finales del siglo 19, y algunas de esas primeras aplicaciones todavía permanecen en servicio.(1) Esto significa que dicho concreto, comparable con el convencional actual ha demostrado que puede durar en servicio útil más de 100 años. Sin embargo, numerosas estructuras de concreto construidas entonces y después, se deterioraron en el curso de pocos años y tuvieron que ser reconstruidas o reemplazadas; lo cual puso de manifiesto que la durabilidad del concreto no es una característica inmanente e invariable, sino que debe ser motivo de procuración en cada caso.

El tiempo que una estructura de concreto puede prestar satisfactoriamente el servicio para el que se construye, depende de numerosos factores entre los cuales destacan la adecuada selección y uso del concreto, y la correcta realización del diseño, la construcción y el mantenimiento de la estructura. Al considerar la selección y uso del concreto, se entiende que debe adecuarse a las condiciones específicas de cada obra, en lo que se refiere al medio ambiente, el medio de contacto y el tipo de servicio al que debe someterse el concreto en la estructura.

En lo relativo a las condiciones ambientales, es necesario tomar en cuenta no sólo las que prevalecen durante la etapa de construcción, sino también las que debe soportar la estructura en funciones. En el primer caso, el medio ambiente ejerce influencia sobre el concreto en sus estados fresco, recién fraguado y en curso de endurecimiento, en tanto que en el segundo caso actúa sobre el concreto ya endurecido.

En regiones donde ocurren variaciones extremosas del clima entre las temporadas estival e invernal, deben tomarse medidas para proteger el concreto contra ambas condiciones extremas, según el caso. Por ejemplo, si la estructura se construye durante el invierno, deben adoptarse las precauciones recomendables para colados en tiempo frío, y además deben prevenirse las medidas necesarias para proteger a la estructura en servicio de los efectos de la congelación en futuros inviernos. Si por el contrario se construye en verano, las precauciones de colado que deben aplicarse son las de tiempo caluroso, pero de todos modos debe preverse la protección del concreto endurecido contra el daño por congelación durante los períodos invernales de su vida de servicio.

En regiones donde las variaciones estacionales son amortiguadas por la posición geográfica, no suelen ocurrir grandes diferencias entre las condiciones climatológicas de verano e invierno, y por consiguiente tienden a prevalecer condiciones ambientales parecidas en las etapas de construcción y de servicio de las estructuras. De este modo en las zonas de clima tropical, casi en cualquier época del año en que se construye una estructura de concreto, son aplicables las recomendaciones para colados en tiempo caluroso, y en ningún caso se justifica tomar medidas para prevenir los efectos de la congelación en el concreto en servicio.

CLIMA CALUROSO

El clima lo determina el conjunto de fenómenos meteorológicos (insolación, precipitaciones pluviales, vientos, presión atmosférica) cuyas manifestaciones en un lugar determinado dependen principalmente de su altitud y latitud, y secundariamente de aspectos locales como su proximidad al mar, por ejemplo. De la participación proporcional de estos fenómenos resultan las condiciones ambientales, cuyas características más representativas son la temperatura y el grado de humedad.

Al examinar los límites en que varía normalmente la temperatura ambiental que se produce a lo largo del año en las regiones pobladas del planeta, se observan valores medios mensuales mínimos del orden de -15°C y máximos cercanos a 40°C, lo que implica valores mínimos y máximos absolutos aun más extremosos. En relación con el uso del concreto, esta amplitud de variación de la temperatura puede considerarse subdividida en tres intervalos que definen otras tantas condiciones térmicas ambientales de trabajo: a) intervalo de baja temperatura, en que deben aplicarse medidas para proteger el concreto fresco y endurecido contra el frío excesivo, b) intervalo de temperatura moderada, en las que no se requieren medidas especiales en este aspecto, para el uso del concreto en la construcción de estructuras ordinarias, y c) intervalo de altas temperaturas, durante cuya manifestación se recomiendan precauciones con objeto de prevenir los efectos perjudiciales del calor excesivo sobre el concreto fresco y recién colocado principalmente.

Las bajas temperaturas, a partir de las cuales debe protegerse el concreto contra los efectos detrimentales del frío, se hallan delimitadas con relativa precisión (2) pues básicamente se definen en términos de prevenir el riesgo de congelación del concreto recién colocado y en curso de endurecimiento. No existe sin embargo la misma precisión para definir el nivel en que comienzan a producirse las condiciones de alta temperatura, debido a la injerencia de otros factores. Así, en el informe ACI 305 (3) se define el clima caluroso, para fines del uso del concreto, como una "combinación de las siguientes condiciones, que tiende a perjudicar la calidad del concreto fresco o endurecido: a) alta temperatura del ambiente, b) alta temperatura del concreto, c) baja humedad relativa, d) velocidad del viento, e) radiación solar". Es pertinente observar que, en cuanto a la temperatura, no sólo es necesario considerar la del ambiente sino también la del concreto.

A causa del calor que se genera durante la hidratación del cemento, hay tendencia a que se incremente la temperatura del concreto después de colocarlo en las formas;

incremento que depende de aspectos tales como el tipo y consumo de cemento, los espesores de los miembros estructurales, y las facilidades para la disipación del calor interno. Debido a la gran variabilidad de estos aspectos, no es factible establecer un tope de carácter general para la temperatura de colocación del concreto, de modo que al incrementarse no rebase un cierto límite máximo considerado como perjudicial. Por tal motivo, cuando así se requiere, lo recomendable es que el nivel máximo permisible de temperatura del concreto al ser colocado se defina individualmente, de acuerdo con las condiciones específicas del caso. Sin embargo, a manera de orientación para la construcción de estructuras ordinarias (no voluminosas) en el USBR (4) suele especificarse una temperatura máxima de colocación del concreto igual a 27°C para trabajos en clima árido y caluroso, y 32°C cuando el clima es caluroso pero no seco.

Comúnmente los agregados poseen una temperatura más alta que la del ambiente, si permanecen expuestos al sol; el cemento puede suponerse a una temperatura entre la del ambiente y un valor más alto que depende de su antigüedad de fabricación; el agua, por su alto calor específico, tiende a mantenerse en un nivel de temperatura algo menor y más uniforme que el del medio ambiente, salvo que reciba directamente calor solar. De esta manera, si no se ejercen acciones en contrario, al hacer el balance de estas condiciones puede esperarse que la temperatura del concreto al ser mezclado tienda a ser, en promedio, algo mayor que la del medio ambiente. Es decir, para poder colocar el concreto a una temperatura máxima de 27 o de 32°C, sin adoptar medidas preventivas, es necesario que la temperatura ambiente sea menor de 27°C en clima seco y menor de 32°C en clima húmedo.

La razón básica para establecer distinción entre las temperaturas máximas permisibles en ambos tipos de clima, se debe a la disminución de la velocidad con que se evapora el agua a medida que es mayor la humedad relativa del ambiente; de este modo, para que se produzca una cierta velocidad de evaporación, es posible admitir una temperatura más alta conforme aumenta la humedad relativa. Se sabe (3, 5) que si el agua superficial del concreto recién colocado en las formas se evapora a una velocidad del orden de 1 kg/m^2/hr, existe el riesgo de que ese concreto se agriete por efecto de la contracción plástica, y que para producir esa velocidad crítica de evaporación en un ambiente con viento suave (16 km/hr) se requiere una humedad relativa de 40 por ciento si la temperatura es de 27°C, y de 60 por ciento cuando la temperatura es de 32°C.(3)

CONDICIONES LOCALES

La República Mexicana se localiza entre los 15 y 33 grados de latitud norte, de manera que el Trópico de Cáncer (23° 27') la divide en dos porciones aproximadamente iguales; de este modo, astronómicamente, a la porción sur le corresponde clima tropical y a la porción norte clima templado. Sin embargo, de acuerdo con las condiciones locales de altitud y distancia al mar, hay cierta tendencia a la definición de tres zonas que presentan diferente clima regional: zona A, que corresponde a la región centro-norte, en donde el medio ambiente suele ser seco, caluroso en verano y frío en invierno; zona B, que rodea la anterior pero con un clima menos extremoso, pues exhibe mayor grado de humedad y

más moderación en las variaciones estacionales de temperatura; y zona C, que comprende principalmente la región sursureste y la llanura costera del litoral del Golfo de México, en donde el clima es sensiblemente tropical, cálido y húmedo, con relativamente menos variaciones entre el verano y el invierno.

En la Fig 1, formada con datos de la Ref (6), se hace una delimitación tentativa de estas tres zonas climáticas, cuyas principales características de temperatura y humedad son aproximadamente como sigue:

Conceptos (Límites probables)	Zonas climáticas (A)	 (B)	 (C)
Temperatura mínima absoluta en invierno, °C	-22/-8	-12/0	0/12
Temperatura máxima absoluta en verano, °C	36/50	36/48	40/46
Humedad relativa media anual, %	30/40	40/60	60/80

De acuerdo con estos datos, es evidente que durante el verano se manifiestan de manera general elevadas temperaturas ambientales en la mayoría del territorio nacional, si bien bajo tres diferentes condiciones de humedad relativa. De esta manera, en relación con el uso del concreto durante la construcción de estructuras, las condiciones más desfavorables se presentan en la zona A, en donde se conjugan las altas temperaturas estivales con las bajas humedades relativas, lo cual origina situaciones propicias para que se incremente la velocidad de evaporación del agua, y por consiguiente para que se produzca una rápida desecación del concreto recién colocado. En la zona B las condiciones se presentan menos desfavorables, pues las temperaturas máximas probables tienden a disminuir, en tanto que las humedades relativas ambientales tienden a aumentar; pero aun así, continúan representando condiciones riesgosas para el concreto durante su utilización. Por último en la zona C la humedad relativa suele ser bastante alta, pero como la temperatura ambiental también lo es, subsiste el riesgo de que se conjunten condiciones propicias para la desecación prematura del concreto en un momento dado: por ejemplo, si la humedad relativa es de 70 por ciento, el riesgo de que se produzca la velocidad de evaporación crítica (1 kg/m^2/hr) ocurre cuando la temperatura del concreto es de 35°C (3), lo que no es difícil que suceda en esta zona si no se adoptan medidas para evitarlo.

EFECTOS PERJUDICIALES EN EL CONCRETO

El clima caluroso tal como se ha definido, ejerce acción perjudicial sobre el concreto principalmente en la etapa comprendida desde su elaboración hasta que adquiere su resistencia de proyecto; a diferencia del clima frío que puede causarle daño tanto al actuar en esta primera etapa como después durante su vida de servicio. En tal concepto, las medidas para proteger al concreto de los efectos adversos del clima caluroso, normalmente se circunscriben al período constructivo de las estructuras y el inmediato posterior.

Las altas temperaturas suelen afectar al concreto recién elaborado y en curso de fraguar y endurecer, debido principalmente a los efectos que dichas temperaturas producen sobre: 1) la hidratación del cemento, 2) la evaporación del agua, y 3) los cambios de volumen del concreto; cuyos efectos se interrelacionan y acumulan para converger en la merma de propiedades el concreto y el acortamiento de su duración potencial en servicio.

Hidratación del Cemento

El proceso de hidratación del cemento, como sucede en cualquier reacción química, se hace más rápido conforme aumenta la temperatura; lo cual se manifiesta por una mayor resistencia mecánica del concreto en sus primeras edades. Sin embargo, esta manifestación inicial no persiste, pues a edades posteriores la adquisición de la resistencia se vuelve más lenta, de modo que ya a la edad de proyecto el concreto hidratado en alta temperatura registra menos resistencia a compresión que el mismo concreto hidratado en baja temperatura, según se indica en la Fig 2. (7)

La justificación de que así ocurra, se muestra en la Fig 3 (8) que representa esquemáticamente la forma como se hidrata una partícula de cemento a temperatura baja (13°C) y a temperatura alta (49°C). Como ahí se hace notar, la hidratación a temperatura elevada conduce a la formación de una especie de costra alrededor de la partícula, que inhibe el acceso del agua hacia el núcleo de la misma para su ulterior hidratación, con lo cual esta última no se alcanza a completar aunque exista suficiente humedad en el concreto. De esta manera, se origina un residuo de cemento que permanece sin hidratar y que no aporta resistencia al concreto, lo que no solamente representa una deficiencia técnica sino también un inconveniente económico.

Este desarrollo anómalo del proceso de hidratación del cemento es atribuible básicamente al efecto de las altas temperaturas, ya que inclusive se manifiesta en presencia de agua; de modo que su manifestación adversa a la resistencia mecánica del concreto, que es irreversible, puede ocurrir dondequiera que el concreto se produzca y endurezca a temperatura elevada, independientemente del grado de humedad que prevalezca en el medio de exposición. Es decir, se trata de una consecuencia perjudicial del clima caluroso que puede manifestarse en cualquiera de las tres zonas del territorio nacional, cuando existen altas temperaturas ambientales.

Para dar una idea hasta dónde puede tolerarse la elevación de la temperatura de hidratación sin afectar demasiado la resistencia del concreto, en la Fig 4 formada con datos de la Ref (7), se presenta la forma como usualmente varía la resistencia a compresión del concreto a diversas edades, conforme aumenta la temperatura de curado. Según ahí se hace notar, para una temperatura de 32°C (en condiciones de curado húmedo) la resistencia del concreto a 28 días sólo se reduce alrededor de 5 por ciento con respecto a la que se obtiene en curado estándar a 23°C; lo cual se juzga tolerable y parece apoyar

el criterio de considerar 32°C como temperatura máxima admisible para el endurecimiento del concreto en ambiente húmedo.

Es pertinente señalar que la aceleración del proceso de hidratación del cemento con el aumento de la temperatura, también se refleja en una sensible disminución del tiempo de fraguado, que puede ocasionar dificultades para la adecuada colocación y compactación del concreto en las formas. Sin embargo, a diferencia de la afectación de resistencia que no es corregible, la evolución del fraguado sí es susceptible de modificación y ajuste mediante aditivos. Es por ello que para colados en tiempo caluroso suele recomendarse el uso de aditivos retardadores del fraguado, en forma tal que pueda conciliarse la duración del fraguado inicial del concreto a la temperatura de la obra con el tiempo requerido para su transporte, colocación y compactación en la estructura.

Evaporación del Agua

La excesiva pérdida anticipada del agua del concreto, puede ocasionarle perjuicios desde que se elabora hasta que se encuentra en servicio, por cuyo motivo las condiciones de exposición que estimulan la pérdida de agua por evaporación, resultan indeseables. Tal como se ha dicho, el ambiente caluroso en que se conjugan temperaturas altas, humedades relativas bajas y presencia de viento, representa las peores condiciones en este aspecto. Los efectos perjudiciales que la intensa evaporación del agua produce en las sucesivas etapas de la vida del concreto, se describen a continuación.

Durante la elaboración del concreto y en el curso de su manipulación para transportarlo, colocarlo y compactarlo, se presentan las primeras manifestaciones inconvenientes del clima caluroso, bajo el aspecto de la evaporación del agua. En primer término, al aumentar la temperatura ambiental se incrementa la demanda de agua de mezclado del concreto, con sus correspondientes consecuencias adversas sobre la resistencia mecánica, la permeabilidad y los cambios volumétricos; esto es, si el concreto requiere más agua y con ello se incrementa la relación agua/cemento, puede esperarse que manifieste menos resistencia, más permeabilidad y mayor contracción por secado, todo lo cual repercute adversamente en su durabilidad. En segundo término, en dichas condiciones ambientales las mezclas de concreto recién elaboradas pierden revenimiento con mayor rapidez, lo que puede conducir a defectos de construcción por la dificultad de colocarlas correctamente en la estructura; se sabe que los defectos de construcción suelen dar pie a la degradación anticipada del concreto en servicio.

Al término de su compactación y acabado, si el concreto durante la etapa de fraguado pierde agua por evaporación con demasiada rapidez (del orden de 1 kg/m²/hr), experimenta contracciones causantes de esfuerzos de tensión que es incapaz de resistir en esta etapa, con lo cual sufre agrietamientos por "contracción plástica". Este fenómeno detrimental es particularmente probable en los pavimentos de concreto recién colados, por su gran superficie expuesta a la evaporación, y es fuente potencial del deterioro prematuro de las estructuras en que ocurre, cuando dichos agrietamientos no se corrigen oportunamente.

La continua presencia de humedad suficiente en el concreto durante sus primeras edades de endurecimiento es una condición esencial, e insustituible, para que el proceso de hidratación del cemento se desarrolle con normalidad, y por consiguiente para que el concreto pueda adquirir todas sus propiedades potenciales. La falta de agua suficiente para que el cemento se hidrate en forma sostenida, es por sí misma una condición detrimental que no requiere necesariamente acompañarse de una elevada temperatura para ocasionarle perjuicio al concreto. La magnitud de este perjuicio se pone de manifiesto en la Fig 5 (9) en donde se representa la adquisición de resistencia a compresión del concreto con la edad, en diferentes condiciones de humedad pero sin la influencia de la temperatura. Como en dicha figura se observa, a medida que el concreto padece más deficiencia de humedad se manifiesta mayor detrimento en su resistencia; si bien es pertinente mencionar que este efecto detrimental no es completamente irreversible (como el que produce la excesiva temperatura), pues una aportación tardía de agua externa que reponga la humedad necesaria en el concreto, puede permitirle una cierta recuperación de la resistencia potencial no adquirida por deficiencia de agua.

De acuerdo con la manera de producirse el efecto anterior, puede considerarse que la deficiencia de agua de hidratación es una situación igualmente perjudicial para el concreto que se emplea durante clima caliente o clima frío; sin embargo, también debe considerarse que el riesgo de que se produzca esa deficiencia de agua se incrementa en la medida que la temperatura es más alta y la humedad relativa es más baja, porque en estas condiciones la evaporación del agua es más rápida. Es decir, el requerimiento de conservar el concreto húmedo en sus primeras edades, es igualmente necesario durante los trabajos en verano y en invierno, pero las medidas para lograrlo requieren ser más oportunas y eficaces en un medio ambiente estival caluroso y seco, como suele ocurrir en las zonas climáticas A y B del territorio nacional, previamente señaladas.

La conservación del concreto en condición húmeda durante un cierto lapso posterior al colado, constituye el tratamiento de curado húmedo que deben recibir todas las estructuras recién construidas, y para cuya realización existen diversos procedimientos, materiales y productos que se hallan ampliamente descritos en el informe del Comité ACI 308. (10) Aunque la ejecución de este tratamiento es un requisito que se contempla normalmente en las especificaciones de construcción, en la práctica es una actividad que con frecuencia sólo se cumple a medias, e incluso no se cumple, para lo cual existen variadas explicaciones posibles: negligencia de quien construye y/o de quien supervisa, inaccesibilidad de la estructura, procedimientos ineficientes, materiales y/o productos de mala calidad, etc.

No obstante que la realización de un buen curado húmedo es una necesidad reconocida desde los primeros usos del concreto, ha sido en épocas recientes cuando se ha demostrado la influencia que ejerce no solamente en la resistencia mecánica, sino también en prácticamente todas las propiedades del concreto endurecido que de alguna manera tienen injerencia en su durabilidad. Debido a la mencionada frecuencia con que se producen omisiones y deficiencias en la realización del curado húmedo, y por el hecho de que estas deficiencias se agudizan en condiciones de clima caluroso, es pertinente destacar los efectos detrimentales que produce en el concreto la falta de humedad en sus primeras edades, a fin de darle al curado húmedo la importancia que merece.

Se sabe que para la completa hidratación de la pasta de cemento, se requiere una proporción de agua del orden del 25 por ciento del peso del cemento que contiene, en tanto que para mezclar el concreto se utilizan cantidades de agua que normalmente representan entre el 35 y 75 por ciento del cemento, en peso; es decir, en el momento de elaborar el concreto siempre existe agua en exceso de la que el cemento estrictamente necesita para hidratarse. Sin embargo, a partir de su elaboración el concreto comienza a perder agua por diversas causas (Fig 6) de las cuales prácticamente todas, excepto la evaporación, cesan de actuar una vez que el concreto adquiere su fraguado final. De esta manera, si la pérdida por evaporación subsiste en el concreto recién fraguado, al cabo de algún tiempo la proporción de agua remanente se reduce a menos de la que el cemento requiere para continuar su hidratación y ésta se frena, e incluso se suspende por falta de agua.

La pérdida de agua por evaporación se produce principalmente por las superficies de la estructura expuestas al medio ambiente, de modo que el perjuicio por este concepto es máximo en dichas superficies, con tendencia a disminuir hacia el interior del concreto. Se dice (11) que normalmente la pérdida de agua por evaporación solamente afecta una capa superficial de concreto de 5 a 10 cm de espesor, y que más adentro el concreto conserva suficiente humedad (más de 80 por ciento) para su hidratación. Es por ello que este efecto detrimental no se detecta cuando se comprueba la calidad del concreto colocado en la estructura mediante la determinación de la resistencia en núcleos extraídos a mayor profundidad. Sin embargo, hay que tomar en cuenta que la capa de concreto superficial es la que de ordinario resulta expuesta a las acciones perjudiciales que ponen en riesgo la durabilidad de las estructuras.

Además de la resistencia mecánica en sus diversos aspectos, la permeabilidad del concreto es la propiedad que más influencia tiene en la durabilidad de las estructuras. La resistencia mecánica no sólo es índice de la capacidad del concreto para soportar esfuerzos, sino también para resistir otras acciones físicas como la de abrasión mecánica e hidráulica, por ejemplo. Por su parte, la permeabilidad representa el grado de resistencia que el concreto opone a la penetración de fluidos agresivos (líquidos y gaseosos) que actúan como agentes promotores del deterioro prematuro del concreto y del acero de refuerzo. Debido a que ambas propiedades tienen como parámetro común la porosidad del concreto, resultan igualmente afectadas cuando la hidratación del cemento no se realiza cabalmente por deficiencia de humedad.

Para conservar el concreto con suficiente humedad durante sus primeras edades en que la hidratación evoluciona con mayor intensidad, existen dos procedimientos básicos aplicables en las superficies de la estructura expuestas al ambiente: 1) aportación de agua externa, para mantener continuamente húmedas las superficies expuestas a la evaporación, y 2) interposición de un elemento impermeable entre el concreto y el medio ambiente, para prevenir la fuga del agua interna por evaporación hacia el exterior. En teoría el primer procedimiento es más eficaz cuando se asegura su continuidad, y por ello se recomienda darle preferencia para el curado húmedo de concretos hechos con bajas relaciones agua/cemento (menos de 0.45); sin embargo, el impedimento de asegurar su continua aplicación en superficies verticales o de difícil acceso, o sobre superficies de concreto recién colocado, ha favorecido el uso del segundo procedimiento, ya sea mediante la utilización de telas plásticas o de compuestos líquidos que forman membrana.

En el caso de pisos y pavimentos de concreto hidráulico, y otros revestimientos similares, su durabilidad depende en gran medida de que la capa superficial de concreto posea una adecuada resistencia a la abrasión; pero debido a su gran superficie expuesta por unidad de volumen, son estructuras proclives a perder mayor cantidad de agua por evaporación, particularmente cuando se cuelan en un ambiente cálido, seco y con viento. Por tal motivo, la ejecución de un buen curado resulta de vital importancia en estos casos, para mantener húmedo el concreto recién colocado y así evitar que sufra agrietamientos por contracción plástica y la capa superficial se vea reducida en su resistencia mecánica, especialmente a la abrasión. La elección del procedimiento apropiado para curar estas estructuras requiere tomar en cuenta su oportunidad y eficacia, es decir, que pueda aplicarse a la mayor brevedad posible después del acabado superficial del concreto y que resulte efectivo para restringir la pérdida de agua por evaporación.

En una investigación de campo para definir el procedimiento de curado que debería aplicarse al revestimiento de concreto de los canales vertedores de una presa en el Estado de Chiapas (12) se ensayaron seis sistemas de curado cuyos efectos sobre la resistencia a compresión del concreto superficial se indican en la Fig 7 (12). Con base en estos resultados, el sistema elegido consistió en la aplicación inmediata de una membrana de curado sobre el concreto recién acabado (para cumplir con el aspecto de oportunidad seguido por la colocación de una tela de polietileno al cabo del fraguado final (para complementar el requisito de eficacia). Procede hacer notar que en estas pruebas se juzgó la eficacia del curado mediante la resistencia a compresión del concreto superficial, al considerar ésta como índice de sus propiedades mecánicas en general; cuya inferencia también resulta válida para la resistencia a la abrasión, según se observa en la Fig 8 (13 que representa la mejoría que se obtiene en este aspecto cuando el concreto se cura convenientemente.

En cuanto a la permeabilidad, también existen numerosas evidencias del beneficio que se obtiene en este aspecto en el concreto superficial de las estructuras como consecuencia de un buen curado húmedo. A manera de ejemplo, en la Fig 9 (14) se compara la permeabilidad superficial del concreto cuando se expone al ambiente sin protección o humedecimiento y cuando se conserva continuamente húmedo; cuyo efecto positivo reafirma la importancia que tiene el curado para la durabilidad de las estructuras de concreto de todo tipo que deben prestar servicio en contacto con fluidos potencialmente agresivos al concreto y al acero de refuerzo, como es el caso de las que tienen contacto con aguas freáticas o suelos con alto contenido de sulfatos y/o cloruros, y de las que permanecen expuestas al agua de mar y sus salpicaduras, o simplemente al aire marino, ya que el efecto dañino de estos fluidos depende en buena medida de su grado de penetración en el concreto superficial.

Cambios Volumétricos

El concreto experimenta normalmente cambios volumétricos de diversa índole, desde que se encuentra recién colocado en la estructura hasta el final de su vida de servicio. Dichos cambios, que pueden ser predecibles, no suelen afectar la durabilidad de las estructuras cuando son moderados y se toman adecuadamente en cuenta en el diseño estructural; sin embargo, cuando se trata de contracciones que resultan mayores de lo previsto, y/o no se previenen las medidas estructurales necesarias, pueden ocurrir agrietamientos que con cierta frecuencia se convierten en puntos débiles por donde se inicia el deterioro de las estructuras de concreto.

Las causas de los principales cambios volumétricos del concreto son sus variaciones de humedad y temperatura, de modo que al referirse en particular a las contracciones, éstas se relacionan con la disminución de la humedad y el descenso de la temperatura. En tal concepto, las condiciones ambientales más desfavorables son las que propician la evaporación del agua, como sucede en los sitios de clima caluroso y seco, y las que manifiestan grandes intervalos de variación entre las temperaturas máximas y mínimas.

La primera contracción importante del concreto es la que ocurre en el curso de la etapa de fraguado, que como se ha dicho se denomina contracción plástica y se acrecienta cuando el concreto recién colocado en las formas pierde agua por evaporación con demasiada rapidez. Se ha dicho también que la velocidad de evaporación se torna crítica cuando se aproxima a 1 kg/m^2/hr porque la contracción plástica alcanza magnitudes que fácilmente agrietan el concreto, ya que éste carece en esta etapa de capacidad para resistir los esfuerzos de tensión que se generan. Tales agrietamientos pueden evitarse normalmente con el uso de la medidas precautorias que más adelante se mencionan; pero cuando se producen y son detectados antes del fraguado final, pueden ser corregidos mediante la recompactación por apisonamiento enérgico del concreto en la superficie agrietada, cuidando de restablecer su integridad.

En la etapa de endurecimiento, que propiamente se inicia a partir del fraguado, el concreto sigue contrayéndose en la medida que continúa disminuyendo su contenido de humedad. Esta contracción que se conoce como contracción por secado, tiene una evolución relativamente rápida durante las primeras semanas, y después se hace más lenta pero continúa manifestándose en el curso del tiempo, incluso al cabo de 30 años, mientras el concreto permanece seco. Dicha contracción por secado, que es causa frecuente del agrietamiento de numerosas estructuras, depende significativamente de las características de los agregados y del contenido de pasta de cemento en el concreto, pero también puede ser incrementada por aspectos inherentes al clima caluroso, tales como el contenido de agua de mezclado del concreto y la humedad relativa del medio ambiente en que presta servicio la estructura.

En el primer aspecto, hay que tomar en cuenta que la demanda de agua de mezclado del concreto tiende a incrementarse en clima caluroso, por lo cual resulta útil toda medida que permita contrarrestar esa sobredemanda de agua en el mezclado de concreto, como puede ser por ejemplo el uso de aditivos reductores de agua, y en particular los llamados reductores de alta eficiencia (superfluidificantes) que permiten lograr reducciones hasta de 30 por ciento en el requerimiento de agua de mezcla. En cuanto al segundo aspecto, es un hecho evidente que al disminuir la humedad relativa del ambiente se intensifica la pérdida de agua por evaporación del concreto, y esto repercute en la magnitud de su contracción por secado según se indica en la Fig 10 (15). Para moderar los efectos que en este aspecto producen las condiciones ambientales cálidas y secas, también ejerce influencia positiva la ejecución de un buen procedimiento de curado húmedo, conforme se hace notar en la Fig 11 (16). Aquí es oportuno señalar que una medida importante para reducir los agrietamientos debidos a la contracción por secado (y por los cambios volumétricos en general) consiste en definir juntas de servicio adecuadas (contracción y/o expansión) en las estructuras, para lo cual dicha contracción debe evaluarse conforme a los métodos de predicción de uso aceptado, como por ejemplo el ACI 209 (17).

La contracción debida al primer descenso de temperatura del concreto se produce en el curso del tiempo que el concreto de la estructura tarda en igualar su temperatura con la del medio ambiente; y su magnitud depende básicamente del coeficiente de expansión térmica del concreto y de la diferencia que existe entre la temperatura máxima del concreto y la del ambiente. El coeficiente de expansión térmica del concreto lo determina principalmente la naturaleza de los agregados que lo constituyen, de modo que en un caso dado no es un factor susceptible de maniobra; por consiguiente, para moderar la contracción de origen térmico el recurso básico consiste en restringir la sobreelevación de temperatura del concreto en la estructura.

La temperatura máxima del concreto después de ser colocado en estructuras ordinarias (no voluminosas) se produce en el término de pocos días (Fig 12) y su magnitud es consecuencia de diversos factores entre los que destacan: la temperatura inicial del concreto, el tipo y consumo unitario de cemento, y las condiciones de acumulación y disipación de calor que son propias de cada estructura. De estos factores el que propiamente guarda relación con el medio ambiente se refiere a la temperatura inicial del concreto, de manera que la disminución de ésta es un medio adecuado para reducir el intervalo en que debe descender la temperatura, según se muestra en la Fig 12, y por consiguiente para reducir también la magnitud de la contracción térmica y su correspondiente riesgo de agrietamiento. Para disminuir la temperatura inicial de colocación del concreto en clima caluroso existen diversas medidas aplicables que se hallan descritas con detalle en el informe del Comité ACI 305 (3) y de las cuales se hace un resumen más adelante.

De igual manera que en el caso de la contracción por secado, una medida necesaria para prevenir los agrietamientos por contracción térmica consiste en la adecuada definición de juntas en la estructura que permitan absorber los cambios volumétricos de origen térmico: tanto la primera contracción debida al enfriamiento inicial del concreto a edad temprana, como las expansiones y contracciones subsecuentes que se producen en el curso de la vida de servicio de la estructuras por efecto de las variaciones de temperatura ambiental, particularmente las que se localizan en regiones como la zona A del territorio nacional en que los cambios estacionales son extremosos, para cuyo objeto debe hacerse la correspondiente evaluación anticipada de dichos cambios volumétricos de origen térmico

(17). Es asimismo pertinente señalar que las juntas de servicio no solamente deben ser motivo de una adecuada localización y diseño, sino también de una apropiada construcción y mantenimiento, pues de no ser así pueden convertirse en sitios localizados de debilidad en la estructura, especialmente susceptibles al deterioro prematuro.

MEDIDAS PREVENTIVAS Y CORRECTIVAS

Las medidas para prevenir y corregir los efectos perjudiciales del clima caluroso sobre el concreto, y sus correspondientes repercusiones adversas a la durabilidad de las estructuras, deben adoptarse en las etapas de diseño y construcción y complementarse en la etapa de servicio con un adecuado mantenimiento. La aplicación de estas medidas, cuya eficacia guarda cierta relación con su grado de dificultad y costo, se define normalmente en función de las características e importancia de la estructuras y de las condiciones climáticas en que éstas deben construirse y prestar servicio. Muchas de estas medidas van encaminadas principalmente a evitar o por lo menos restringir los agrietamientos del concreto, ya que éstos suelen dar motivo al deterioro anticipado de la estructuras (18).

Previsiones de Diseño Estructural

En las estructuras que se justifique, debe preverse la instalación de juntas de servicio para absorber en la mayor medida posible las contracciones y/o expansiones del concreto, debidas a las variaciones de humedad y temperatura; sin menoscabo del acero de refuerzo necesario para tomar los esfuerzos inducidos por estos cambios volumétricos. (Conviene prever el refuerzo adecuado para distribuir las grietas, a modo de no exceder en éstas una abertura máxima especificada). Para tal fin deben estimarse adecuadamente dichos cambios de volumen, de acuerdo con las características de la estructura (dimensiones, forma, espesores), del concreto (naturaleza de los agregados, tipo de cemento, proporciones) y del clima y ambiente de servicio (temperatura, humedad relativa), conforme a los procedimientos y criterios usuales (17).

Diseño de la Mezcla de Concreto

El diseño de la mezcla de concreto no sólo debe ser adecuado para lograr las propiedades especificadas en el concreto endurecido, sino también para obtener un adecuado comportamiento del concreto en estado fresco y en curso de endurecimiento, mediante una acertada selección de sus componentes. Dado que la fuente de suministro de los agregados frecuentemente no es opcional, las posibilidades selectivas se refieren principalmente al cemento y los aditivos.

Es deseable emplear preferentemente un cemento portland tipo II, porque su moderado calor de hidratación resulta útil para restringir la sobreelevación de temperatura del concreto en sus primeras edades y su correspondiente cambio volumétrico. Una alternativa viable para el mismo fin consiste en utilizar un cemento portland-puzolana; pero como hay cementos de esta clase que incrementan la demanda de agua de mezclado y/o aceleran la pérdida de revenimiento del concreto recién mezclado, es recomendable efectuar pruebas al concreto en condiciones de obra con el cemento propuesto. Otra condición que debe evitarse es el uso de un cemento con falso fraguado.

Los aditivos para concreto de uso más frecuente en clima caluroso son los que tienen la doble función de ser reductores del agua de mezcla y retardadores del fraguado. El primer efecto tiene utilidad para compensar el incremento en la demanda de agua que ocurre al mezclar el concreto en tiempo caluroso, en tanto que el segundo suele ser necesario para contrarrestar la aceleración del fraguado que se produce al aumentar la temperatura. Existen dos categorías de aditivos de esta índole: los reductores de agua normales y los de alta eficiencia; actualmente hay la tendencia a preferir los segundos, no obstante que su costo unitario es más alto. En cualquier caso es recomendable verificar en obra el comportamiento de la mezcla de concreto con el cemento y el aditivo propuestos, pues hay cementos y aditivos que no son compatibles porque generan reacciones inmediatas que rigidizan rápidamente el concreto recién mezclado.

La mezcla de concreto debe diseñarse con el mínimo consumo de cemento que sea posible, con el fin de reducir los cambios volumétricos por sobreelevación de temperatura y por secado. Si la estructura se construye en donde ocurren muy bajas temperaturas invernales (como hay sitios en la zona climática A del territorio nacional) debe prevenirse el uso de un aditivo inclusor de aire, no obstante que la construcción se efectúe en condiciones de clima caluroso.

Elaboración del Concreto

Conforme se ha dicho, la temperatura inicial del concreto es uno de los factores que determina la temperatura máxima que alcanza el concreto en la estructura. Por tal motivo, para moderar esta sobreelevación de temperatura, un medio eficaz consiste en disminuir al mínimo posible la temperatura inicial del concreto, lo cual en clima caluroso requiere la adopción de una serie de medidas con diverso grado de eficacia, cuya aplicación depende de la diferencia entre la temperatura natural del concreto sin el uso de estas medidas y el nivel de temperatura que se pretende darle. También se ha mencionado que no hay un tope único para la temperatura inicial de colocación del concreto porque deben tomarse en cuenta las condiciones de generación, acumulación y disipación de calor de cada estructura en particular. Así, en términos generales, para estructuras voluminosas los requerimientos son muy estrictos, por lo que se especifican temperaturas máximas de colocación del concreto (en cualquier clima) que suelen variar entre 12 y 24°C, aproximadamente, de acuerdo con su espesor y otros aspectos. Para estructuras no voluminosas, pero de grandes dimensiones y/o con importantes requisitos de servicio, es frecuente cuando se construyen en clima caluroso especificar para la temperatura de colocación del concreto un valor máximo comprendido entre 24 y 32°C, aproximadamente, según el caso. Finalmente en estructuras menores y/o de poca importancia no es común que se especifique una temperatura máxima permisible de colocación del concreto, aun cuando es deseable no sobrepasar los 38°C durante tiempo caluroso.

Para conseguir que la temperatura del concreto recién mezclado no rebase el límite máximo que se especifique, cuando la temperatura ambiente es del mismo orden o mayor que éste, existen diversas medidas aplicables que se mencionan a continuación, cuya complementación debe intentarse gradualmente a fin de obtener la temperatura requerida al menor costo posible:

En los componentes--Regar continuamente con agua por aspersión el agregado grueso, para mantenerlo superficialmente húmedo. Proteger con aislamiento térmico el depósito y las tuberías de circulación del agua para el mezclado del concreto. No utilizar cemento caliente (no más de 60°C, aproximadamente). Proteger del sol los depósitos de agregados para uso inmediato, adjuntos a la planta de concreto.

Al elaborar el concreto--Efectuar los colados en las horas de menor temperatura ambiental (de ser posible por la noche). Pintar de blanco exteriormente la revolvedora central y proteger del sol el sitio de mezclado. Evitar el exceso de tiempo de mezclado. Utilizar hielo en escamas o molido en substitución de una fracción del agua neta de mezclado, o bien utilizar en su totalidad agua preenfriada como agua de mezcla, o bien inyectar nitrógeno líquido en la revolvedora en movimiento para enfriar la mezcla de concreto en curso de elaboración.

Transporte, Colocación, Compactación y Acabado

Durante colados en clima caluroso, estas operaciones requieren ser planeadas cuidadosamente debido a la influencia adversa que estas condiciones climáticas ejercen sobre la pérdida de revenimiento y la rigidización prematura del concreto recién mezclado, de modo que dichas operaciones se realicen de manera expedita y con la debida protección al concreto contra la acción directa del sol y del viento. Cuando el concreto se transporta en mezcladoras en tránsito, es útil pintarlas exteriormente de blanco, y cuando se coloca con bomba, es conveniente cubrir las tuberías con aislamiento térmico. Atención especial requieren las operaciones de acabado de las superficies libres, a fin de que se lleven a cabo oportuna y rápidamente para poder darle protección inmediata contra la evaporación a estas superficies.

Protección al Concreto recién Colocado

Al término del acabado, las superficies libres deben protegerse contra la pérdida de agua por evaporación. A fin de poder hacerlo de inmediato, es conveniente el uso de una membrana de curado de buena calidad, aplicada por aspersión muy fina sobre la superficie recién terminada. Es también conveniente complementar dicha protección cubriendo estas superficies ya fraguadas con una tela de polietileno o manteniéndolas continuamente humedecidas. En las superficies moldeadas, las cimbras no soportantes deben retirarse lo más pronto que permita el endurecimiento del concreto, a fin de proteger éste con la citada membrana de curado o por humedecimiento continuo. Las cimbras soportantes deben retirarse cuando el concreto alcance la resistencia requerida para ello, y las superficies recién desmoldadas deben protegerse del mismo modo; de ser posible, el concreto debe conservarse húmedo en el lapso anterior al descimbrado mediante la aplicación de agua a las cimbras, o cubriéndolas con lienzos húmedos.

Mantenimiento de las Estructuras

El mantenimiento más eficaz es el que se realiza de manera preventiva, es decir, corrigiendo las causas más que los efectos. En relación con las estructuras que prestan servicio en ambiente cálido, y en especial si es húmedo (como prevalece en la zona climática C del territorio nacional) merece particular atención la existencia de agrietamientos, porque éstos son motivo frecuente de efectos deteriorantes como la corrosión del acero de refuerzo por ejemplo. Por tal consideración es muy necesario detectar oportunamente la aparición de grietas y darles el tratamiento correctivo adecuado a su origen (19), a fin de prolongar la vida útil de las estructuras.

AGRADECIMIENTOS

Se agradece a la Gerencia de Ingeniería Experimental y Control de la Comisión Federal de Electricidad las facilidades brindadas para la elaboración de este artículo, y a la Srita. Martha Gómez Toledo la eficiente transcripción del manuscrito.

Manuel Mena Ferrer,
Ingeniero Consultor,
México, D.F.

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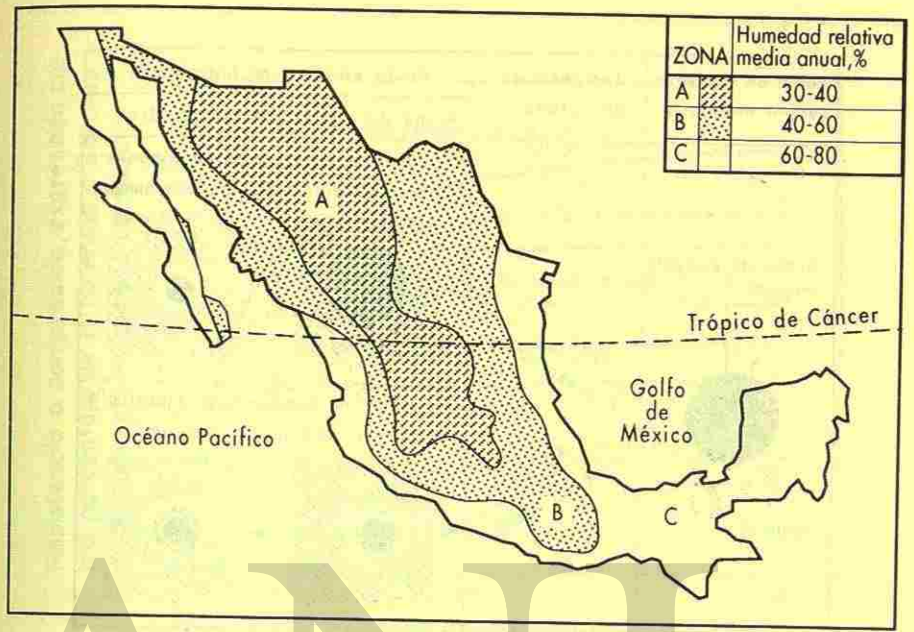

Fig. 1--Zonas locales definidas por humedad relativa (6)

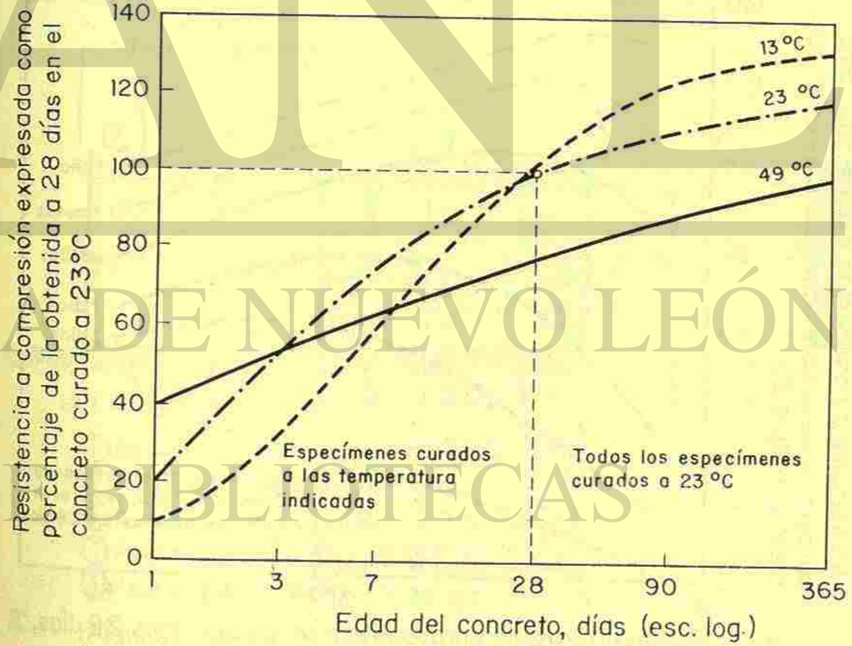

Fig. 2--Evolución de resistencia según la temperatura (7)

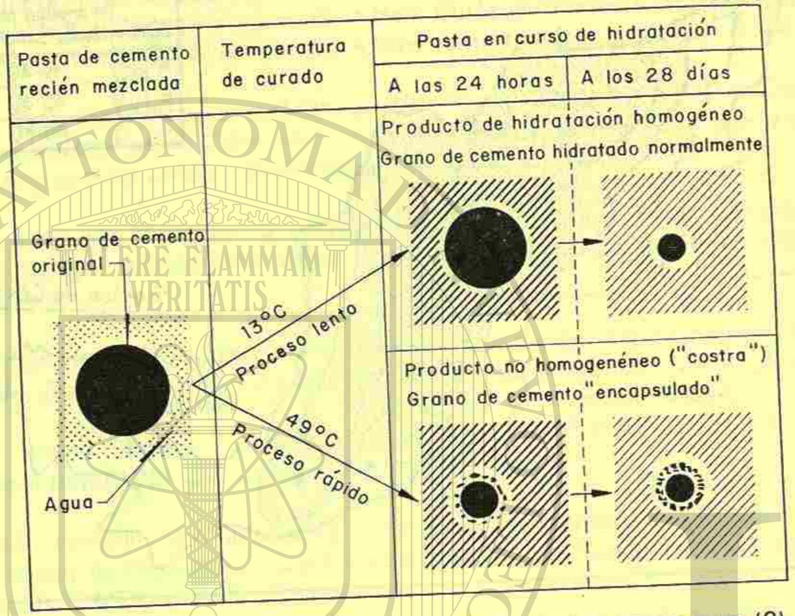

Fig. 3-- Hidratación del cemento en alta y baja temperatura (8)

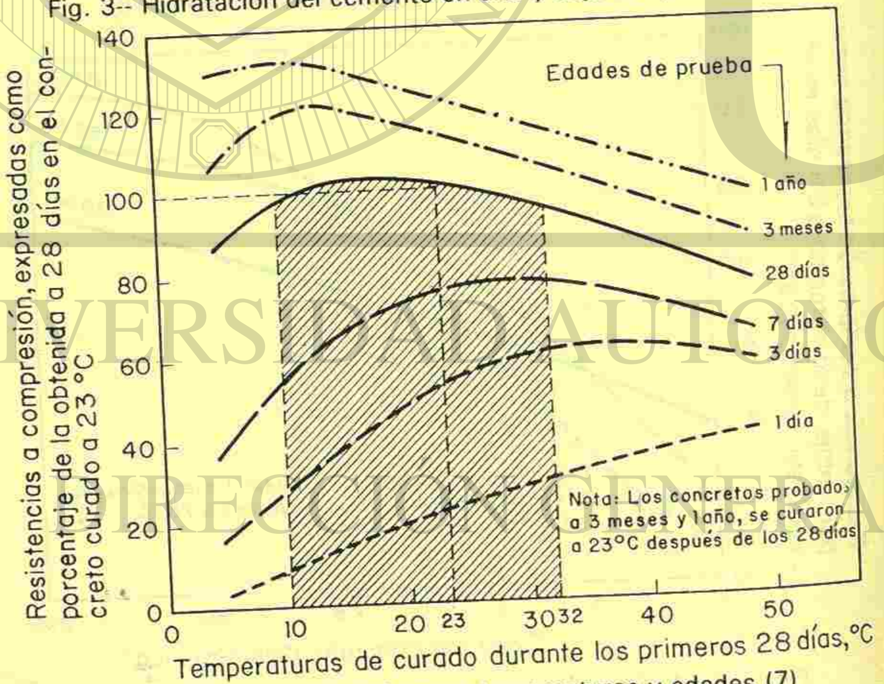

Fig. 4--Resistencias a diversas temperaturas y edades (7)

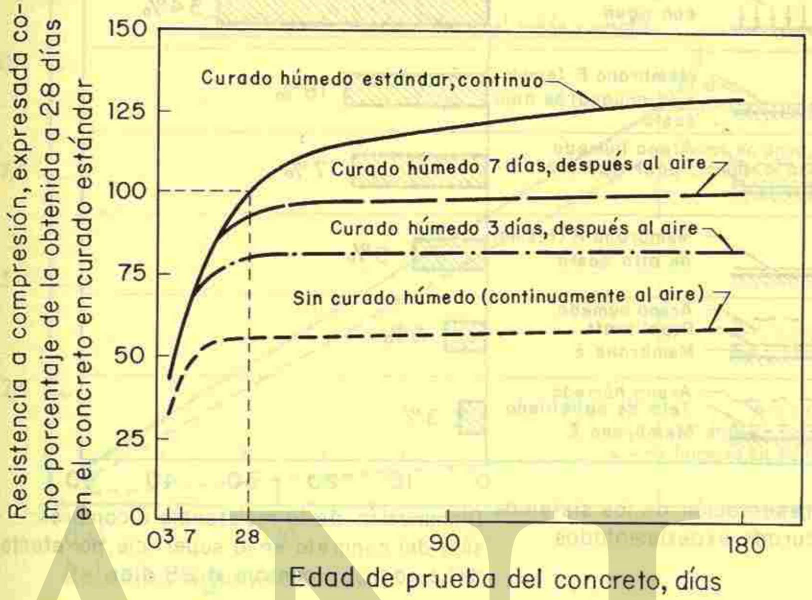

Fig. 5-- Detrimento en la resistencia a compresión del concreto por falta de curado húmedo (9)

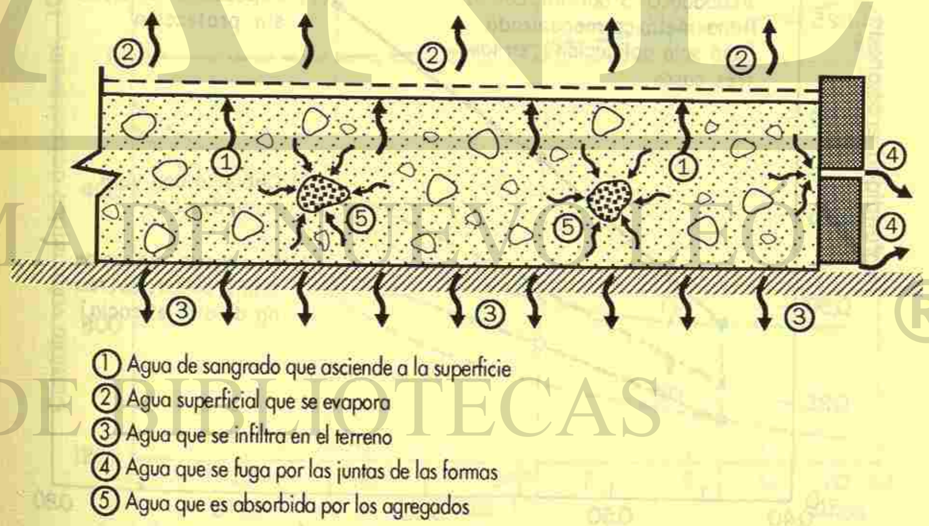

Fig. 6-- Algunas causas frecuentes de la pérdida de agua en el concreto recién colocado.

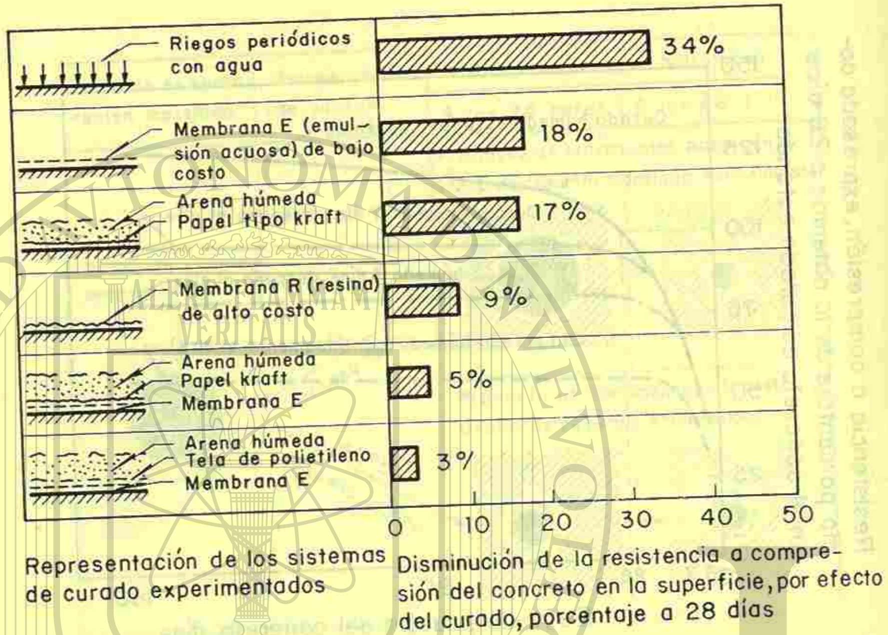

Fig. 7--Eficiencia de diversos sistemas de curado (12)

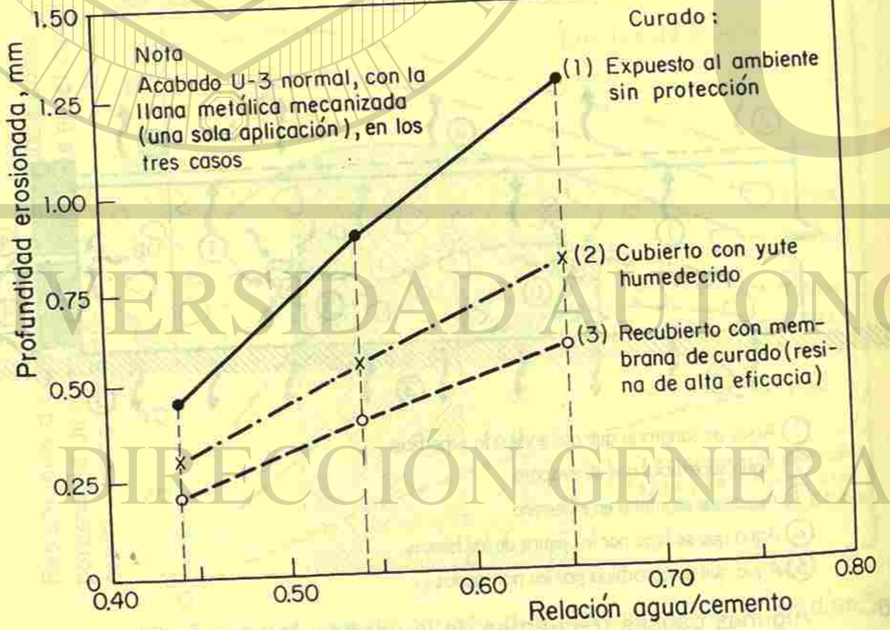

Fig. 8--Resistencia a la abrasión con diverso curado (13)

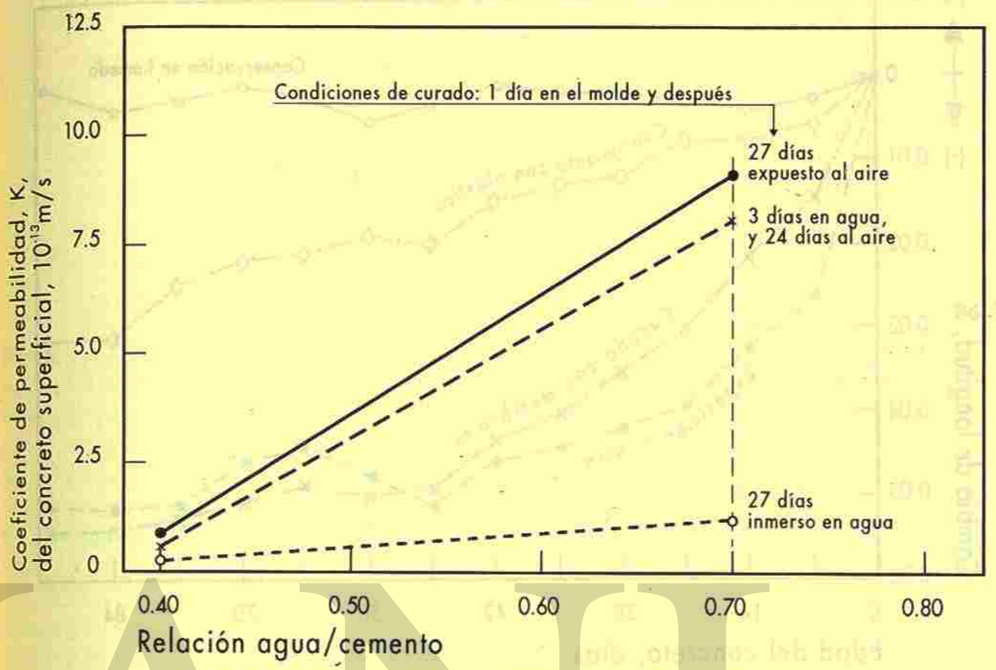

Fig. 9--Permeabilidad superficial según el curado (14)

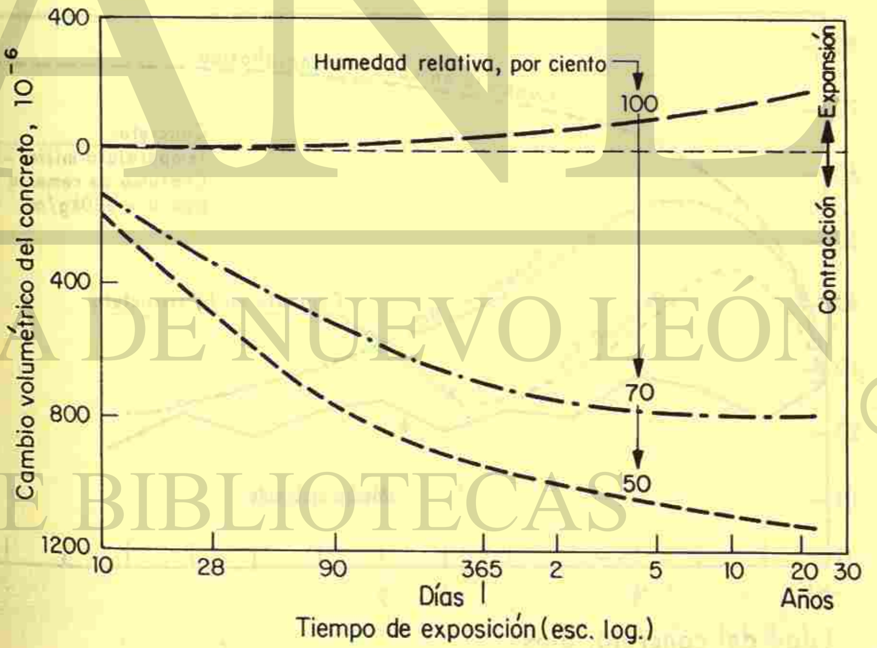

Fig. 10--Contracción por secado en distinta humedad (15)

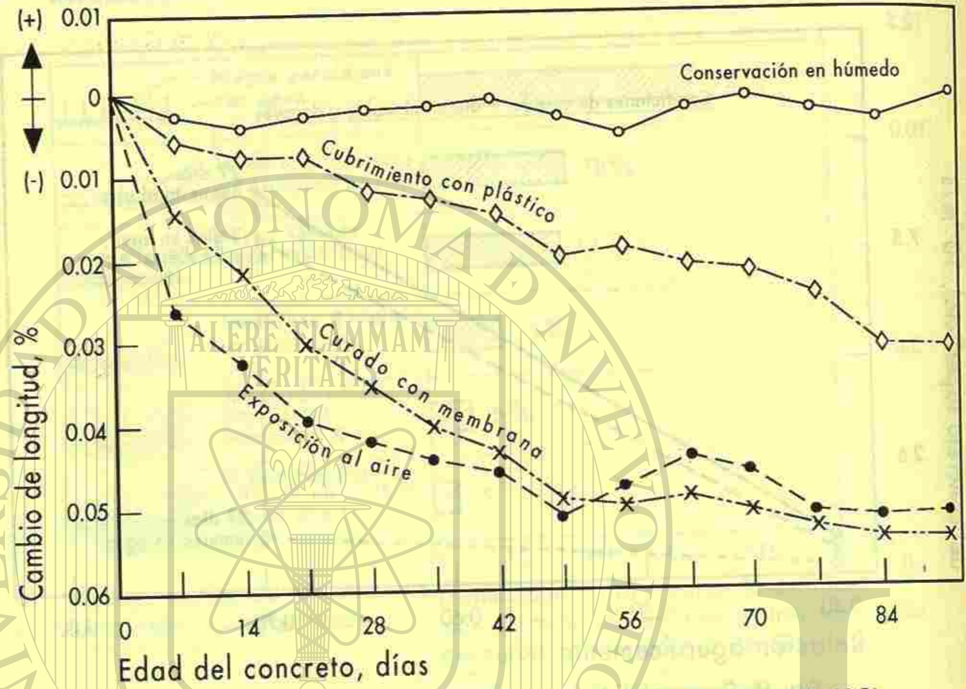

Fig. 11--Influencia del curado en la contracción por secado(16)

Fig. 12--Evolución del calor interno en varias condiciones

ANALISIS DEL CONTENIDO DE AIRE EN EL CONCRETO FRESCO

por

Carolyn M. Hansson y Anders Henrichsen

Sinopsis: Las técnicas normalizadas para determinar los parámetros de contenido de aire en concretos con aire incluido son costosas, consumen tiempo y son subjetivas. Además, estas son técnicas para el *control de la calidad* ya que las medidas se hacen en el concreto endurecido, el cual no puede, obviamente, ser modificado. Este trabajo describe una nueva técnica para asegurar la *calidad*, que se ha desarrollado para permitir determinar los mismos parámetros en el concreto fresco mientras aún está plástico. La técnica involucra la dispersión de los huecos de una muestra de mortero del concreto en un líquido viscoso. Las burbujas en el líquido ascienden con velocidades que son proporcionales a su tamaño y, mediante el monitoreo del volumen de aire que asciende en función del tiempo, los parámetros de los huecos de aire (volumen de aire, superficie específica y factor de espaciamiento) pueden ser determinados aproximadamente en 30 minutos. Además, la técnica es automática y aporta una determinación más objetiva de los parámetros de huecos de aire que los ensayes regulares.

Palabras clave: concreto con aire incluido, distribución de huecos de aire, superficie específica, factor de espaciamiento, resistencia al congelamiento/descongelamiento.

AIR VOID ANALYSIS OF FRESH CONCRETE

by

Carolyn M. Hansson and Anders Henrichsen

Synopsis: The standard techniques for determining the air void parameters in air entrained concrete are costly, time consuming and subjective. Furthermore, there are *quality control* techniques since the measurements are made on the hardened concrete which cannot, obviously, be modified. This paper describes a new, *quality assurance*, technique which has been developed to allow the same parameters to be determined in the fresh concrete while it still plastic, thereby allowing mixes to be modified, if necessary. The technique involves dispersating the void from a mortar sample of the concrete into a viscous liquid. The bubbles in the liquid rise at rates proportional to their size and, by monitoring the volume of air rising as a function of time, the air void parameters (air volume, specific surface and spacing factor) can be determined within approx. 30 min. Moreover, the technique is automatic and provides a more objective determination of air void parameters than do the same standard tests.

Keywords: Air entrained concrete, air void distribution, specific surface, spacing factor, freeze/thaw resistance.

Carolyn M. Hansson is Professor and Head of Materials and Metallurgical Engineering at Queen's University, Kingston, Canada since 1990. Prior to this, she spent nine years in industrial research and consulting in Denmark. She also worked in industrial research (at Martin Marietta Laboratories and At&T Bell Laboratories) and academic (at Columbia University and the State University of New York at Stony Brook) in USA. She graduate from Imperial College, London University and has spent most of her career since then investigating degradation mechanisms of different materials and techniques for improving their durability.

Anders Henrichsen Dansk Beton Teknik A/S, Hellerup Denmark.

INTRODUCTION

When concrete is exposed to repeated freezing and thawing, the resultant expansion and contraction of the pore solution can produce internal stresses which are sufficient to crack and spall the concrete. It has been shown [1] that a system of fine air voids well dispersed throughout the cement paste can provide space for the water to expand into as it freezes and, thereby, reduce or eliminate the internal stresses. The air void system is usually provided by the addition to the concrete mix of an air entraining agent. These are normally organic compounds, such as vinsol resin, with long straight molecular chains with a hydrophobic group at one end and a hydrophilic group at the other. The effect of the hydrophilic group is that the compound is soluble in aqueous solutions while the hydrophobic group reduces the solution's surface tension. This makes it easier for air to be mixed into the concrete and for bubbles to form without coalescing.

Experiments have demonstrated that, for good freeze/thaw resistance, the voids should be in the range of 5 - 200 μm; voids larger than this do not contribute to the frost resistance [2]; the specific surface (ie. the surface area to volume ratio) should be ≥ 20 mm^{-1} and the average spacing between the voids should ≤ 200μm. These values are now generally included in specifications for frost resistant concrete. Questions have recently been raised about the necessity of these stringent requirements in, for example, concretes containing pozzolans and having very low water/cement ratios, but that is not the subject of this paper.

Having specified an air void system, it is, of course, then necessary to be able to determine whether or not the concrete meets the specifications and a method of so doing is described in the ASTM 457: [3]. In this method, the void system is quantitatively analyzed on a polished surface of the hardened concrete. While, theoretically, this is an acceptable technique, it does have some serious disadvantages which may be enumerated as follows.

1. The concrete must be hardened by which time it is too late to rectify any problems.

2. Sample preparation requires coring, sectioning, grinding and polishing. This is time consuming and costly and it is difficult to produce a representative section because, for example, pull out of particles from the surface results in holes which can be interpreted as air voids.

3. The optical micrographic analysis is both tedious and time consuming. More importantly, the reliability of the results is dependent on the ability of the operator to distinguish firstly between voids in the aggregate and voids in the paste and, secondly, between glassy spheres in fly ash and voids in the paste.

The fresh concrete air void analysis technique was developed [4] to give the same information as is provided by the ASTM 457 Standard Practice while avoiding the disadvantages of that procedure described above.

PRINCIPLES OF THE TEST METHOD

The air voids developed in the fresh concrete by the addition of an air entraining agent (together with the entrapped air) are released into a viscous liquid, the properties of which allow the resulting bubbles to retain their original size and neither coalesce nor disintegrate into a number of smaller bubbles. The bubbles then rise through the liquid at rates dependent on their size (according to Stokes Law) and enter a column of water above the liquid. The viscosity of the liquid slows the initial rise of the bubbles and provides a measurable separation in time between the appearance at the top of the column, of bubbles of different sizes rising from the same layer of the liquid. The bubbles rise through the column of water and collect under a submerged buoyancy recorder which is attached to a balance. The change in buoyancy with time is monitored by computer. The bubble size distribution, such as that illustrated in Fig. 1, in each 15 s interval has been measured empirically and, on the basis of this empirical calibration, the recorded change in buoyancy with time can be related to the number of bubbles of different size. From this data, the following air void parameters specified by ASTM Standard Practice 457 are calculated:

- air volume in voids of diameter < 3.0 mm, < 1.0 mm and < 0.5 mm (as volume % of concrete and as volume % of paste)
- spacing factor (in mm)
- specific surface (in mm^{-1})

These parameters have been calculated to correspond to those that would be obtained from linear traverse measurements on a plane surface of the hardened concrete making the assumptions used in the ASTM 457 practice, namely (i) that the average measured chord length is equal to 2/3 of the true void diameter and (ii) for calculations of specific surface and spacing factor, that the voids are all of the same size and that they are located at the lattice points of a regular cubic array.

In addition, the data can be presented as (i) a graph of cumulative fraction of voids (as % of concrete) versus the true void diameter; (ii) a bar chart of the actual void volume (as % of the paste) in different ranges of void diameter.

TEST PROCEDURE

Samples can be taken at any time, for example, at the batching station, on arrival at the construction site or after placement and compaction. To determine the air void parameters corresponding to those obtained on a core of the hardened concrete, the sample should be taken after final compaction. The procedure is non-destructive, however, because only a very small amount (20 ml) of concrete is removed and this can easily be immediately replaced.

A sample of the mortar fraction of the concrete is taken from a representative area of the concrete by inserting a wire cage into the concrete under vibration. This causes the mortar to enter the cage while excluding all aggregates larger than the wire spacing of 6 mm. A syringe is inserted into the mortar in the cage to withdraw a 20 ml sample for analysis.

The sample is injected from the syringe into the special liquid in the bottom of a riser column, illustrated schematically in Fig. 2, and the computer control is started. The computer immediately (i) sets the balance to zero, (ii) causes the mortar to be gently stirred for 30 s to release the air voids into the liquid and (iii) monitors the balance readings.

In the early stages of the measurement, the size distribution of bubbles arriving at the buoyancy recorder range from a few mm down to a few µm but, for each succeeding period, the maximum size of bubbles decreases as all the larger bubbles have already risen to the top of the column. This is illustrated schematically for four bubble sizes in Fig. 3 whereas, in practice all bubble sizes must be considered.

The measurement procedure continues for 25 min which has been demonstrated to be an appropriate time period to give air void parameters comparable to those obtained by the ASTM linear traverse method with acceptable accuracy (typically ± 10%). While voids of diameter $\leq 7\mu m$ will continue to rise after this period, neglecting them is found to have a negligible effect on the results.

RESULTS

The total air content and the void size distribution of the sample are calculated from the measurements of weight change as a function of time. From these data, the following parameters are calculated: (1) the total air content; (2) the air content in voids < 3mm, < 1mm and < 0.5mm as a volume % of both the concrete and the paste; (3) the specific surface and (4) the spacing factor.

Unlike the linear traverse method where the number of voids counted is of the order of a few hundred, the number of voids involved in the measurements by present method is of the order of millions. It is not, therefore, possible to measure the bubbles individually and present them in a detailed histogram for each 20 µm size range as is the current practice for the linear traverse method. Instead, graphic representation of the results is accomplished by plots of the cumulative fraction of voids in ten different size ranges as a volume percentage of the concrete, and by the fraction of the voids in these size ranges as a percentage of the cement paste volume. Examples of these plots are given in Figs. 4 and 5. Because a specific volume of air represents a far greater number of voids of small diameter than it does of voids of large diameter, histogram in Fig. 5 is usually fairly flat rather than the exponentially decreasing form normally observed in the linear traverse histograms of "number of voids in different void size ranges".

COMPARISON WITH ASTM 457 LINEAR TRAVERSE DATA

At Dansk Beton Teknik, five bars were cast from different concrete mixes. Five samples for the fresh concrete tests were taken at 10 cm intervals from each of the bars. After the bars were hardened, five impregnated, polished cut sections from each bar were prepared and analyzed according to the ASTM 457 procedure. The agreement between these two sets of data, illustrated in Fig. 6, is within ± 10%. Similar comparative studies are currently being carried out within the European Community and at Laval University, Quebec City, and the results of these investigations should be available within the next year.

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Carolyn M. Hansson, Professor and Head Materials and Metallurgical Engineering, Queen's University, Ontario, Canada.

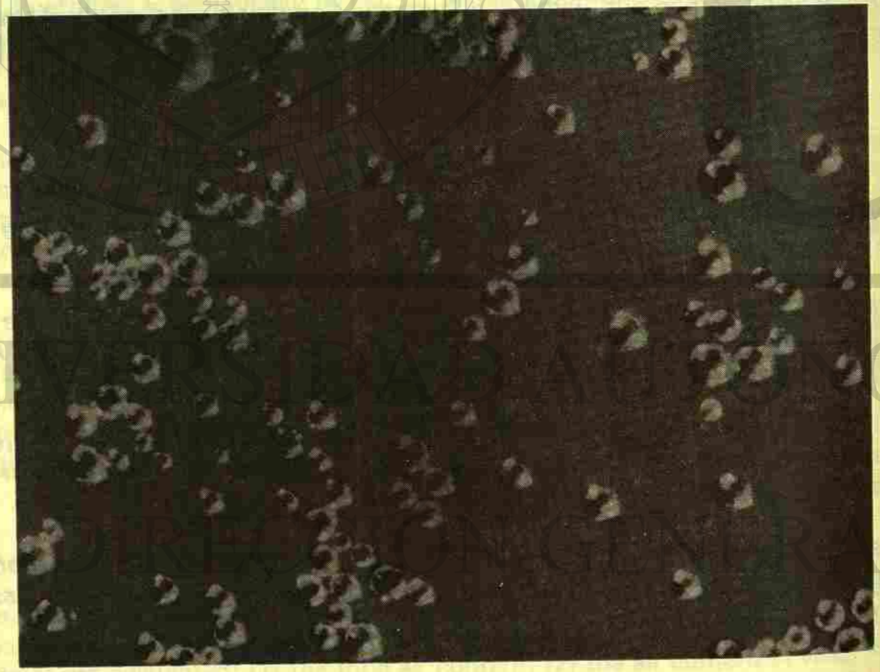

Fig. 1. Typical distribution of bubbles obtained from air entrained concrete.

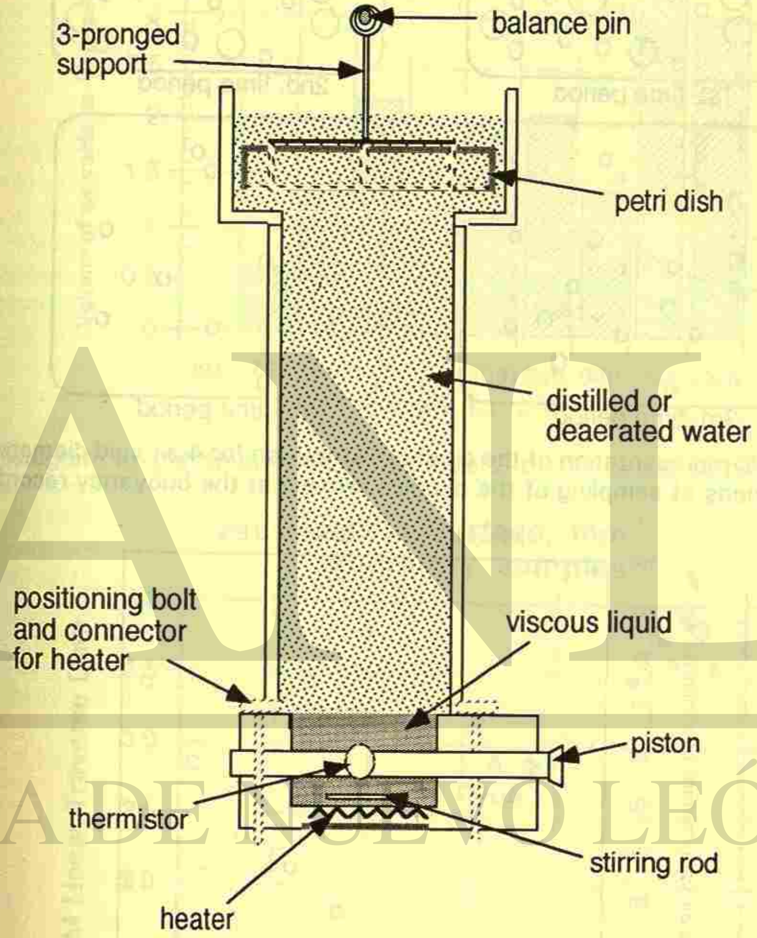

Fig. 2. Schematic representation of the riser column.

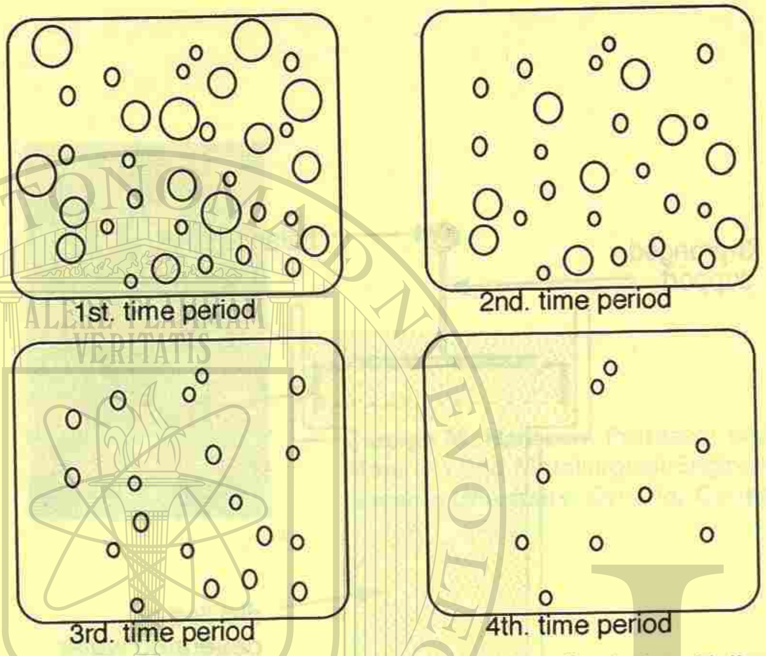

Fig. 3. Schematic representation of the air void distribution for 4 air void diameters in four different periods of sampling of the bubbles arriving at the buoyancy recorder.

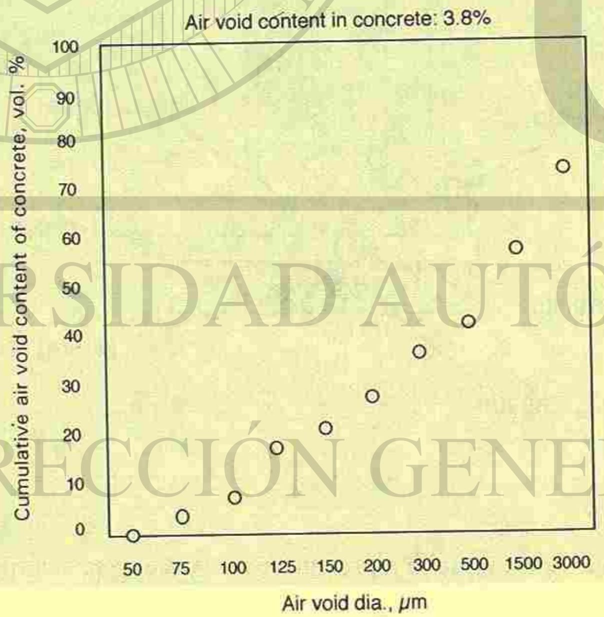

Fig. 4. Cumulative air void volume in concrete.

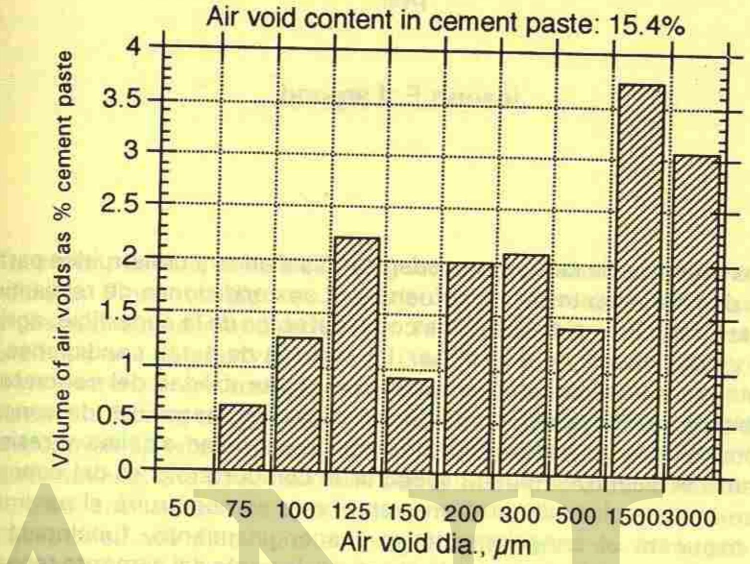

Fig. 5. Histogram of the amount of air (as a vol% of the paste) in different void size ranges.

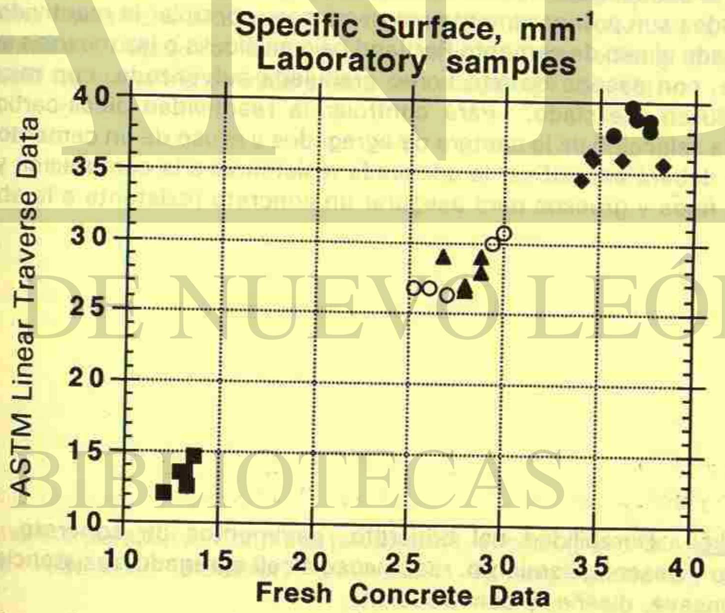

Fig. 6. Comparison of specific surfaces determined on the same laboratory concretes by the ASTM 457 procedure and the fresh concrete air void analysis techniques.

DURABILIDAD DE LOS PAVIMENTOS DE CONCRETO

por

Joseph F. Lamond

Sinopsis: Los pavimentos de concreto pueden ser diseñados y construidos para minimizar las condiciones de relajamiento de los esfuerzos. Las condiciones de relajamiento de los esfuerzos del pavimento han sido definidas como defectos de la superficie, agrietamiento, deformaciones y deficiencias de las juntas. La mayoría de estas condiciones no pueden ser directamente identificadas como un problema de durabilidad del concreto. Los tres principales problemas de durabilidad relacionados con los materiales de concreto son la resistencia al congelamiento y descongelamiento, reactividad alcalina y resistencia a la abrasión. Durante el diseño, se puede asegurar el comportamiento del concreto a largo plazo cuando se conoce el medio ambiente en el que se construirá el pavimento. Si el pavimento es expuesto al congelamiento y descongelamiento, funcionará cuando el pavimento en servicio no esté críticamente saturado, la pasta del cemento tenga el sistema adecuado de vacíos de aire, los agregados no sean susceptibles al congelamiento y los químicos para el descongelamiento no sean usados en concretos inmaduros. Si se sabe que los agregados son potencialmente reactivos para controlar la reactividad álcali-sílica debe ser evaluado el uso de cemento Portland bajo en álcalis o las mezclas adecuadas con ceniza volante, con escoria de alto horno granulada pulverizada, con microsílica o con cemento hidráulico mezclado. Para controlar la reactividad álcali-carbonato, deberá especificarse la selección de la cantera de agregados y el uso de un cemento Portland bajo en álcalis. Se deberá especificar la adecuada resistencia a la compresión y la calidad de los agregados finos y gruesos para asegurar un concreto resistente a la abrasión.

Palabras Clave: Durabilidad del concreto, pavimentos de concreto, resistencia al congelamiento y descongelamiento, reactividad álcali-agregado, resistencia a la abrasión, métodos de ensaye, diseño, y construcción.

DURABILITY OF CONCRETE PAVEMENTS

by

Joseph F. Lamond

Synopsis: Concrete pavements can be designed and constructed to minimize pavement distress conditions. Pavement distress conditions have been identified as surface defects, cracking, deformations, and joint deficiencies. Most of these conditions cannot be directly identified as a concrete durability problem. The three mayor durability problems related to the concrete materials are resistance to freezing and thawing, alkali reactivity, and abrasion resistance. During the design, long term pavement performance can be assured when the environment in which the pavement is to be constructed is known. If exposed to freezing and thawing, the pavement will perform when the pavement in service is not critically saturated, the cement paste has the proper air-void system, the aggregates are non-frost susceptible, and deicing chemicals are not used on immature concrete. If aggregates are known to be potentially reactive, low alkali portland cement or proper blends of fly ash, ground granulated blast-furnace slag, or silica fume or blended hydraulic cement should be evaluated and used to control alkali-silica reactivity. Selective quarrying of aggregates and use of low alkali portland cement should be specified to control alkali-carbonate reactivity. Adequate compressive strength and quality fine and coarse aggregates should be specified to assure abrasion resistant concrete.

Keywords: Concrete durability; Concrete pavements; Freezing and thawing resistance; Alkali-aggregate reactivity; Abrasion resistance; Testing methods; Design; and Construction.

Joseph F. Lamond is a Consulting Engineer in Springfield, Virginia. He is a past director of the American Concrete Institute. A fellow of ACI, he serves as chairman of the Fellows Nominating Committee and the technical committee on Removal and Reuse of Concrete and is a member a number of other board and technical committees. He serves on the Concrete and Aggregates, and Cement committees of the American Society for Testing and Materials (ASTM) and Concrete committee of the Transportation Research Board. He is a member of the International Concrete Repair Institute and a Registered Professional Engineer.

INTRODUCTION

Durable concrete pavements require good quality concrete. If properly proportioned for the environment to which it will be exposed, concrete is inherently a durable material. When a concrete pavement deteriorates, it can be caused by the adverse performance of either the aggregates or the paste in the concrete; corrosion of the reinforcement; improper design or construction, changes in traffic use, or inadequate maintenance.

Concrete pavements are usually designed for a certain life expectancy based on anticipated future traffic conditions. Service histories indicate, however, that the service life of concrete varies considerably from that expected when designed and constructed. Often, certain factors associated with the contractor's performance, concrete materials, other materials, inspection practices and a change in traffic will determine the ultimate service life of a pavement.

The existing credibility gap between pavement design and pavement performance can be substantially diminished by using a total quality assurance management system that includes all attributes of designing and constructing a concrete pavement. Quality assurance management is an effort to achieve the desired quality and is concerned with structural design, specifications, proper utilization of concrete materials, and the concrete quality control.

The three major concrete durability problems caused by the concrete materials in pavements are alkali reactivity, resistance to freezing and thawing and abrasion resistance. Avoiding these durability problems by providing proper design and specifications, requirements, and construction practices will be discussed in this paper. Chloride-induced corrosion of the reinforcing steel, although a problem in concrete bridges, has not been reported as a problem in reinforced concrete pavement. Although changes in traffic use and inadequate maintenance practices affect concrete pavement durability, they will not be covered in this paper.

PAVEMENT DISTRESS CONDITIONS

Guidance for making a condition survey of concrete pavements, such as highways, airfields, parking lots, or traffic areas in warehouses after a number of years of exposure is in ACI committee report 201.3R [1]. A condition survey is an examination of exposed concrete to identify and define the areas of distress. Distress manifestations have been categorized and their severity and extent or frequency of occurrence quantified by several organizations including the Strategic Highway Research Program (SHRP)[2].

Distresses visually observed in pavements are surface defects, deformations, joint deficiencies, or cracks. These distresses by themselves usually do not indicate the reason for a concrete or pavement durability problem. Investigations beyond the visual observation are needed to determine the cause of the distress. Corrective action should not be taken until the cause is determined.

ACI 201.3R [1] shows a number of different conditions for distress conditions. The surface defects in the report are polishing, spalls, scaling, crazing, and wheel track wearing. These surface defects may be related to the concrete materials, construction practices, or concrete mixture proportions. Faulting, settling, and pumping are shown as deformations and these may be caused by pavement design or construction but not concrete materials or mixture proportions.

Joint deficiencies are identified in ACI 201.3R [1] as joint creeping, loss of joint sealant, loss of bond, cohesion, or debris impregnation of the joint sealant, and joint or crack spalling or failure. These deficiencies may be related to pavement design, construction or concrete materials. One of the most common deficiencies seen in concrete pavements are cracks. Cracks may be longitudinal, meandering, corner, D-cracking, transverse, diagonal, edge, map, or plastic shrinkage. Cracking may be related to design, construction, or concrete materials.

D-cracking is one of the few distress conditions in pavements that can be directly related to the concrete materials. D-cracking is freezing and thawing distress damage due to the frost susceptible coarse aggregate.

FREEZING AND THAWING

Introduction

One of the most important causes of concrete deterioration is freezing and thawing. Theory and research have provided a good understanding of the mechanisms of frost action in concrete. For many years these findings provided a sound basis for void spacing factor concept, which has been the basis for assessment of the adequacy of air void systems in the paste. However, even good quality concrete may suffer damage from cyclic freezing under certain conditions, such as when critical saturation is reached or when nondurable aggregates are used. The use of deicing salts contributes to this condition and can accelerate surface deterioration.

The use of high-range water reducers, fly ash, ground granulated blast-furnace slag, and silica fume may affect the entrained air void systems. The air void spacing values above those generally accepted are often found in concrete containing these materials and may or may not result in poor resistance to freezing and thawing durability.

The four separate conditions that have to be considered when explaining the resistance of concrete to freezing and thawing are as follows:

Critical saturation,

Cement paste,

Aggregates, and

Use of deicing chemicals.

Studies by Verbeck and Landgren [3] as well as those by Powers [4] make it clear that the paste and aggregate have to be considered separately. The application of deicing salts to pavements presents a third condition that has to be considered separately. When freezing takes place in the presence of deicing chemicals, localized failure of the exposed surface occur that is called scaling.

Mather [5] stated that concrete will be immune to the effects of freezing and thawing if the pores containing freezable water are never more than 91 percent filled, i.e., not critically saturated. He also points out, as others did, the importance of sound aggregate and maturity of the paste. ACI 201.2R [6] states that air-entrained concrete should be able to withstand the effects of freezing as soon as it attains a compressive strength of about 3.5 MPa provided that there is no external source of moisture. At 10 degrees C most well proportioned concrete will reach this strength some time during the second day. Before being exposed to extended freezing while critically saturated, concrete should attain a compressive strength of 27 MPa. A period of drying following curing is advisable.

Critical Saturation

The theory of critical saturation is accepted without question. One cubic centimeter of water occupies about 1.09 cubic centimeters of space after freezing and any void in the aggregate or cement paste which is more than 91 percent full of water will be subjected to pressures when freezing water turns to ice, unless the excess water can be forced from the void during freezing [7]. Critical saturation can be controlled by good design practices. ACI 201.2R [6] states structures should be designed for good drainage and joints not necessary for volume change kept to a minimum. Good drainage is especially necessary in concrete pavements to prevent low spots conducive to the ponding of water. Walls are divided into small squares by control joints more by tradition and for appearance than for consideration of volume change.

Freezing of Cement Paste

Concrete freezing and thawing deterioration mechanisms occur in the pore structure of the cement paste. The submicroscopic pore system of hardened cement paste is composed of gel pores, capillary cavities, and if purposely air-entrained paste, microscopic air bubbles. The gel pores are generally filled with water and if the paste is saturated the capillary cavities will also be filled. However, the gel pores are so small that it is impossible for water to freeze unless the temperature drops far below normal ranges. It is estimated that freezing cannot occur at any temperature above -78 degrees C. In frozen concrete, therefore water in the gel pores is supercooled but not frozen. Capillary cavities, on the other hand are sufficiently large to accommodate ice crystals and will freeze. Air-entrained bubbles are not normally filled with water.

To fully understand the freezing mechanism, it would be helpful to understand the structure of the cement paste. However, this is beyond the scope of this paper and the paper by Powers [8] is recommended on this subject. Freezing in the capillaries and generation of hydraulic pressure was explained by Cordon [7]. In a water-soaked paste the capillary cavities and gel pores are full, or nearly full of water. When the temperature falls to a point where freezing should begin, ice crystals appear in the largest capillary cavities. When the water in the larger cavities begins to change to ice, the volume of water plus ice will exceed the original capacity of the cavity. Therefore, during the time when the water in the capillaries is changing to ice, the cavity must dilate or excess water must be expelled from it.

Cement paste is a permeable material, although the coefficient of permeability is extremely low. Hence, there is a possibility that excess water can escape from the capillary during the process of freezing. The forcing of excess water from the capillary involves the generation of pressure. Factors affecting the pressure include:

Coefficient of permeability of the paste,

Distance of the capillary to the void boundary, and

Rate at which freezing occurs.

In general, during the process of freezing, hydraulic pressure will exist throughout the paste. This pressure will be higher for points farther from an escape boundary. If a point is sufficiently remote from an escape boundary, the pressure will be high enough to stress the surrounding gel beyond its elastic limit, or beyond its tensile strength, and thus produce permanent damage.

Every air void in cement paste is assumed to be bordered by a zone in which the hydraulic pressure cannot become high enough to cause damage. Theoretically, pressure increases approximately in proportion to square of the distance from the void. By reducing the distance between the voids to the point where the protected zones overlap, generation of disruptive hydraulic pressures during the freezing of water in the capillaries can be prevented.

If the cement paste contains an approximate distribution of entrained-air voids characterized by a spacing factor of about 0.02 mm, freezing does not produce destructive stress [9].

Paste Problems

Carrasquillo [10] studied the effect of fly ash on the air entrainment, and freeze-thaw durability. Two different fly ashes meeting ASTM C 618 Class C and Class F [11] were used to replace 0, 20, and 35 percent of a portland cement, by weight. Three different air entraining admixtures were used. It was found that the use of fly ash in concrete could reduce the effectiveness of air entraining admixtures depending on the properties of the admixtures such as the loss on ignition. However, concrete containing fly ash exhibited freeze-thaw resistance equal or better than concrete containing portland cement only, provided both had similar entrained air contents.

In concrete pavements exposed to freezing and thawing, the loss on ignition value for the fly ash should be known. The amount of air-entraining admixture should be continually adjusted to maintain the average total air content within the recommended levels for the degree of exposure of the concrete.

CANMET investigated freezing and thawing resistance of non air-entrained and air-entrained concrete incorporating various percentages of silica fume [12]. Non air-entrained concrete, regardless of the amount of silica fume showed very low durability factors when tested in accordance with ASTM C 666 [11]. The concrete appeared to show somewhat increasing distress with increasing amounts of the silica fume. Air-entrained concrete, regardless of the water-cementitious material ratio and containing up to 15 percent silica fume as partial replacement for cement performs satisfactorily when tested in accordance with ASTM C 666 [11]. However, concrete incorporating 30 percent silica fume and a water-cementitious materials ratio of 0.42 performs poorly.

ACI Committee report 226.1R [13] on ground granulated blast-furnace slag (GGBF) as portion of the cementitious material reported on studies by a number of researchers on resistance to freezing and thawing. Results of these studies generally indicate when portland blast-furnace slag cement was tested in comparison to Type I or II portland cement, their freezing and thawing in concrete were essentially the same [14] [15] [16]. As with all hydraulic cement concretes, proper air content and bubble spacing factor are necessary for adequate protection in freezing and thawing environments. Recent studies of air-entrained concrete containing GGBF slag used as 50 percent of the total cementitious material was found to be frost resistant even though a measurable difference in weight loss was found when compared to Type II portland cement and tested using ASTM C 666 [11]. Similar results were found by Malhotra [17] using various percentages of GGBF slag with portland cement.

Freezing of the Aggregates

Insofar as aggregate durability is concerned, D-cracking and popouts are known freeze-thaw failures in the aggregate particles. ACI 221R [18] states that concrete containing frost-resistant paste may not be totally resistant to freezing and thawing if it contains any aggregate particles which become critically saturated. An aggregate is considered to be critically saturated when there is insufficient unfilled pore space to accommodate the expansion of the water which accompanies freezing [3]. The pore structure, permeability, and tensile strength are factors influencing the failure mechanism of the coarse aggregate. Experience has yet to show that fine aggregates are directly associated with freezing and thawing deterioration of concrete.

Testing

Coarse aggregates can be evaluated in concrete using ASTM Test Methods C 666 and C 682 [11]. A number of laboratory test methods have been developed on unconfined aggregates which are intended as a measure of soundness, resistance to freezing and thawing, and a general indicator of quality of the coarse aggregate. These aggregate methods have not been found to relate well to freezing and thawing performance as those used to evaluate the aggregates in concrete.

The Strategic Highway Research Program has recently developed an aggregate evaluation method. The test method involves submerging pre-dried aggregates in water in a pressure chamber. Under a pressure of approximately 8 MPa, water enters the pores and compresses the air within these pores. When the pressure is rapidly released, the compressed air forces the water out of the pores. The pressure exerted by the water, while being forced out, can result in fracture of frost susceptible aggregates.

Petrographic examination of aggregates using ASTM C 295 [11] may help identify the types of particles present that may break down in freezing and thawing.

Aggregate Problems

Distress due to freezing and thawing by critically saturated aggregate particles is commonly manifested in the general deterioration of the concrete, popout of the aggregate particle, or usually in a phenomenon known as D-cracking.

A popout is characterized by breaking away of a small portion of the concrete surface due to internal pressure, thereby leaving a shallow and typically conical spall in the surface of the concrete above the aggregate particle. These popouts may develop on any surface directly exposed to moisture and freezing and thawing cycles. Chert particles of low specific gravity, limestone containing clay, and shalely materials are well known for this behavior. Occasional popouts in many applications may not detract from serviceability.

One common indication of freezing and thawing deterioration is the appearance of cracks which run approximately parallel to the joints or edges of concrete pavements. As deterioration progresses these parallel cracks occur farther away from the joint. This type of cracking has been designated D-cracking. Eventually, the concrete near the joint disintegrates and spalls .

Several theories have been advanced to explain the cause of D-cracking. The most accepted theory is the concrete next to the joint or a corner is exposed to more moisture than the inside portion of the slab. Moisture is also available from the base material. Thus the concrete next to the joint or a crack has a higher degree of saturation during freezing weather. D-cracking is caused by water freezing in the voids of the aggregate and the paste if it is not properly air-entrained. ACI 221 R [18] indicates nearly all occurrences of D-cracking are associated with sedimentary aggregates, including limestone, dolomite, shale, and sandstone.

Chemical admixtures

Resistance to freezing and thawing of concrete containing chemical admixtures meeting ASTM C 494 Type F or G [11] and called high-range water reducers or superplasticzers has had intensive research since the commercial introduction of the material. Most researchers reported excellent frost resistance for concrete containing these admixtures, despite widely reported increases in the bubble-spacing factor over 0.2 mm commonly accepted for durability. Philleo [19] concluded in a synthesis of highway practice that these chemical admixtures do not appreciably alter the pore structure of hardened concrete, except with some materials the air-void system is altered.

Using Deicing Chemicals

Concrete that is adequately air-entrained for frost resistance may nevertheless be damaged by repeated application of deicing chemicals. The surface disintegrates in the form of the loss of mortar with pitting and scaling. The mechanism by which deicing chemicals damage concrete is fairly well understood. It is generally agreed that the action is physical rather than chemical. The mechanism involves the development of destructive hydraulic pressures during freezing, principally in the paste, similar to the previously discussed mechanism of frost action in the cement paste. Litvan [20] concluded that deicing chemicals cause a high degree of saturation in the concrete, and that saturation is mainly responsible for there detrimental effect.

Scaling is likely to occur on surfaces that have been overvibrated, trowelled too early or too long, subjected to plastic shrinkage cracking, or where excessive bleeding occurred. Such surfaces tend to have a weak layer of paste or mortar either at the surface or just below, and may have microcracks or bleeding channels that can transport surface solutions to lower levels. Careful attention to mixture proportions, placing, and finishing should eliminate many potential problems.

The use of ASTM C 672 [11] will demonstrate the acceptability or failure of a given concrete mixture. ACI 201.2R [6] indicates that the benefit from air entrainment exposed to deicers is explained in the same way as for ordinary frost action. Laboratory tests and field experience have confirmed that air entrainment greatly improves resistance under severe conditions. It is now possible to consistently build scale-resistant pavements. Klieger in his studies [21] showed air-entrained concrete had to attain attaining approximately 2.5 MPa and a period of drying after curing before opening to traffic to be scale resistant to deicing chemicals.

Summary

Consideration has to be given to the environment when the pavement is to be exposed to freezing and thawing in saturated condition, long-term durability from effects of freezing and thawing must be considered during the design of the project. The proper air void system has to be specified for the paste. The aggregates have to be non-frost susceptible. The chemical and mineral admixtures and supplementary cementitious materials should be evaluated to determine if they have an affect on the air void system to provide durable concrete. A quality control plan has to assure the concrete is properly air-entrained during construction. Whiting and Stark [22] listed 35 factors that effect air entrainment. De-icing salts should not be applied to immature pavement.

ALKALI REACTIVITY

Introduction

Chemical reaction of aggregates and alkalies in concrete can result in deterioration and reduced service life of concrete pavements. This reaction involves certain constituents in the aggregates and soluble alkali compounds which significantly increase the pH of the pore solution in the concrete. In order for this reaction to take place, the concrete must be subjected to sufficient moisture in service. Stark, et al, reported [23] the relative humidity in a pavement slab in the California desert region were above 80 percent threshold level in the winter season. Expansive ASR was found through the full thickness of the pavement. The alkaline components are usually associated with the alkali in portland cement, but may also be contributed by aggregates and admixtures in concrete. External sources such as deicing chemicals may also supply alkalies. There are two known forms of alkali-aggregate reactivity; 1) alkali-silica reactivity, and 2) alkali-carbonate reactivity. Alkali-silica reaction (ASR) is the most widespread, extensively known and studied.

Mechanism of Alkali-Silica Reaction

Concrete is composed of aggregates and paste. The paste is composed of the products of the hydration between the cementitious materials and water. Although appearing as a solid (discounting the entrained and entrapped air voids) the cementitious paste actually consists of a matrix of solid material with an internal network of interconnected pores. The volume of these pores and their interconnectedness depends on factors such as water-cement ratio, curing regime, and the cementitious materials used.

Water can enter and move through this network of pores, dissolving soluble salts of calcium and alkali metals (sodium and potassium). The dissolution of these salts increases the hydroxide ion (OH) concentration or pH of the resulting solution. In the portland cement paste, the hydration reaction produces sufficient calcium hydroxide to saturate the pore solutions with respect to this compound. In solutions saturated with calcium hydroxide, the maximum pH is about 12.5; however, increasing amounts of alkali will increase the pH. Consequently, the soluble alkali content of a concrete's cementitious paste affects the upper limit of pH or alkalinity of the pore solution.

The ASR process begins with the reaction of hydroxide ions from a highly alkaline pore solution with silica in the aggregate. This reaction produces a gel which may cause the aggregate and the concrete to crack. The cracking is also believed to be associated with the expansion of the gel due to water absorption. Depending on the aggregate mineralogy and reaction conditions, even very small amounts may cause cracking; under different conditions alkali-silica gel may not be deleterious.

Factors Affecting the Potential for ASR Related Distress

In order for ASR related distress to occur in concrete, three factors are required:

1) The presence of reactive forms of silica in the aggregate;

2) A sufficiently high alkali content in concrete pore solution;

3) Sufficient moisture.

Other factors may affect the development of ASR related distress. Among them are admixtures, deicing chemicals, concrete permeability, and conditions placement and curing.

Reactive silica SiO_2, occurring in a variety of forms, is a common constituent of many aggregates. In neutral and acidic environments, the various forms of silica are chemically very stable. As a consequence of this stability, silica forms the bulk of the natural sands and gravels. However, in highly alkaline environments, such as occur in concrete, the solubility of silica increases dramatically.

The reactivity of silica, in addition to being affected by the alkalinity or pH, is also affected by the form, grain size, or crystalline structure of the silica. Descriptions of the various forms can be found in ASTM C 294 [11]. Amorphous, or non-crystalline and poorly crystalline silica, such as volcanic glasses and opal, are the most reactive forms. The high temperature materials, tridymite and cristobalite, and chalcedony, a fibrous form occurring in some chert, are also fairly reactive. The above forms can all be classed roughly as rapidly-reactive and typically will be classified as potentially deleterious by traditional tests such as ASTM C 227 and C 289 [11]. These rapidly-reactive forms often exhibit the pessimum effect, where the maximum expansion occurs when the aggregates contain a certain proportion of the reactive element. At proportions greater or less than the pessimum, the expansion is less. For example, an opal content in aggregate about 5 percent (the pessimum amount) yields the maximum expansion. Greater or lesser amounts of opal yield lower expansions, whereas, for some other mineral maximum expansion is achieved when aggregate consists of 100 percent of that mineral.

More slowly reacting forms of silica include metamorphically strained quartz, and micro crystalline quartz. These forms have been found to cause deleterious reactivity in concrete, although it may take longer to develop than with rapidly-reactive aggregates. These forms of silica are a primary constituent of quartzite, sandstone and sand and gravel. Chert, composed of microcrystalline quartz and sometimes chalcedony, is found in sand and gravel and some carbonate rocks. Unstrained quartz, may be found in granite and granitic gneiss appear to be the least reactive form of silica but has been associated with deterioration due to ASR [24]. The greater the porosity of the aggregate due to either its natural internal structure or fractures induced during crushing, the greater the extent of the reaction is due increased surface area of silica available for reaction.

Alkalies

The potential for ASR increases with the alkali content of the concrete. This is because the hydroxide ion concentration (alkalinity, pH) increases as the alkali content increases. Portland cement is usually the primary source of alkalies in concrete. Although these alkalies are reported as Na_2O and K_2O the readily soluble portions in the cement generally occur as alkali sulfates. Other internal sources include mineral and chemical admixtures; some aggregates (typically glassy volcanic rocks) and mix water. Deicing chemicals and seawater are external sources which supply alkalies to the pore solution.

Moisture

Water as a constituent of the pore solution in concrete is necessary for ASR to proceed. Concrete which is allowed to dry after proper curing and will remain dry in service is unlikely to be affected by ASR. However, concrete members with large cross-sections may retain enough residual moisture to cause reactivity even though they are in a dry environment.

Concrete which is in contact with ground or water or is periodically wetted will most likely contain sufficient moisture to cause reactivity. Moisture in excess of 80 percent relative humidity in concrete is necessary to cause the gel to swell [23]. A certain level of protection can be provided to concrete by decreasing the permeability, and thus the availability of external moisture to the system.

Fly Ash, Ground Granulated Blast-Furnace Slag, and Silica Fume

Fly ash, ground granulated blast-furnace slag (GGBF), and silica fume can be effective in preventing deleterious expansions resulting from ASR. The primary constituent in each of these materials is amorphous or glassy silica. Because they are fine grained they react very rapidly with the hydroxide ions in the pore solution forming a compound similar to that formed by the hydration of portland cement. This compound fills in spaces between the hydrated cement grains reducing the permeability of the concrete. It effectively ties up the alkalies thus reducing the potential for ASR.

Identification of ASR Distress in Concrete

There are three commonly used ways to identify the occurrence of ASR in a concrete structure. Visual signs include expansion and exudation of gel. Signs of expansion include cracks, closing of joints, and displacement of pavement in severe cases. The cracks will be either longitudinal or map-type. SHRP Handbook [25] is a very good photographic guide for showing ASR cracking in pavement.

The examination of hardened concrete by procedures in ASTM C 856 [11] petrographic practice can identify ASR. A gel fluorescence test was developed under SHRP to identify ASR gel in hardened concrete [25].

Evaluation of Materials and Environment

There are several methods for evaluating aggregates. They include:

Guide for Petrographic Examination of Aggregates for Concrete ASTM C 295 [11],

Test Method for Potential Reactivity of Aggregates (Chemical Method) ASTM C 289 [11],

Test method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-bar Method) ASTM C 227 [11], and

Proposed Test Method for Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction (ASTM Proposed Method).

Wherever possible the results of the above tests should be used in conjunction with information obtained from the service record of the aggregates over a period of years in similar concrete exposed to comparable service conditions.

Alkalies are present in varying amounts in all portland cements primarily in readily soluble sulfate compounds. The alkali content is calculated on an oxide basis and reported as Na_2O equivalent. A limit of 0.06 percent Na_2O equivalent is listed in the optional chemical requirements of the portland cement specification ASTM C 150 [11]. this limit should be specified when portland cements may be used with aggregates which could be potentially deleteriously reactive. However, the 0.60 percent limitation on the alkali content does not always provide protect from ASR. Several factors play a role in such cases:

Alkalies are concentrated in the concrete due to cyclic wetting and drying,

Alkalies are contributed from other ingredients,

Alkalies are contributed from external sources, and high cement contents.

Blended hydraulic cements specified in ASTM C 595 [11] includes an optional mortar expansion requirement using test method ASTM C 441 [11] to evaluate effectiveness in preventing expansions due to ASR. The procedure discussed in the following paragraph on testing and evaluation is suggested. Fly ash specified in ASTM C 618 [11] should also be evaluated as discussed in the paragraph on testing and evaluation. Class F fly ashes generally have a lesser amount of available alkalies and contain larger amounts of silica and alumina which make them more effective than Class C fly ashes in minimizing the alkali-silica reaction. Silica fume is effective in reducing expansions caused by ASR in concrete provided the silica fume is completely distributed throughout the mixture during the concrete mixing. GGBF slag specified in ASTM C 989 [11] should also be evaluated as discussed in the paragraph on testing and evaluation.

Chemical admixtures have not been identified as being a significant contributor to ASR. However, chloride ions may increase the potential for expansion of ASR gel. Research has found the introduction of lithium salts prevent deleterious ASR. This was first revealed by McCoy and Caldwell [26] and extensive confirming research was recently done by SHRP [23].

Testing and Evaluation

Each material considered for use should be tested and evaluated in accordance with the appropriate ASTM procedure by a laboratory qualified to do the work.

ASTM C 441 [11] is the test method for evaluating the effectiveness of fly ash, GGBF slag, and silica fume in reducing expansions caused by ASR. The procedure is similar to ASTM C 227 [11] but uses pyrex glass as a standard reactive aggregate. A criteria of a maximum of 0.02 percent expansion at 14 days is the recommended limit. This test method and criteria should not be used because of difficulties in reducing expansions below this limit.

An alternative is to determine the expansion of a control and test mixtures. Each mixture uses 900 grams of pyrex glass aggregate as specified in ASTM C 441 [11]. The amount of mixing shall be that needed to produce a flow of 110 to 115 as determined by ASTM C 109 [11]. The control mixture shall be made with 400 grams of portland cement with an equivalent alkalies of 0.50 to 0.60 percent. The test mixture shall be made with a portland cement with a total alkali content exceeding 0.08 percent and blended with a fly ash, GGBF slag or silica fume or with a blended hydraulic cement. The total weight of the cement plus fly ash, GGBF slag, or silica fume or blended hydraulic cement shall be 400 grams. Except for proportioning of the mortar, prepare, store, and measure the specimens as required by the applicable sections of ASTM C 227 [11]. The 14 day expansions of the test mixture shall be no greater than the control mixture.

It has been proposed by others that the proposed ASTM test method be used to test the effectiveness of fly ash, GGBF slag, or silica fume or blended hydraulic cements [23].

Mechanism of Alkali-Carbonate Reaction

Alkali-carbonate reaction (ACR) occurs between the hydroxide ions in the alkaline pore solution in concrete and carbonate aggregates which are characterized by the presence of discreet crystals of dolomite. The reaction is believed to involve dedolomitization producing calcium carbonate and alkali carbonate, while releasing magnesium ions which combine with hydroxide ions to form brucite. In a subsequent process, the alkali carbonate reacts with calcium hydroxide to form calcium carbonate and alkali hydroxide. Thus the alkalies which originally involved in attacking the dolomite are released back into the pore solution for further assault. The expansion which accompanies this reaction is not completely understood, is believed to be related to the swelling of the interstitial clay caused by association with alkaline pore solution [27].

Factors Contributing to Alkali-Carbonate Reactivity

Aggregates which are susceptible to ACR seem to exhibit a characteristic lithology. They consist roughly equivalent amounts of calcite and dolomite with an insoluble residue of 10 to 30 percent consisting primarily of clay minerals. They are usually dark colored, fine grained, and exhibit conchoidal fracture. The distinctive microtexture identifiable in thin sections is a rhombs of dolomite, roughly 25 microns across, floating in a fine grained matrix of calcite and clay.

The amount of ACR expansion is directly related to maximum size of the aggregate particle and the amount of reactive constituent in the rock. As either increases, the amount of concrete expansion increases. Factors that cause ASR, i.e., source of alkalies and moisture are also the same factors that cause ACR.

Identification of Alkali-Carbonate Reactivity Distress In Concrete

ACR causes expansions of concrete and, as such, the outward manifestations of distress are similar to ASR or any other process which causes differential volume change in a mass. Most evident will be map and pattern cracking. If the pavement is retrained in a particular direction, the cracking will align itself in that direction. Signs of expansion include closing of joints, offsets or misalignments, crushing of concrete, and pavement blowups. Petrographic examination by ASTM C 856 [11] has to be used to confirm expansion distress is caused by ACR.

Evaluation of Materials

ASTM C 289, C 227, and proposed ASTM test method [11] are appropriate for determining ACR. Petrographic examination of the aggregate by ASTM C 295 [11] and the Handbook of Concrete Aggregates [28] should be used. Rocks suspected of being reactive on the basis of compositional and textural characteristics should be subjected to further testing by the rock cylinder test ASTM C 586 [11]. The concrete prism test, ASTM C 1105 [11] is a method to determine the potential of ACR in concrete. However, this test method is time consuming requiring 6 to 12 months for results. Service record may also be evaluated.

Minimizing Potential for Alkali-Carbonate Reactivity Distress

When the concrete's service environment is conducive to ACR and the aggregate to be used is potentially reactive, several factors should be considered to reduce the risk of deleterious expansion.

1. The amount of expansion is directly related to the amount of reactive rock in the aggregate.
   a. Carbonate rocks are stratified. If layers containing the reactive lithology can be identified, they can be avoided in the quarrying process.
   b. Reactive rocks can be blended with non-expansive aggregates.

2. The amount of expansion is directly related to the top size of the aggregate. Reduce the size of the reactive rock.

3. The amount of expansion is directly related to alkali content of the cement. Use low alkali cement and use lower than 0.06 percent Na_2O equivalent, if available.

4. The use of fly ash, GGBF slag, silica fume or blended hydraulic cement does not appear to be effective in controlling ACR.

ABRASION RESISTANCE

Introduction

Abrasion resistance is defined by the ACI committee on terminology as "ability of a surface to resist being worn away by rubbing and friction" [29]. Abrasion of pavements may result from vehicular traffic on highways and warehouse floors. The use of studded tires or chains may accelerate the wear of concrete road surfaces by attrition, scraping and percussion.

Materials related factors which may reduce the resistance of concrete to abrasive action should be considered in design and construction of pavements. Frequently the failure of concrete to resist abrasion can be traced to cumulative effects such as inadequate compressive strength, excessive air content, soft or resistant aggregates, inadequate bond strength of the aggregate, improper cuing or finishing, or overmanipulation of fresh concrete surface. Reducing the wear resistance of concrete involves the surficical hardened cement paste, which initially resists abrasive forces. As the paste is worn away, coarse aggregate ultimately is exposed. As the pavements wear adequate friction resistance may be lost, which is necessary to reduce the hazard of vehicle skidding under wet conditions.

Compressive Strength

Whitte and Backstrom [30], as researchers before and after them, consider the compressive strength the most important factor responsible for the abrasion resistance of concrete. Studies indicated 34 MPa concrete develops twice as much resistance to abrasion as 20 MPa concrete [31]. ACI 201.2R [6] recommends that in no case should concrete compressive strength be less than 27.4 MPa and that strength level be attained by a low water-cement ratio and the lowest consistency for proper placing and consolidation.

Aggregate Selection

Next in importance is the quality of the fine and coarse aggregates. Smith [32] concluded no correlation exists between concrete abrasion resistance and coarse aggregate test results by ASTM Test Method C 131 [11]. However, Concrete resistance can increase by using hard, dense aggregate, such as traprock, granite, or silica.

Testing

There are three ASTM test methods that may be used for testing for abrasion resistance; one is recommended for use in evaluating pavement concrete. ASTM C 779 [11] has three procedures: procedure A is a revolving disk machine which operates by sliding and scuffing of steel disks in conjunction with abrasive grit; procedure B is a dressing-wheel machine which operates by impact and sliding of a steel dressing wheel; procedure C is a ball-bearing machine which operates by high-contact stresses, impact and sliding friction from steel balls. ASTM STP 169 B [33] recommends the ASTM C 779 [11] for pavement concrete as follows: procedure A for light to medium traffic, procedure B for heavy tire and steel wheel traffic, and procedure C for heavy steel and track vehicles.

Skid Resistance

The problem of friction resistance to skidding has been studied for many years. ASTM [33] has over twenty specifications, test methods and practices which focus on how to measure traveled surface characteristics.

ACI 201.2R [6] discusses the skid resistance of concrete pavements as depending on the surface texture. Two types of texture are involved; the macro-texture (large scale) resulting from irregularities "built in" at the time of construction; and micro-texture (small scale) resulting from hardness and type of fine aggregate used in the concrete. ACI 221R [18] discusses the properties of various aggregates to polishing and frictional resistance.

Construction Practices

With a given concrete mixture, compressive strength at the surface can be improved by:

Avoiding segregation,

Eliminating bleeding,

Properly timed finishing, and

Proper curing.

SUMMARY

Concrete pavements can be designed and constructed for long-term performance without deteriorating provided the materials selection, traffic predictions, and maintenance all conform to adequate standards. Evaluation of concrete materials during the project design will provide information for specification requirements and construction practices to assure concrete pavements will perform in service. Concrete materials engineering is necessary to negate concrete durability problems.

The air-void system for the concrete materials specified must be adequate for resistance to freezing and thawing of the paste. The aggregates must be evaluated to determine that they are non-frost susceptible.

If the aggregates available for the project are potentially alkali-silica or alkali-carbonate reactive, the specifications must include provisions to negate reactivity in service.

Deicing chemical usage has to be known to prevent scaling or recognize an external source of alkalies which may contribute to alkali reactivity.

The compressive strength of the concrete and the aggregate quality and properties have to be adequate to provide abrasion resistant concrete.

Testing and good construction practices must be followed to assure long-term pavement performance without concrete durability problems.

The construction of quality concrete highways and the process that assures achievement of such quality have to be the concern of all involved in the process. Quality assurance is too important to be left to those responsible for the acceptance of the product. Quality is really the business of everyone associated with design, construction operation, and maintenance of the concrete pavement.

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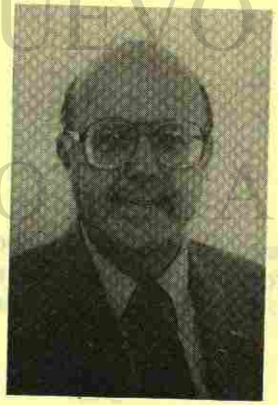

Joseph F. Lanomd,
Consulting Engineer,
Springfield, Virginia, USA

CONCRETO REFORZADO CON FIBRA DE ACERO Y LANZADO

por

Peter C. Tatnall

Sinopsis: Este reporte describe la tecnología y usos de los concretos hechos con cemento Portland y reforzado con fibra de acero. Las propiedades mecánicas, en particular la ductilidad, dureza, resistencia al impacto, resistencia a la flexión y corte, asi como la tolerancia a la fatiga son mejoradas por la adición de fibra de acero. Se discuten estas mejoras y otras propiedades comunes. Se describen las técnicas de mezclado y proporcionamientos de las mezclas, colado y acabado.

Se describen las aplicaciones en todo el mundo del concreto reforzado con fibra de acero, incluyendo pavimentos, pisos industriales, estructuras marinas, concreto lanzado, unidades prefabricadas y refractarias. Se discute brevemente la información y acercamientos sobre diseño disponible.

Palabras clave: Aplicaciones, diseño, durabilidad, pisos, propiedades mecánicas, proporcionamientos de las mezclas, concreto lanzado, especificaciones, concreto reforzado con fibra de acero, dureza.

STEEL FIBER REINFORCED CONCRETE AND SHOTCRETE

by

Peter C. Tatnall

Synopsis: This report describes the technology and uses of steel fiber reinforced portland cement concretes. Mechanical properties, particularly ductility, toughness, impact resistance, flexural and shear strength, and fatigue endurance are improved by the steel fiber addition. These improvements are discussed along with other typical properties. Techniques for mixing and mixture proportioning, placing and finishing are described.

Worldwide applications of steel fiber reinforced concrete including pavements, industrial floors, marine structures, shotcrete, precast units and refractories are described. Available design information and approaches are briefly discussed.

Keywords: Applications, Design, Durability, Floors, Mechanical Properties, Mix Proportioning, Shotcrete, Specifications, Steel Fiber Reinforced Concrete, Toughness.

Pete Tatnall is Technical Service Manager - Dramix Steel Fibers, Bekaert Corporation, Marietta, Georgia. He is a BSCE graduate of Rensselaer Polytechnic Institute and has over 28 years experience in the concrete industry.

Pete currently serves as Chairman of ASTM Subcommittee C09.42 on Fiber Reinforced Concrete and as a member of ASTM Subcommittees on Shotcrete and Concrete Reinforcement. He also serves on the Executive Subcommittee of ACI Committee 544 on Fiber Reinforced Concrete and as Chairman of its Subcommittee A on Steel Fiber Reinforced Concrete, and is a member of ACI Committees 302, 360 and 506.

He is a member of the Transportation Research Board, the American Society of Civil Engineers and the American Underground Space Association.

INTRODUCTION

Steel fiber reinforced concrete (SFRC) is concrete made of hydraulic cements containing fine or fine and coarse aggregate and discontinuous discrete steel fibers. In tension, SFRC fails only after the steel fiber breaks or is pulled out of the matrix.

Properties of SFRC in the freshly mixed state and the hardened state, including durability, are a consequence of its composite nature. The mechanics of how the fiber reinforcement strengthens concrete, extending from the elastic precrack state to the partially plastic post-cracked state, is a continuing research activity. A current approach to the mechanics of fiber reinforcing develops a crack arrest mechanism based on fracture mechanics that relates the energy to extend a crack and debond the fibers in the matrix to the properties of the composite.

Application design procedures for SFRC generally follow the strength design methodology described in ACI 318 with some modifications as outlined below. Good quality and economic construction with SFRC requires that approved mixing, placing, finishing, curing and quality control procedures be followed. Generally, equipment currently used for conventional concrete construction does not have to be modified for SFRC construction.

SFRC has advantages over conventional concrete for many end uses in construction. One example is the use of SFRC for industrial floor construction. Labor and material costs for placing mesh or reinforcing bars may be eliminated, concrete quantities may be reduced and control joint spacing may be increased. Other applications are presented in this report.

Definition of Fiber Types

Steel fibers for concrete reinforcement are defined as short, discrete lengths of steel, have a length/diameter ratio from about 30 to 100 or more, any of several cross-sections, and are sufficiently small to be randomly dispersed in an unhardened concrete mixture using usual mixing procedures.

ASTM A 820-90 [1] classifies steel fibers based on the product used in their manufacture:

Type I	Cold Drawn Wire
Type II	Cut Sheet
Type III	Melt-Extracted
Type IV	Other

and provides for straight or deformed fibers, minimum tensile strength of 50,000 psi (345 MPa), a 90 degree bending test and tolerances on the length, diameter and aspect ratio (fiber length divided by the fiber diameter or equivalent fiber diameter).

Manufacturing Methods

Straight (non-deformed) steel fibers are produced by cutting or chopping wire, by shearing steel sheet or by a chatter machining process which produces elongated chips with irregular surfaces. Deformed fibers are produced by full-length crimping of wire or the wire residue from the steel wool production process, bending the ends of wire, flattening the ends of wire or sheet, and flattening wire along its length. Alloyed and stainless steel fibers are also produced by the melt extraction process where a rotating wheel contacts a molten metal surface and lifts off liquid metal and rapidly solidifies it into fibers. These fibers have an irregular surface and a "kidney" shaped cross-section.

History

Research on closely spaced wires and random metallic fibers in the later 1950's and early 1960's led to patents on SFRC based on fiber spacing and on bond and aspect ratio [2]. Applications of SFRC since the mid 1960's have included highway and airfield pavements, industrial floor slabs and overlays, refractory materials, shotcrete tunnel linings and repairs and concrete precast products. The first commercial SFRC pavement in the United States was placed in August, 1971, at a truck weighing station near Ashland, Ohio [3].

The usefullness of SFRC has been aided by other new developments in the concrete field such as high range water reducing admixtures. These developments and new applications as outlined below have increased the use of steel fibers to an estimated 110,000 tons (100,000 metric tons) per year in 1992.

PHYSICAL PROPERTIES

Fiber Properties

The fiber tensile strength, stiffness and the ability of the fibers to bond with the concrete are important fiber reinforcement properties. Because bond is dependent on the deformation characteristics and the length versus the diameter of the fiber, a convenient parameter describing a fiber is its aspect ratio as defined above. Typical aspect ratios range from about 30 to 100, while length dimensions range from 0.75 to 3 in. (19 to 76 mm).

Steel fibers have a relatively high tensile strength, generally 50,000 to 175,000 psi (345 to 1,200 MPa), and modules of elasticity, 29 million psi (200 GPa). Their specific gravity is 7.85; they are protected from corrosion in the alkaline environment of the cementitious matrix and long-term loading does not adversely influence the mechanical properties of steel fibers. In particular environments such as high temperature refractory applications, the use of stainless steel fibers may be required.

Properties Of Freshly Mixed SFRC

The properties of SFRC in the freshly mixed state are influenced by the aspect ratio of the fiber, the fiber geometry, its volume fraction, the matrix proportions including the maximum size aggregate (MSA) and the fiber-matrix interfacial bond characteristics [4].

In the typical ranges of volume fractions used for cast-in-place SFRC (0.25 to 1.5 volume %), the addition of steel fibers reduces the measured slump of the composite as compared to a non-fibrous mixture up to 50%. Since compaction by mechanical vibration is recommended in most SFRC applications, assessing the workability with either the Vebe consistometer, described in British Standards Institution Standard BS 1881, or by ASTM C 995 Inverted Cone Time [5] is recommended in lieu of the conventional slump measurement. Studies have established that a SFRC mixture with a relatively low slump can have good compaction properties (workability) under vibration [6]. Once appropriate workability is established, the consistency of SFRC can be monitored with the slump test [7]. Slump loss characteristics with time for SFRC and non-fibrous concrete are similar [8],

In addition to the above considerations, the clumping together or "balling" of the fibers must be avoided. The tendency of a SFRC mixture to produce balling of fibers in the freshly mixed state has been found to be a function of the maximum size aggregate and the overall combined fine and coarse aggregate gradation used, the aspect ratio of the fibers, the volume fraction, the fiber shape and the method of introducing the fibers into the mixture. The larger the MSA and aspect ratio, the less volume fraction of fibers can be added without the tendency to ball. Guidance for determining the above factors for adequate fresh and hardened properties is given in the section on Preparation Technology below.

Properties of Hardened SFRC

The mechanism of steel fiber reinforcement of the cementitious matrix in concrete has been extensively studied in terms of the resistance of the fibers to pullout from the matrix resulting from the breakdown of the fiber-matrix interfacial bond. As a consequence of the gradual nature of fiber pullout, steel fibers impart post-crack ductility to the cementitious matrix which would otherwise behave in a brittle manner.

Improvements in ductility and strength depend on the type and volume fraction of fibers present. Fibers with enhanced resistance to pullout are fabricated with a crimped or wavy profile, surface deformations, or improved end anchorage provided by hooking, teeing or end enlargement. These types are more effective than equivalent straight uniform fibers. Fibers with a high aspect ratio also enhance resistance to pullout. Consequently, the amount of these fibers (deformed and high aspect ratio) required to achieve a given level of improvement in ductility and strength is less than the amount of equivalent straight uniform fibers [9].

Steel fibers improve the ductility of concrete in all modes of loading, but their effectiveness in improving strength varies between various loading modes. Various mechanical properties are briefly discussed below:

Improvements in Strength--Improvements in the ultimate strength of SFRC composites varies between compressive, direct tension, flexure, torsion, shear, fatigue and impact as discussed below:

Compressive--Steel fibers usually have only a minor effect on compressive strength, slightly increasing or decreasing the results for 6 x 12 in. (150 x 300 mm) cylinders. Since smaller cylinders give higher strengths for conventional concrete and promote fiber alignment in SFRC, small cylinders with long fibers may give unrealistically high strengths. Cubes have also been used for compressive testing, but the relationship between cube strength and cylinder strength has not been determined for SFRC.

Flexural--Increases in the ultimate flexural strength or modules of rupture of SFRC are substantially greater than in tension or in compression because ductile behavior of the SFRC on the tension side of the beam alters the normally elastic distribution of stress and strain over the member depth. The altered stress distribution is essentially plastic in the tension zone and elastic in the compression zone, resulting in a shift of the neutral axis toward the compression zone [10].

Although some early studies showed that flexural strength can be more than doubled with about 4 volume % fibers in a sand-cement mortar, it is recognized that the presence of coarse aggregate coupled with normal mixing and placing considerations limits the maximum fiber content to about 1.5 volume %. A summary of corresponding strength data [11] shows that the flexural strength of SFRC can be about 10% to 70% more than that of a similar non-fibrous mixture in the normal third-point bending test, ASTM C 1018 [12]. The use of long fibers in thin sections can produce much higher strength increases because of beneficial fiber alignment.

Direct Tensile--In direct tension, the improvement in strength is significant, with increases on the order of 30% to 40% reported for the addition of 1.5 volume % of fibers in mortar or concrete. The split cylinder tensile test is not appropriate for SFRC since the stress distribution after first-crack is unknown. The relationship between direct tensile, split tensile and modules of rupture is unknown for SFRC.

Shear and Torsion--Steel fibers generally increase the shear and torsional strength of concrete, although there are little data dealing strictly with the shear and torsional strength of SFRC as opposed to that of reinforced beams made with a SFRC matrix and conventional reinforcing bars. There is some laboratory evidence that steel fibers may be an economical alternative by replacing some or all of the stirrups in reinforced concrete beams and/or beam-column joints.

Improvements in Ductility--Improvements to the ductility of SFRC can be characterized in a number of ways as discussed below:

Impact Behavior--The behavior of concrete reinforced with various types of steel fibers and subjected to impact loads induced by explosive charges, drop-weight impact machines, modified charpy machines or dynamic tensile and compressive loads has been measured in a variety of ways. It has been found that the peak loads in dynamic flexure for SFRC were about 40% higher than for non-fibrous concrete and that the steel fibers increased the fracture energy under impact by a factor of 2.5 to 3.5. When SFRC is compared under impact loading versus static loading, increases in peak loads up to 3 times and up to 5 times for increases in fracture energy, have been reported [13].

Fatigue Behavior--Experimental studies show that for a given type-amount of steel fiber, there is a significant increase in flexural fatigue endurance with increasing percentage of steel fibers. Depending on the fiber type and concentration, a properly designed SFRC mixture will endure unlimited flexural loading repetitions (over 500,000 cycles) when loaded to 65% to 90% of its static flexural strength [14]. This compares to a level of loading of 50% to 55% for non-fibrous concrete.

Toughness--Early in the development of SFRC, toughness was recognized as the characteristic which most clearly distinguishes SFRC from concrete without steel fibers. For purposes of this report, toughness is defined as the amount of energy needed to produce a specified damage condition or complete failure of the material, usually by separation into two or more parts.

Under impact conditions, toughness can be qualitatively demonstrated by a recent industrial accident where a 500 pound (227 kg) wrecking ball was dropped onto a six year old, 6-inch (15 cm) thick SFRC slab-on-grade and only left a small depression in the surface. A non-fibrous slab would certainly have cracked and perhaps shattered.

Under slow flexure conditions, toughness can be qualitatively demonstrated by observing the flexural behavior of simply supported beams. A concrete beam containing steel fibers suffers damage by gradual development of single or multiple cracks with increasing deflection but retains some degree of structural integrity and post-cracking resistance to load even with considerable deflection, 0.12 inch (3mm) or more. A similar beam without fibers fails suddenly at a small deflection, 0.002 inch (0.05 mm) or less, by separation into two pieces.

These two simple examples of toughness serve not only to identify the characteristic of toughness, but also exemplify the two categories of testing techniques for quantifying toughness: namely, techniques involving high-rate application of load (impact) or a single slow-rate application of load.

Since many of the applications of SFRC involve flexural loading, standardized systems for determining toughness using flexural loading have been developed; ASTM C 1018 and JCI SF-4 [12, 15]. In these systems a measure of toughness is derived from analysis of the load-deflection diagram as indicated in Figure 1. Details of these methods along with a discussion of their merits and drawbacks are presented by Johnston [16] and Gopalaratnam, et.al. [17]. ASTM C 1018 I_5, I_{10} or I_{20} toughness indices are routinely specified for SFR-Shotcrete tunnel linings in North America, and JCI SF-4 Equivalent Flexural Strength or ASTM C 1018 $R_{10,50}$ values are specified for slab-on-grade design.

Other Hardened Properties--The changes or lack of changes in the properties of SFRC as compared to similar concretes without steel fibers listed below are for steel fiber contents less than 1.5 volume percent.

- Creep and Free Shrinkage: No significant effect.
- Modules of Elasticity and Poisson's Ratio: No significant effect.
- Thermal Conductivity: Small increases.
- Electrical Conductivity: No significant effect.
- Abrasion Resistance (low impact): No significant effect.
- Restrained Shrinkage Cracking: Significant reduction in crack widths, five-fold [18].

Durability

Freezing and Thawing--All the well-known practices for making durable concrete apply to SFRC. For freezing and thawing resistance, the same air entrainment criteria should be used as is recommended in ACI 201.2R.

Corrosion of Fibers--Experience has shown that for well compacted concrete with a 28-day compressive strength over 3,000 psi (21 MPa), corrosion of steel fibers will be limited to the surface skin of the concrete. Once the surface fibers corrode, there is no propagation of the corrosion much more than 0.10 inch (2-3 mm) below the surface, typically carbonation depth. This limited surface corrosion exists even when the concrete is highly saturated with chloride ions [19]. Since the fibers are short, discontinuous and rarely touch each other, there is no continuous conductive path for stray or induced currents or currents from electromotive potential between different areas of the concrete.

Long-term laboratory and field testing of cracked SFRC in an environment containing chlorides has indicated that cracks in concrete can lead to corrosion of the fibers passing across the crack if the crack is wider than 0.004 in. (0.1 mm) [20]. If these cracks are limited in depth, the consequences of this localized corrosion may not be structurally significant.

PREPARATION AND PLACING TECHNOLOGY

Mixing of SFRC can be accomplished by several methods with the choice of method depending on the job requirements and the facilities available. It is important to have a uniform dispersion of the fibers and to prevent the segregation and balling (clumping) of the fibers during mixing.

Balling of the fibers during mixing is related to a number of factors. The most important factors are the aspect ratio of the fiber, the volume percentage of the fibers, the maximum size and gradation of the aggregates and the method of adding the fibers to the mixture. As the first three of these factors increase, the tendency for balling increases. The "Guide for Specifying, Mixing, Placing, and Finishing Steel Fiber Reinforced Concrete," ACI 544.3R, provides additional information.

Mix Proportions

Compared to conventional concrete, many SFRC mixtures are characterized by higher cement content, higher fine aggregate content and decreasing slump with increasing fiber content. Conventional admixtures and pozzolans are commonly used in SFRC mixtures for air entrainment, water reduction, workability, shrinkage control, set control and economy.

A mix proportioning procedure is provided by Schrader and Munch which has been used for paving and the repair of hydraulic structures [21]. Experience has shown that if the combined fine and coarse aggregate gradation envelopes as shown in Table 1 are met, the tendency to form balls is minimized and workability is enhanced. Recommendations for trial mixes and the maximum fiber content for good workability are available from most steel fiber manufacturers. Once a mixture has been selected, it is advisable that a field batch be processed prior to the actual start of construction with the mixing and placing equipment that will be used on the project.

Mixing Methods

It is very important that the fibers be dispersed uniformly throughout the mixture. This must be accomplished during the batching and mixing phase. Several mixing sequences have been successfully used, including the following:

1. Add the fibers to the truck mixer after all other ingredients have been added and mixed. Steel fibers should be added to the truck mixer hopper at the rate of about 100 lbs. (45 kgs.) per minute, with the mixer rotating at full speed. The fibers should be added in a clump-free state so that the mixer blades can carry the fibes into the mixer. The mixer should then be slowed to the recommended mixing speed and mixed for 40 to 50 revolutions. Steel fibers have been added manually by emptying the containers into the truck hopper, or via conveyor belt or blower. Using this method, steel fibers can be added at the batch plant or on the job site.

2. Add the fibers to the aggregate stream in the batch plant before the aggregates are added to the mixer. Steel fibers can be added manually on top of the aggregates on the charging conveyor belt or via another conveyor emptying onto the charging belt. The fibers should be spread out along the conveyor belt to prevent clumping.

3. Add the fibers on top of the aggregates after they are weighed in the batcher scale. The normal flow of the aggregates out of the weigh batcher will distribute the fibers throughout the aggregates. Steel fibers can be added manually or via conveyor belt.

SFRC delivered to projects should conform to the applicable provisions of ASTM C 1116 [24]. For manual steel fiber charging methods workers should be equipped with protective gloves and goggles. The method of introducing the steel fibers should be proven in the field during a trial test batch.

Placing and Finishing

All the well-known recommended practices for placing finishing and curing conventional concrete apply to SFRC. For most applications consolidation of SFRC by mechanical vibration is recommended. Also the use of wooden floats is generally not recommended as they tend to pull up the fibers. Further guidance may be found in "Guide for Specifying, Mixing, Placing, and Finishing Steel Fiber Reinforced Concrete," ACI 544.3R.

DESIGN CONSIDERATIONS

The designer may best view SFRC as a concrete with increased strain capacity, impact resistance, energy absorption, fatigue endurance and tensile strength. The increase in these properties will vary from nil to substantial depending on the type and quantity of fibers used.

Several approaches to the design of members with SFRC are available. These are based on conventional design methods generally supplemented by special procedures for the fiber contribution. Additional information can be found in "Design Considerations for Steel Fiber Reinforced Concrete," ACI 544.4R.

In applications where the presence of continuous tensile reinforcement is not essential to the safety and integrity of the structure, e.g., floors on grade, pavements, overlays, ground support and shotcrete linings, the improvements in flexural strength, impact resistance, toughness, and fatigue performance can be used to reduce section thickness, improve performance, or both.

Guidance for the design of floors-on-grade is available from fiber manufacturers, the Battelle Memorial Institute [25], the CUR Recommendation 10 [26], and the FAA [27]. These methods take advantage of the improvements to properties described above.

APPLICATIONS

The applications of SFRC will depend on the ingenuity of the designer and builder in taking advantage of the improvements to the mechanical properties of this composite material. The uniform dispersion of fiber throughout the concrete provides isotropic strength properties and the ability to re-distribute stresses not common to conventionally reinforced concrete.

Cast-in-Place SFRC

Many cast-in-place SFRC applications involve slabs-on-grade, either in the form of pavements or industrial floors. As early as 1983, 22 airport paving projects had been completed in the United States, and over 200 million square feet (18.6 million square meters) of industrial flooring have been constructed throughout the world.

An example of SFRC industrial floors is the 1.5 million sq. ft. (140,000 sq.m.) Chrysler Jefferson North Assembly Plant in Detroit, Michigan. These floors are 5 and 6.5 inches (127 and 165 mm) thick, and contain 0.38 volume percent, or 50 lbs. per cu. yd. (30 kg. per cu.m.) of 2.4 in. (60 mm) long deformed fibers. The owner reports substantial savings from this contractor proposed alternate to the original design [28].

In 1984, 500,000 sq. ft. (46,000 sq. m.) of 4 in. (100 mm) thick SFRC was placed as a replacement of the upstream concrete facing placed in 1909 at the Barr Lake Dam near Denver, Colorado. The SFRC mixture contained 0.6 volume percent, or 80 lbs. per cu. yd. (47 kgs. per cu.m.) of 2.4 in. (60 mm) long deformed fibers, and 1-1/2 in. (38 mm) maximum size aggregate. The SFRC was pumped to a slip-form screed to pave the 47 feet (14 m) high, 2.5 to 1 slope facing. [29].

Other applications of cast-in-place SFRC include:

- Repairs and new construction on major dams and hydraulic structures to provide resistance to cavitation and large debris impact [22].
- Repairs and rehabilitation of marine structures such as piling and cassions [19].
- Bonded overlays in industrial floors and highways.
- Both conventional and latex-modified SFRC bridge deck overlays in the U.S. and Canada.
- Slip-formed, cast-in-place tunnel linings.
- Impact resistant turbine test facility encasement.

Precast SFRC

Many precast applications for SFRC make use of the improvement in properties such as impact resistance or toughness. Other precast applications use steel fibers to replace conventional reinforcing in utility boxes and septic tanks. Some recent applications are cited:

- Dolosse: 30,000 cu.yd. (22,900 cu.m.) of SFRC were placed in over 1,500 - 42 ton (38 MT) dolosse by the Corps of Engineers in Northern California. SFRC was specified in lieu of conventional reinforcing bars to improve the wave impact resistance of the dolosse [30].

- Vaults and Safes: Since 1984, most of the vault and safe manufacturers in North America have used SFRC in precast panels that are then used to construct vaults. Wall thicknesses have been reduced by up to two-thirds over the cast-in-place methods and still are tough enough to resist penetration.

- Mine Crib Blocks: These units, made with conventional concrete masonry machinery, are routinely supplied throughout the world for building roof support structures in coal mines. Steel fibers are used to increase the compressive toughness of the concrete to allow controlled crushing and thus prevent catastrophic failures [31].

- Precast Garages: SFRC is used in Eruope to precast complete automobile garages for single family residences.

Shotcrete

The first practical application of steel fiber reinforced shotcrete (SFRS) was a trial use for rock slope stabilization in 1974 along the Snake River in Washington [32]. Since that time, many applications have been made in slope stabilization, in ground support for hydroelectric, transportation and mining tunnels, and in soldier pile retaining walls as concrete lagging.

A recent example of SFRS use in tunnels is the twin highway tunnels at Cumberland Gap between Kentucky and Tennessee in Eastern United States [33]. These tunnels are 4,100 ft. long (1,250 m) 40 ft. wide (12.2 m) and 30 ft. high (9.1 m) running 40 ft. (12.2 m) apart through Cumberland Mountain which is mostly quartz sandstone, shale, siltstone, mudstone and coal. These tunnels were constructed by S.A. Healy/Lodigiani-JV using the principles of the NATM approach to excavation and ground support. This technique involves allowing the ground to deform slightly after excavation so that it can relieve stresses and essentially support itself. In order to succeed, this approach requires a thin, ductile initial lining to provide initial support in blocky, fractured rock. This lining must be able to support localized roof falls and be able to deform with the ground. The ability of SFRS to carry substantial load after cracking is the ideal material for this lining.

Specifications required the 4 inch thick (100 mm) SFRS in the initial linings to have a flexural strength of 700 psi (4.8 MPa), and a Residual Flexural Strength of 420 psi (2.9 MPa) at 7 days. The Residual Flexural Strength was defined as the strength capacity of the ASTM C 1018 test beam after deforming to 5.5 times the first-crack deformation, or to 0.013 inches (0.33 mm) in this case.

Quality control testing of one series of 606 ASTM C 1018 test beam specimens (202 sets) show the following mean results:

Flexural Strength:	870 psi	(6.0 MPa)
Standard Deviation:	73 psi	(0.5 MPa)
Residual Flexural Strength:	507 psi	(3.5 MPa)
Standard Deviation:	82 psi	(0.6 MPa)

These results are considered outstanding when the method for obtaining specimens is considered. 30 by 30 by 1-inch thick (76 by 76 by 25 cm) test panels were shot from production SFRS at the face of the tunnel excavaions, and moved from the face to a curing shed outside within 8 hours in the bucket of a rubber-tired front end loader. Beam specimens were sawed from the panels at an age of 4 days and then cured in water.

The Contractor selected a wet-process SFRS mixture which included silica fume, high range water reducers and 90 lbs. per cu. yd. (53 kgs. per cu.m.) of 1.2 inch (30 mm) long bent-end steel wire fibers primarily for the economics involved versus reinforcement with welded wire fabric, but safety was a major concern. The SFRS was applied with robotic shotcrete machines which allowed tunnellers to always be protected by the shotcrete lining in the tunnel. If WWF had been used, they would have been exposed to unsupported ground while placing the mesh. Over 39,000 cubic yards (29,800 cubic metres) of SFRS was placed in the initial lining requiring over 3.5 million pounds (1.6 million kgs.) of steel fibers. Both tunnels were holded through in 1992 without a lost time accident.

In addition to ground support, SFRS applications include thin-shell domes shot on inflatable forms, artificial rockscapes, repair and reinforcing of structures such as lighthouses, bridge piers and abutments, port facilities, channel linings, and lining oil storage caverns. Additional references and more information may be found in "State-of-the-Art Report on Fiber Reinforced Shotcrete," ACI 506.1R.

SIFCON - Slurry Infiltrated Fiber Concrete

Slurry Infiltrated Fiber Concrete (SIFCON) is a type of fiber reinforced concrete in which formwork molds are filled to capacity with random-oriented steel fibers and the resulting fiber network is infiltrated by a cement-based slurry. Infiltration is usually accomplished by gravity flow aided by light vibration.

SIFCON composites differ from conventional SFRC in at least two respects: they contain a much larger volume fraction of fibers (up to 12 volume percent) and they use a matrix consisting of very fine particles. As such they can be made to exhibit outstanding strengths and ductilities as listed below:

- Compressive: strengths to 20 ksi (140 MPa), and strain capacities to 10 percent.
- Tensile: strengths to 6 ksi (41 MPa), and strains to 2 percent.
- Moduli of Rupture: up to 13 ksi (90 MPa).
- Shear: strengths of up to 4 ksi (28 MPa).

Since SIFCON is not inexpensive, only applications requiring very high strength and toughness have so far benefitted from its use. These include impact and blast resistant structures, refractories, protective revetments and taxiway and pavement repairs.

Refractories

Steel fibers have been used as reinforcement in monolithic refractories since 1970. Steel fibers have been added to refractory concretes to provide improvements in resistance to cracking and spalling in applications where thermal cycling and thermal shock have limited the service life of the refractory. The presence of fibers acts to control the cracking in such a way that cracks having relatively large openings are less frequent and crack-plane boundaries are held together by fibers bridging the crack-plane. Fracture toughness is a method of characterizing this property, and a convenient technique used involves the measurement of a flexural toughness index (ASTM C 1018) [12].

Both carbon steel and stainless steel fiber reinforced refractories have shown excellent performance in a number of applications including ferrous and nonferrous metal production and processing, petroleum refining, rotary kilns used for producing portland cement and lime, coal-fired boilers and municipal incinerators.

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Table 1 Recommended Combined Aggregate Gradations for Steel Fiber Reinforced Concrete [22,23]

PERCENT PASSING FOR MAXIMUM SIZE OF:

US STANDARD SIEVE SIZE		3/8"	1/2"	3/4"	1"	1-1/2"
in.	(mm)	10 mm	13 mm	19 mm	25 mm	38 mm
2	51	100	100	100	100	100
1-1/2	38	100	100	100	100	85-100
1	25	100	100	100	94-100	65- 85
3/4	19	100	100	94-100	76- 82	58- 77
1/2	13	100	93-100	70- 88	65- 76	50- 68
3/8	10	96-100	85- 96	61- 73	56- 66	46- 58
# 4	5	72- 84	58- 78	48- 56	45- 53	38- 50
# 8	2.4	46- 57	41- 53	40- 47	36- 44	29- 43
# 16	1.1	34- 44	32- 42	32- 40	29- 38	21- 34
# 30	0.6	22- 33	19- 30	20- 32	19- 28	13- 27
# 50	0.3	10- 18	8- 15	10- 20	8-20	7- 19
# 100	0.15	2 - 7	1 - 5	3 - 9	2 - 8	2 - 8
# 200	0.08	0 - 2	0 - 2	0 - 2	0 - 2	0 - 2

Note: Aggregates should be well graded from the largest to the smallest size. Aggregate should not vary from near the maximum allowable percent passing one sieve to near the minimum allowable percent passing the next sieve size.

Peter Tatnall,
Technical Service Manager
Dramix Steel Fiber
Bekeart Corporation, Marietta,
Georgia, USA.

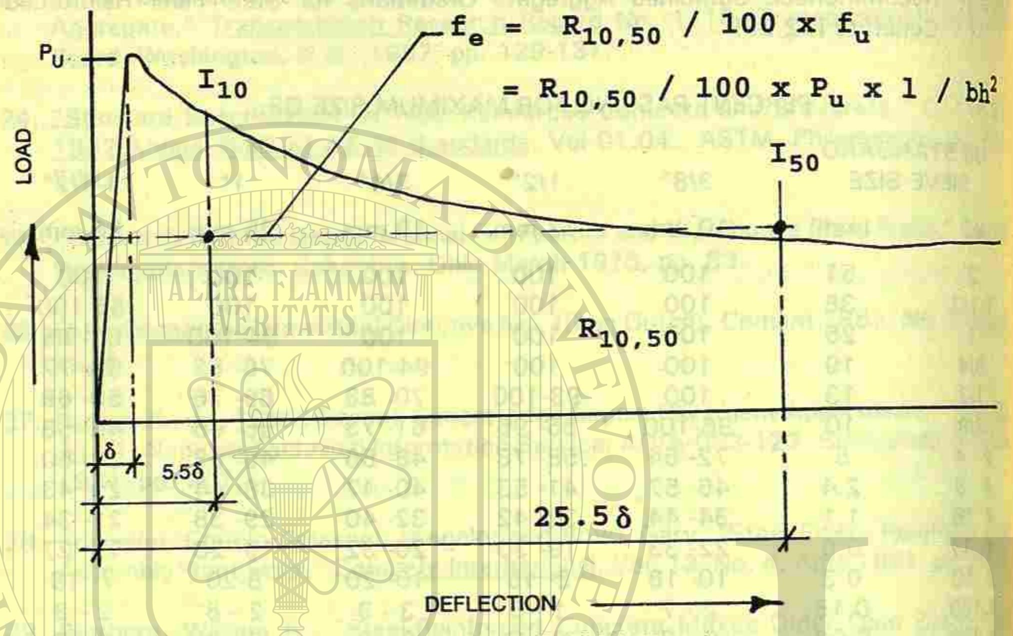

Fig. 1 (a). ASTM C 1018 Toughness

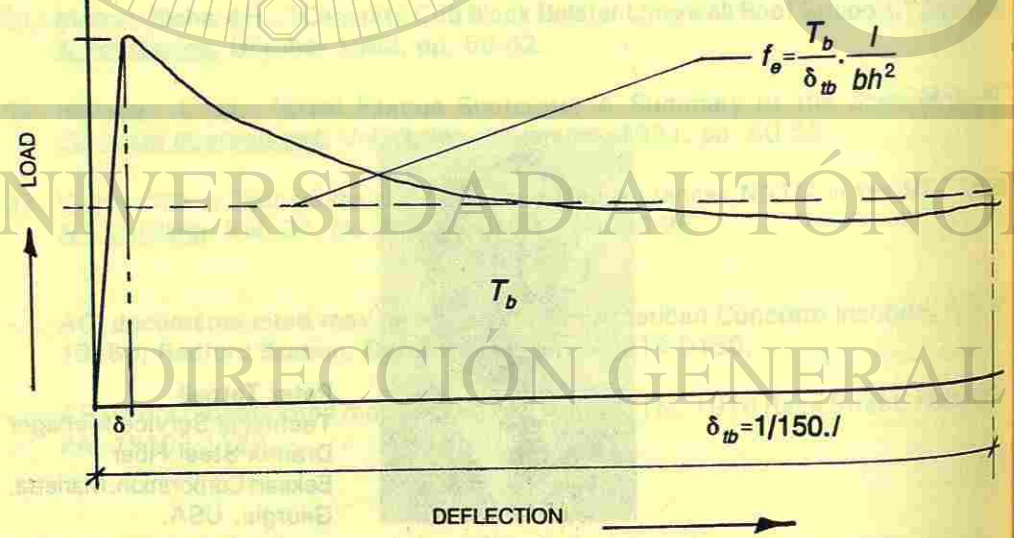

Fig. 1 (b). JCI (JSCE) SF-4 Toughness

ROLE OF PREMIXED CONCRETE IN DURABILITY

EN LA DURABILIDAD

by

por

Marcos José Faradji Capón

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Synopsis: In general, premixed concrete producers are professional enough and customers adequate to offer their products for their specific climatic agents and environmental structural requirements. However, if buyers do not specify their needs to the producer and contractor does not interprete that idea, probably he can not request to the producer the durability quality specification needed.

Contractors frequently ask for high slump concrete, which makes easy its placing without taking into account that this is accomplished adding more water, cement and aggregate, which cause contraction and concrete damage, obtaining inadequate structures. That is they are in good condition just for the opening ceremony.

In this paper, specifications to be taken into account by designers and producers are presented.

Keywords: Concretes, premixed, durability, blending, aggregates, petrographic test.

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Contractors frequently ask for high slump concrete, which makes easy its placing, without taking into account that this is accomplished adding more water, cement and fine aggregate, which cause contraction and concrete damage, obtaining **Inauguration Structures**. That is, they are in good conditions just for the opening, and are damaged later.

In this paper, specifications to be taken into account by designers and producers are presented.

Keywords: Concrete, premixed, durability, bleeding, segregation, owner needs, petrographic test.

PAPEL DEL CONCRETO PREMEZCLADO EN LA DURABILIDAD

por

Marcos José Faradji Capón

Sinopsis: En general los productores de concreto premezclado son lo suficientemente profesionales y éticos, y por tanto se preocupan de ofrecer al cliente un producto de resistencia adecuada a los requerimientos estructurales y de resistencia adecuada a los agentes climáticos y del medio ambiente, sin embargo no son adivinos y si el propietario no trasmite al constructor sus necesidades y el constructor no interpreta adecuadamente esa idea, es poco probable que se le solicite al productor y se le especifique claramente cuales son las cualidades de durabilidad que requiere el concreto.

Es común que el constructor solicite concretos de alto revenimiento para facilitar la colocación sin considerar que estos se logran con más agua, más cemento y más finos los cuales provocan contracciones y con ello deterioro del concreto, obteniendo OBRAS de INAUGURACION. Sólo están bien el día de la inauguración y se destruyen en poco tiempo.

Se presentan las especificaciones que deben tomar en cuenta los diseñadores y los productores.

Palabras clave: Concreto; premezclado; durabilidad; sangrado; segregación; necesidades del propietario, examen petrográfico.

Marcos José Faradji Capón.- Estudios: Fac. de Ingeniería- UNAM (1956-1960) Ingeniero Civil 21 Agosto 1961, División del Doctorado (1962-1963) Maestría en Ingeniería 9 de Marzo de 1965 Especialidad en Estructuras. Portland Cement Asociatión Skokie, Illinois 1972 Tecnología Avanzada del Concreto.
Labores Docentes: 1961-1967.- UNAM Prof. de Dibujo y Mecánica de Materiales, 1964-1981.- Universidad Iberoaméricana Prof. de Dibujo y Mec. Materiales, Desde 1981.- UIA Prof. Especificaciones, Control de Calidad y Tecnología del Concreto Maestría en Construcción., 1976-1993.-UNAM Prof. Control de Calidad Centro de Educación Continua.
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INTRODUCCION

Para iniciar este trabajo es necesario definir claramente este concepto en su relación con el concreto hidráulico. Cuando alguien piensa en estructuras de concreto, considera que son estructuras eternas, luego simplemente que son estructuras que durarán toda la vida y el durarán incluye que estarán en perfectas condiciones durante toda la vida del individuo, por fijar un plazo durante ochenta años, sin embargo en esta forma de pensar no se está tomando ningún criterio ingenierîl que incluya resistencia, costo y tiempo.

Si partimos de la siguiente expresión:
Para que un artículo sea considerado de CALIDAD, este debe desarrollar sus cualidades al utilizarse en forma adecuada, dentro de la vida útil que se le ha fijado al mismo y al costo más bajo que pueda resultarle al productor.

Esta expresión es la que en los últimos 30 años se ha tratado de difundir relacionándola al concreto hidráulico y principalmente al concreto premezclado, que es el que adecuadamente dosificado y adecuadamente mezclado es entregado al constructor por un productor.

El constructor conoce las cualidades que deberá cumplir el producto y tanto el constructor como el propietario desean que la estructura funcione en óptimas condiciones siempre, sin embargo normalmente no le comunican al productor adecuadamente esos deseos y necesariamente esta falta de comunicación es la causante de muchos problemas.

En general los productores de concreto premezclado de México y recientemente también los de Estados Unidos de Norteamérica, son lo suficientemente profesionales y éticos, y por tanto se preocupan de ofrecer al cliente un producto de resistencia adecuada a los requerimientos estructurales y de resistencia adecuada a los agentes climáticos y del medio ambiente, sin embargo no son adivinos y si el propietario no tránsmite al constructor sus necesidades y el constructor no interpreta adecuadamente esa idea, es poco probable que se le solicite al productor y se le especifique claramente cuales son las cualidades que requiere el concreto.

Las cualidades del concreto son tanto en estado fresco como en estado endurecido y no basta con decir:
quiero $36\ m^3$ de concreto 250-N-20-14 a las 8 y cada 30 mín., es necesario especificarle al productor la características que realmente requieren.

Nuestras normas aunque son las más adelantadas del mundo adolecen de varios errores que el autor desea exponer en este documento.

La norma NOM C155 del Concreto Premezclado en lo que se refiere a la resistencia para el grado de cálidad B, fija 3 condiciones mismas que se deben cumplir:

1. No más del 10 % de los valores de pruebas de resistencia podrá ser inferior a f'c.
2. No más del 1% de los promedios de 3 pruebas consecutivas podrá ser inferior a f'c.
3. No más del 1% de los valores de pruebas de resistencia podrá ser inferior a f'c - 35 kg/cm^2

Si bien la segunda condición es la más exigente para productores con S menor de 35 kg/cm^2, en todos los casos se está exigiendo valores mínimos y no se está proponiendo valores promedio deseables y variabilidad máxima permitida.

Si un productor surtiera para un concreto f''c 250 varias unidades con resistencia 400 o 450, lo cual causaría trastornos en la colocación y terminado, esto no sería reclamable y si intercalara una unidad de resistencia 50 kg/cm^2 ó de 0 kg/cm^2 tampoco sería reclamable.

La norma nacional no exige como la C94ASTM al productor de concreto premezclado entregar un concreto con temperatura adecuada al elemento por colar y a la temperatura ambiente que prevalece en el lugar del colado.

La norma nacional no prohibe el uso de cloruro de calcio como acelerante. La norma nacional no exige el uso de inclusores de aire cuando el concreto quedará sometido a congelación y deshielo.

En ningún párrafo de la norma se limita el que el concreto pueda tener sangrado siendo este concepto el más delicado, principalmente en lo que se refiere al comportamiento estructural y la durabilidad de la estructura y esto no se distingue en las pruebas de resistencia que se desarrollan para la verificación de la calidad, pues al presentarse el sangrado simplemente los cilindros se encogen y por menor esbeltez son más resistentes, la relación agua-cemento en el interior disminuye y por tanto son más resistentes (sólo en la parte superior aumenta la relación A/C), el agua en la superficie permite unos segundos más de curado y por tanto más resistencia, sin embargo, en la estructura, esta fuga de agua provoca encogimiento y un agrietamiento y falta de continuidad del elemento y necesariamente el comportamiento estructural ya no es el adecuado y este montón de pedazos de concreto ensartados con varilla será más deformable y más susceptible al deterioro, porque a través de las grietas entrarán las substancias agresivas del ambiente dejando sales que expanden, oxidando el acero que ya no es de refuerzo y al expandir el óxido de fierro destruyendo más al concreto.

Lo que dicen los buenos libros: Diseño y Control de Mezclas de Concreto de la PCA y los del ACI al respecto es muy poco.

pág. 2.- Del de PCA "El concreto debe ser trabajable, pero no se debe segregar ni sangrar + excesivamente. El sangrado es la migración del agua hacia la superficie superior del concreto recién mezclado provocada por el asentamiento de los materiales sólidos - cemento, arena, piedra - dentro de la masa".

Del ACI 301²
Especificaciones para el concreto estructural en edificios.

Pág. 18
" 3.8..- Selección de proporciones.

3.8.1.1 Las proporciones de materiales para concreto deben establecerse a fin de asegurar:
a) Adecuada trabajabilidad y consistencia a fin de permitir que el concreto se coloque rápidamente en las cimbras y alrededor del refuerzo sin que haya una segregación excesiva o sangrado en las condiciones de colocación que se utilicen".

Y si además se observa lo indicado en:

"3.5 Revenimiento

A menos que se especifique o se permita otra cosa, el concreto debe proporcionarse y fabricarse a fin de que tenga un revenimiento de 10 cm o menos, si la (consolidación)* compactación se logrará por medio de vibradores, y de 12.7 cm o menos si la (consolidación)* compactación va a lograrse por otros métodos que no sean por vibración.

Puede permitirse una tolerancia de hasta 2.5 cm por encima de lo indicado, para una colada en cualquier serie de al menos cinco coladas que se sometan a prueba".

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* mala traducción

De lo anterior se desprende que lo que actualmente hacemos en México al pedir concretos con revenimiento superior a 12 cm para disminuir el trabajo del equipo de transporte, bombeo y del personal de obra, va en contra de la cualidad importante que se debe fijar al concreto y es la de DURABILIDAD.

Del ACI-301.

"3.4 Durabilidad

3.4.1 El concreto de peso normal que esté sometido a exposiciones especialmente destructivas (distintas al desgaste o a las cargas) como son el congelamiento y el deshielo, climas severos o agentes químicos descongelantes, debe ser con aire (atrapado) incluido y adecuarse a los límites de contenido de aire de la tabla 3.4.1.

La relación agua - cemento no debe exceder el 0.53 por peso. El concreto ligero para tales condiciones debe contener un 6 + /- 2 por ciento del aire total, cuando el tamaño máximo nominal de los agregados sea mayor a 10 mm o de un 7 + /-2 por ciento del aire total cuando el tamaño máximo nominal sea de 10 mm o menos conforme se determine por el método volumétrico ASTM C173. Las proporciones deben seleccionarse a fin de que proporcionen una resistencia (específica)* especificada de f'c de 210 kg/cm² o más.

3.4.2 El concreto de peso normal para el caso de partes de la estructura que requieren ser impermeables, debe tener una relación agua-cemento que no exceda el 0.50 si va a estar expuesto al agua dulce o de 0.45 si va a estar expuesto al agua de mar. El cocreto ligero para tales aplicaciones debe propocionarse a fin de producir una resistencia a la compresión (específica) f'c de al menos 260 kg/cm² si está expuesto al agua dulce y de 280 kg/cm² si está expuesto al agua de mar.

3.4.3 Para concreto que este expuesto a concentraciones dañinas de sulfatos y otros agentes químicos agresivos, deben usarse cementos de tipo II o tipo IV. Para el concreto de peso normal que se use en este tipo de aplicaciones, la relación agua - cemento no debe exceder el 0.44 por peso. Para concreto ligero, las proporciones deben seleccionarse a fín de que se produzca una resistencia a la compresión (especifica) f'c de al menos 280 kg/cm².

3.4.4 Para el concreto presforzado y para todo aquel concreto en que vayan a ahogarse elementos de aluminio o metales galvanizados, debe demostrarse mediante pruebas, que el agua de mezcla, incluyendo aquella proporcionada por los agregados o cualquier aditivo que se utilice no contiene cantidades dañinas de iones de cloro.

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* *mala traducción*

"Durabilidad

Diseño y Control de Mezclas de Concreto, Portland Cement Asociation, pág. 189.

La durabilidad se refiere a la capacidad del concreto para resistir el deterioro proveniente del medio ambiente o del servicio que reciba.

Un concreto diseñado adecuadamente deberá perdurar sin fallas de importancia durante su vida de servicio.

Para cubrir los requisitos del proyecto, asegurarse o verificar la durabilidad o determinar el efecto de ciertos ingredientes o procedimientos de operación sobre la durabilidad, se pueden efectuar varias pruebas.

La resistencia a la congelación y deshielo	ASTM C666, C671, C682
La resistencia a los descascaramientos provocados por el uso de productos descongelantes.	ASTM C672
La protección contra la corrosión y la actividad de corrosión del acero de refuerzo.	ASTM C876
La reactividad álcali-agregado	ASTM C227, C289, C342, ASTM C441 C586
La resistencia a los sulfatos.	ASTM C452, C1012.
La resistencia a la abrasión.	ASTM C418. (chorro de arena) C779 (disco giratorio de rueda rectificadora y de baleros, de bolas) C994(cortadora rotatoria)"

Tecnología del Concreto³
<u>Adam M. Neville, III</u>

Cap. 7

"Un concreto es durable si soporta las condiciones para las que ha sido diseñado, sin deterioro y durante un período de varios años.

La durabilidad depende del ambiente al que se exponga el concreto o de causas internas del concreto mismo.

Las causas externas pueden ser:
- físicas
- químicas
- o mecánicas

Se pueden deber al intemperismo
a las temperaturas externas
a la abrasión
a la acción electrolítica
al ataque de líquidos o gases naturales o industriales

La extensión del daño producido por dichos agentes dependerá en gran parte de la calidad del concreto, aunque en condiciones extremas cualquier concreto se deteriorará.
Las causas internas:
- reacción álcali - agregado
- los cambios de volumen por diferencia de las propiedades del agregado y la pasta de cemento.
- permeabilidad del concreto (determina la vulnerabilidad del concreto a los factores externos)".

El productor de concreto premezclado, normalmente planifica la operación de sus plantas estudiando con suficiente anticipación el mercado y una vez que considera la factibilidad de instalarse inicia la investigación de las fuentes de abastecimiento de agregados, cemento y aditivos.

Estudia los diferentes bancos de agregados, su calidad y potencialidad y solamente cuando conoce suficientemente las cualidades de los mismos, en el caso de los agregados y el cemento que no se presenten reacciónes químicas que causen deterioro y en base a mezclas de prueba para resistencia a la compresión, flexión, tensión y sometidas a las diferentes acciones de intemperismo acelerado y conociendo el historial de servicio de los concretos producidos en la zona, está en posibilidad de analizar el costo de los insumos y definir un precio adecuado al servicio que prestará en la zona.

El constructor normalmente no cuenta con las facilidades y la capacidad económica para realizar todas estas investigaciones para una obra en particular a menos de que esta obra sea de gran magnitud (superior a los 10,000 m³) o que esté asesorado por un laboratorio de prestigio que cuente con banco de datos de las características de los agregados de la zona y que disponga de especialistas responsables de hacer una dosificación que no se limite a dar la resistencia a la compresión del concreto, sino que tome en cuenta todas las condiciones a las que estará sometida la estructura y todas las condiciones en que se construirá la misma.

Los mismos productores de concreto premezclado se asesoran de los laboratorios de prestigio privados, de los laboratorios serios de las instituciones de enseñanza y de los laboratorios de las dependencias oficiales que han experimentado en esas zonas.

Muy delicada es la actividad de un laboratorio y de su personal que tiene que indagar la suficiente información por parte del comprador o usuario en donde y bajo que circunstancias estará sometido el concreto que se pretende diseñar.

Es básico conocer la exposición del concreto a los procesos de congelamiento y deshielo, bajo que condiciones iniciales de humedad, a que temperatura, etc conocer si el concreto estará expuesto a condiciones atmosféricas, aguas o suelos que contengan sustancias químicas agresivas, si estas están en solución y si además ejerciendo presión, y a que temperatura y si hay humedecimiento y secando cíclico.

Conocer si además el concreto estará expuesto a abrasión, es decir desgaste producido por fricción o frotamiento y si estos efectos ocurrirán combinados con congelamiento y deshielo, contacto con sustancias químicas, altas temperaturas y humedecimiento.

Conocer si existirán en el interior del concreto metales que puedan presentar corrosión por las condiciones ambientales y también por el paso de corriente eléctrica y si las sustancias químicas que se usen para proteger un material no se combinen con otro.

Y aunado a esto conocer si los agregados que debieran ser inertes, no puedan en algún momento reaccionar entre sí, con el agua y las sustancias disueltas en ella y con los compuestos del cemento y los que generan durante el fraguado.

Este último aspecto es el más delicado y el que difícilmente un constructor sin experiencia puede conocer o descubrir y que sin embargo una compañía premezcladora seria, puede estudiar a fondo haciendo examen petrográfico del agregado, barras de mortero, prismas de concreto, pruebas químicas de reactividad potencial, examen petrográfico del concreto, investigación del historial de servicio, etc., análisis del contenido de álcalis en el cemento y en el agua para producir concreto.

Sin embargo, de que puede servir tantos estudios si no hay comunicación entre el usuario comprador y el productor del concreto premezclado, y si la comunicación no es clara y completa en que el usuario indique como se usará el concreto y bajo que condiciones quedará sometido.

Las investigaciones realizadas desde 1940 a la fecha si bien son importantes, no han sido definitivas y todavía hay varios mecanismos de deterioro que no son bien conocidos y la ocurrencia de reacciones químicas adversas es cada vez mayor por disponer de menos fuentes de abastecimiento y las existentes están más contaminadas, y a veces contaminadas de sustancias menos estudiadas y también más peligrosas. No es raro contar con agregados afectados por desechos de plantas industriales o nucleares.

Los laboratorios de investigación seguirán estudiando las diferentes fuentes y los diferentes métodos para descubrir los materiales reactivos, sin embargo, cada vez los controles de impacto ambiental provocan modificaciones en las fábricas de cemento que llevan a producir cementos con características diferentes y en algunos casos más reactivos con otras sustancias.

Como ejemplo de lo anterior tenemos que debido a los filtros exigidos en las chimeneas, el contenido de álcalis en el cemento ha crecido y por lo tanto el riesgo de su utilización con agregados reactivos es mayor.

La comunicación que se requiere para lograr el objetivo de la durabilidad es entre el propietario representado por el diseñador, proyectista o Director del Proyecto, el constructor y el productor del concreto premezclado.

Para que un concreto potencialmente durable gracias al buen diseño del premezclador, alcance la durabilidad deseada se requiere que la mano de obra utilizada en la construcción sea la adecuada, es decir colocación sin segregación, compactación adecuada, terminados sin permitir el humedecimiento y un curado adecuado. Es obligación del diseñador exigir un buen concreto al premezclador y exigir al constructor que ese buen concreto no pierda sus cualidades, sino que se incrementen con la mano de obra adecuada.

Hace más de 20 años, comentando con un productor de concreto premezclado de California mencionó que si un cliente solicitaba a su empresa concreto de más de 7.5 cm de revenimiento, su personal le informaba que esa empresa no surtía concreto a ignorantes o gente que no supiera utilizar adecuadamente el concreto y que gracias a esa política prácticamente no tenía problemas de suministro, ni de resistencia, pués generalmente los concretos de alto revenimiento son los que generan más calor, se contraen más, se agrietan más, se desperdician más, hay más probabilidad de variación en la resistencia y de resistencias bajas, más retraso en las entregas y más desorden en la obra. Este problema lo hemos adquirido en México gracias a la popularidad en el uso del bombeo del concreto ya que los que ofrecen ese servicio piden un concreto para que la bomba no se esfuerce y no se desgaste en sus partes y hace que el constructor se olvide de la estructura.

Es usual concretos de 18 cm de revenimiento y además con la característica de "bombeable", es decir con más finos, esto provoca que el concreto requiera más cemento y por tanto cueste más, sin tomar en cuenta que a medida que se ponga más cemento, más agua y más arena se pondrá menos grava y necesariamente la durabilidad se verá afectada.- El comprador sólo se preocupa de la resistencia y sería interesante que un laboratorio de verificación de la calidad en su informe para estos concretos "bombeables" dijera : resistencia excelente, durabilidad NULA, para que así reaccionarán los diseñadores y constructores y tomarán en cuenta las indicaciones de este trabajo y las que en reunioness sobre durabilidad se están presentando.

Desgraciadamente no existen pruebas comunes que midan la durabilidad como se mide la resistencia, pero aún así la resistencia que se mide en los cilindros estandar de la verificación de la calidad, es la resistencia potencial del concreto con el que podemos contruir una estructura, mas no la resistencia del concreto que hemos colocado en la estructura. Para que así sea representativo y aún mejor el de la estructura debemos colocar, compactar, curar y proteger siguiendo las técnicas adecuadas.

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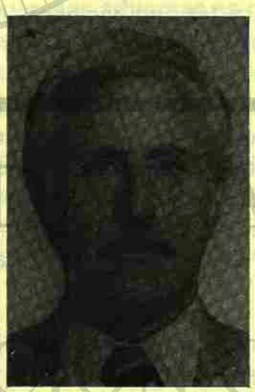


Marcos José Faradji Capón
Director General de
Laboratorio de Alto Nivel en
Calidad, S.A. de C.V.
México, D.F.

IMPORTANCE OF WORKMANSHIP IN DURABILITY AND CONSTRUCTION INSPECTOR CERTIFICATION

by

Lorenzo Flores Castro

Synopsis: Harmful effects for concrete durability which can be cause by poor construction practices resulting from a lack of capability of supervisors are stated. As a solution for this problem, it is presented the ACI program "Supervisors Certification in Concrete Buildings Level II", which in Spanish is doing the ACI Mexico City Chapter.

Keywords: American Concrete Institute, training, certification, durability.

IMPORTANCIA DE LA MANO DE OBRA EN LA DURABILIDAD Y CERTIFICACIÓN DEL SUPERVISOR DE OBRAS DE CONCRETO

por

Ing. Lorenzo Flores Castro

Sinopsis: Se plantean los efectos nocivos que en la durabilidad del concreto pueden ocasionar las malas prácticas constructivas originadas por falta de capacitación de los supervisores. Como solución a ésta problemática, se presenta el programa de "Certificación de Supervisores de Obras de Concreto Nivel II" del ACI que en español, está realizando el Capítulo Cd. de México del American Concrete Institute.

Palabras Clave: American Concrete Institute; capacitación; certificación; durabilidad.

Ing. Lorenzo Flores Castro.- Ingeniero Civil UNAM; Gerente General de INSPECTEC, S.A. de C.V.; Presidente de la Asociación Nacional Laboratorios Independientes al Servicio de la Construcción A.C. (ANALISEC) 1978-1979; Presidente del Comité de Construcción del Sistema Nacional de Acreditamiento de Laboratorios de Prueba (SINALP) 1989-1990; Secretario del Instituto Mexicano de Desarrollo Tecnológico de la Cámara Nacional de Empresas de Consultoría 1989-1991; Consejero de la Cámara Nacional de Empresas de Consultoría 1991-1993; y Secretario de la Comisión Ejecutiva 1992-1994; Presidente del Comité de Certificación del ACI Sección Ciudad de México 1991-1993.

INTRODUCCION

El Comité ACI 201 en su "Guía para la Durabilidad del Concreto" (Guide to Durable Concrete) define a la durabilidad del concreto elaborado con cemento Portland, como su resistencia a la acción del clima, a los ataques químicos, a la abrasión o cualquier otro proceso que cause deterioro.

En el mismo documento se analizan las principales causas del deterioro en el concreto y se proporcionan recomendaciones para prevenirlas.

Como principales causas se discuten la congelación y el deshielo, exposición a sustancias químicas agresivas, abrasión, agregados reactivos, corrosión de acero y materiales ahogados en el concreto.

Sin embargo el comité aclara "que un buen diseño y buenos materiales no garantizan por si mismos que se obtenga un concreto durable; un buen control de calidad y buenas practicas constructivas son absolutamente esenciales para producir concreto durable".

Mucho se ha estudiado y escrito sobre los procedimientos que se deben seguir en cada caso para lograr una estructura duradera, sin embargo se considera que el factor que mayor influencia tiene en la durabilidad del concreto y por lo tanto de las estructuras es la **MANO DE OBRA** y al respecto poco se ha hecho para capacitar adecuadamente al personal que interviene en la construcción de las obras de concreto.

Como ejemplos de los efectos de la mano de obra en la durabilidad del concreto podemos mencionar los siguientes:

Una de las principales causas del deterioro del concreto, es la penetración de soluciones o líquidos abrasivos, ésta penetración depende de la permeabilidad del concreto, la cual a su vez es función directa de la relación agua/cemento, esto es, a menor relación A/C menor será la permeabilidad y por lo tanto mayor la durabilidad, motivo por lo cual cuando el concreto estará expuesto a sustancias agresivas como es el caso de los sulfatos, se fijan valores máximos a la relación agua/cemento. El Comité ACI-309 en su publicación "Standard Practice for Consolidation of Concrete" da recomendaciones respecto a los procedimientos y equipo que debe emplearse para compactar el concreto, sin embargo en las obras es frecuente observar procesos de compactación deficiente, que cuando el vibrado es escaso, seguramente afectará más la permeabilidad que una pequeña variación en la relación Agua/Cemento y si la vibración es excesiva como indica el comité ACI-309, se puede tener otro tipo de problemas que afectan igualmente la durabilidad del concreto como puede ser: asentamiento del agregado grueso, aparición de vetas de arena y en el caso de concretos con aire incluido la pérdida de casi todo ese aire, lo cual puede reducir la resistencia del concreto a la congelación y al deshielo, aún cuando se haya considerado la cantidad de aire incluido recomendable por el Comité ACI-211. Estos procedimientos de compactación, dependen principalmente de la calidad de la mano de obra.

De acuerdo al Comité ACI-308 "Standard Practice for Curing Concrete", la resistencia y durabilidad del concreto se desarrollarán plenamente, solo si se cura de manera adecuada, indicando el Comité los métodos y materiales que deben emplearse; por lo general se ha observado que en gran parte de las obras, el curado es deficiente o no se cura, lo cual al igual que en el caso de las compactaciones es debido a mala calidad en la mano de obra o falta de capacitación del personal responsable.

Otro ejemplo en el que se aprecia la influencia de la mano de obra, es el caso del recubrimiento del acero de refuerzo, que aún cuando en los planos de proyecto se considera el adecuado para que exista la protección necesaria, en la práctica vemos con frecuencia que no se cumple.

Como los ejemplos anteriores se puede mencionar muchos otros, como puede ser el mal almacenaje de cemento y agregados, malas prácticas de dosificación y mezclado, etc., en los que se puede ver la influencia de la mano de obra en la durabilidad del concreto y la necesidad de contar con supervisores certificados para la construcción de obras de concreto.

Certificación ACI

El American Concrete Institute inició en Estados Unidos los programas de certificación de personal, con objeto de ayudar a incrementar la calidad y durabilidad de las obras construidas con concreto.

Los programas de Certificación de personal existentes a la fecha son:

Técnico de campo Nivel I (Muestreo)
Técnico de Acabado de Pisos
Técnico en Pruebas de Laboratorio (Niveles I y II)
Supervisor de Concreto (Nivel II)

Para las condiciones del desarrollo de la industria de la Construcción en México, el Capítulo Cd. de México del ACI consideró conveniente iniciar la certificación de personal con el programa de supervisores de obras de Concreto Nivel II, con la idea de que es inútil preparar buenos laboratoristas cuyo trabajo no sea apreciado adecuadamente en las obras por un buen supervisor.

En México, el Capítulo Cd. de México del ACI inició la certificación de supervisores de obras de Concreto Nivel II en Octubre de 1991, a la fecha (Julio de 1993) se han impartido 9 cursos y certificado a más de 100 supervisores.

Se tiene pensado que antes de que termine éste año, iniciar la certificación de técnicos en pruebas de laboratorio en los niveles I y II.

El programa Mexicano en Español, ha merecido reconocimiento del A.C.I. en una forma más independiente que la de los grupos nacionales en E.U., ya que el Capítulo Cd. de México del A.C.I., está autorizado a hacer en Español, su propio banco de preguntas, acreditar a sus propios examinadores e instructores y fundamentalmente a expedir los **CERTIFICADOS**. Las oficinas centrales del A.C.I. en Detroit, solamente llevan el registro de las personas certificadas.

Debido a que el curso se dirige a personas de un nivel de educación que generalmente no domina el inglés, el programa se pudo realizar gracias a que el Instituto Mexicano del Cemento y el Concreto, ha traducido al Español casi todas las publicaciones del A.C.I.

La Certificación de supervisores de obras de Concreto Nivel II, consiste en evaluar el conocimiento de los candidatos con respecto a ciertos documentos del A.C.I., ASTM (en México Normas Oficiales Mexicanas) y PCA que tienen que ver con manuales de supervisión, prácticas recomendadas, reglamentos de construcción, métodos de ensaye, recomendaciones de diseño, etc.

Para alcanzar la certificación del A.C.I. como Supervisores de Obras de Concreto Nivel II, es necesario satisfacer requisitos no solamente de nivel académico sino también de experiencia laboral.

La construcción de obras modernas a base de concreto implica una serie de tecnologías cada vez más desarrolladas. Los directores de proyectos cada vez más enfocan su atención en aspectos de **CALIDAD, DURABILIDAD, SEGURIDAD Y ECONOMIA**, puntos en los cuales juega un papel importante el supervisor capacitado.

El programa está elaborado como se dijo para mejorar la calidad de las obras de concreto, al evitar los problemas relacionados con la supervisión inadecuada y preparar a la industria de la construcción para una posible certificación obligatoria.

Es evidente que los supervisores certificados tendrán mayores ventajas y más oportunidades para mejorar en un campo tan competido, ya que el Certificado del ACI reconoce que el supervisor ha demostrado su capacidad para aprobar el examen del ACI.

Un supervisor certificado por el ACI representa a un profesional responsable dentro del campo de las obras del concreto. La simple búsqueda de la distinción de contar con el Certificado del ACI, da por hecho que el supervisor es alguien que esta deseando dedicarse a las actividades de supervisión con calidad.

Las especificaciones ACI para el diseño y construcción de obras de concreto son reconocidas y aplicadas en prácticamente todo el mundo y el personal certificado por ésta organización cuenta con el mismo grado de reconocimiento.

El curso de capacitación está diseñado como un curso de la supervisión básica de la construcción del concreto y es el indicado para la preparación del examen, sin embargo el tomarlo **NO GARANTIZA** pasar el examen. En el curso no se pretende enseñar todo lo relativo a la supervisión de concreto, sino más bien enseñar a buscar en la bibliografía donde pueden encontrar la solución a los problemas que se les presentan.

El pasar el examen y ser certificado como Supervisor ACI Nivel II, no significa que ésta persona tiene autoridad para contradecir al proyectista o para imponer su criterio. Se le hace ver que en caso de observar alguna anomalía en el diseño, debe comunicarse con el proyectista (si le es permitido) o bien dejarle anotado en la bitácora correspondiente y hacerle notar sus inquietudes a su supervisor o jefe.

<u>Requisitos para obtener el Certificado.</u>

Para poder obtener el certificado de supervisor de obras de concreto Nivel II se deben satisfacer como mínimo alguno de los siguientes requisitos <u>además de aprobar el examen.</u>

1.- **Licenciatura terminada y dos años de experiencia en la construcción de obras de concreto.**

2.- **Dos años de estudios a nivel licenciatura y tres años de experiencia.**

3.- **Preparatoria, vocacional o similar concluida y cinco años de experiencia.**

Dentro de la experiencia debe incluir:

1.- **Responsabilidad en toma de decisiones y en autoridad.**

2.- **Verificación del cumplimiento de planos, especificaciones y reglamentos.**

3.- **Evaluación de obras de concreto en el campo.**

4.- **Documentación y presentación de informes.**

5.- **Haber participado en las siguientes áreas de supervisión: Cimbras, refuerzo, instalaciones ocultas, muestreo y ensaye de concreto, transporte, colocación y compactación, etc.**

Examen

El examen es escrito, con una duración de 4 horas, consta de 100 preguntas de respuesta múltiple. Cada pregunta contiene 4 respuestas, **pero sólo una es correcta.** Para lograr la certificación se necesita obtener una calificación mínima de 70/100.

El examen es a libro abierto, pero solo se permite el paquete de información que se les ha entregado. Para presentar el examen es necesario mostrar una credencial con fotografía.

En caso de no salir aprobados si se requiere reclamar el fallo o la determinación del examinador, se debe hacer por escrito al ACI Capítulo Cd. de México dentro de los siguientes 60 días, dando a conocer la causa de su inconformidad. De acuerdo a las reglas del programa de certificación no se puede obtener copia del examen, pero se permita que vea el examen en presencia de un representante del ACI Capítulo Cd. de México.

Si durante el examen, el examinador considera que una o más personas han violado cualquier regla del examen, ésta o éstas personas no podrán continuar con el mismo, sin embargo pueden tomar otro examen en un plazo menor a un año y cuando sea ofrecido por el ACI Capítulo Cd. de México, esto se aplica también a los que reprobaron.

El examen no pretende conocer la experiencia de los candidatos, **sino más bien evaluar su pericia para poder encontrar la respuesta y por supuesto su familiaridad con los documentos de consulta.**

No todas las preguntas son fáciles o difíciles, pero se requiere conocimientos en el campo de la construcción de concreto, no se debe actuar a base de experiencia y criterio, sino por el contrario, de acuerdo a lo que dicen los libros y documentos del paquete que se les entregó.

Curso para Supervisores de Obras de Concreto Nivel II.

El programa del curso que comprende toda la información necesaria para realizar y registrar correctamente los resultados de la supervisión previa, durante y después de los colados, se divide en los siguientes módulos:

1.- **Información General Sobre el Programa.**

2.- **Papel del Supervisor de Obras de Concreto.**

a) Proceso de Inspección

- Definiciones.
- Especificaciones/contrato
- Organización

b) Clasificación Funcional.

- Representante de los funcionarios.
- Representante de los contratistas.
- Representante de los fabricantes.
- Representante de organismos gubernamentales.

3.- **Documentos Contractuales.**

a) Especificaciones

- Requisitos del supervisor.
- Documentos relacionados.
- Descripción del trabajo.
- Aseguramiento de la calidad, reglamentos y normas.
- Ejecución de productos.

b) Planos de diseño.

c) Documentos requeridos en las especificaciones.

- Programas.
- Planos de taller.
- Materiales.

4.- **Materiales.**

a) Rellenos y/o subrasante

b) Concreto.

- Principios de calidad del concreto.
- Componentes.
- Proporcionamiento.
- Dosificación y mezclado.

c) Refuerzo.

- Tipo.
- Grados.

d) Elementos ahogados.

e) Selladores de juntas.

f) Agentes de curado.

g) Cimbras.

- Sistemas patentados.
- Cimbras in situ.
- Apuntalamiento y ademado.
- Materiales permanentes.
- Agentes para descimbrar.

5.- Inspección Previa al Colado.

a) Excavación.

b) Apoyo de la cimentación.

c) Preparación de la subrasante.

d) Cimbras.

- Estabilidad.
- Ubicación apropiada.
- Precisión dimensional.
- Preparación de la superficie.

e) Refuezo.

- Tipo y uso.
- Ubicación y espaciamiento.
- Requerimientos de apoyo.
- Empalme y translapes.
- Recubrimiento.
- Limpieza.

f) Juntas formadas.

- Tipo y uso.
- Ubicación.

g) Instalaciones y tapones.

- Tipos.
- Ubicación.
- Tamaño.
- Condición.
- Precauciones.

h) Aprobación final previa al colado.

- Lista de verificación (Check-List).
- Discusión con el Ingeniero de Proyecto.
- Discusión con el Superintendente.

6.- Inspección durante el colado.

a) Preparación.

- Limpieza.
- Humedad.
- Temperatura.

b) Inspección del concreto plástico.

- Procedimeintos.

c) Transporte y colocación.

- Principios generales.
- Equipo y aplicación.

d) Consolidación.

- Métodos.
- Equipo.

e) Acabado.

- Métodos.
- Equipo.
- Resultados finales.

f) Juntas.

- Métodos.
- Programación.

g) Curado y Protección.

- Encharcamiento/inmersión, rociado/neblina, cubiertas saturadas.
- Papel impermeable, láminas plásticas o compuestos formadores de membrana.
- Vapor, serpentinas de calentamiento, calefacción eléctrica.

h) Requerimientos para climas cálidos.

  - Definición de clima caliente.
  - Efectos.
  - Factores adicionales.
  - Propiedades del concreto.
  - Producción y entrega.
  - Colocación y curado.
  - Ensaye e inspección.

i) Requerimientos para climas fríos.

  - Temperatura del concreto.
  - Preparación.
  - Protección.
  - Requerimientos y métodos de curado.
  - Aceleración de la resistencia.
  - Registros de temperatura.

7.- **Inspección posterior al Colado.**

a) Lineamientos básicos para curado y protección.

b) Protección en climas cálidos.

  - Necesidades de curado continuo.
  - Métodos y consideraciones.

c) Protección en climas fríos.

  - Necesidad de protección contra congelación.
  - Métodos y consideraciones.

d) Estimación anticipada de la calidad del concreto.

  - Prueba de curado acelerado.
  - Método del rebote.
  - Método de penetración.
  - Método de extracción.
  - Método de velocidad de pulsos.
  - Método de madurez.

e) Descimbrado.

  - Inspección para obtener un acabado adecuado.
  - Otras consideraciones.

f) Reapuntalamiento.

  - Instalación.
  - Desmontaje.
  - Consideraciones/precauciones.

g) Acabado y reparación de la superficie.

  - Defectos superficiales.
  - Diferentes acabados.

h) Pruebas de aceptación.

  - Obtención de muestras.
  - Ejecución de pruebas de resistencia.
  - Determinación de revenimiento, contenido de aire, peso unitario, etc.

i) Verificación de Tolerancias.

  - Criterio de aceptación, rechazo, reparación; factores relacionados con la resistencia.

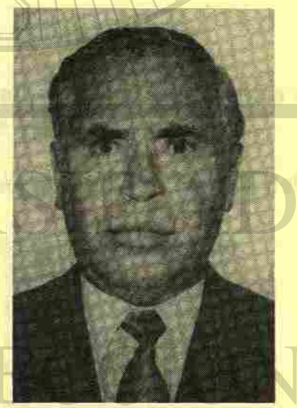
  - Criterios de aceptación, rechazo, reparación; factores ajenos a la resistencia.

8.- **Sistemas y Aplicaciones Especiales.**

  - Inyecciones (lechadas).
  - Concreto Lanzado.
  - Concreto precolado.
  - Concreto postensado.
  - Concreto pretensado.
  - Concreto arquitectónico.
  - Mampostería.

CONCLUSION

En los últimos años, los avances en la tecnología del concreto han sido notables, principalmente en lo que se refiere a las dos características más buscadas, la resistencia y la durabilidad, como lo vemos en el empleo cada vez mayor, de concretos de altas resistencias elaborados a base de micro-sílice y el uso de superfluidificantes. Sin embargo para lograr una aplicación exitosa de éstas tecnologías, es necesario que el personal que interviene en los procesos de producción, transporte, colocación y control, tanto de la parte contratista como de la supervisión, esté debidamente capacitado, de ahí la importancia de los programas de certificación de personal implementados por el American Concrete Institute.



Lorenzo Flores Castro,
Gerente General INSPECTEC, S.A
México, D.F.

CONCRETOS DE ALTO COMPORTAMIENTO: PROPIEDADES MECANICAS Y ASPECTOS SOBRE LA DURABILIDAD

por

R. Pleau, M. Pigeon, A. Delagrave y M. Azabi

Sinopsis: Este trabajo presenta algunos hallazgos de la investigación desarrollada recientemente el la Universidad de Laval, sobre las propiedades del concreto de alto comportamiento (HPC). Se cubren diferentes aspectos del HPC incluyendo la estructura capilar de los poros, su resistencia a la compresión, su resistencia al congelamiento y descongelamiento; la influencia del envejecimiento en su permeabilidad al ion cloro, así como su resistencia al ataque químico. Los resultados reportados aquí ilustran no sólo las ventajas ofrecidas por el HPC en relación con la resistencia mecánica, sino que también las relacionadas con la durabilidad de la estructura del concreto. Se encontró que las propiedades del HPC estaban íntimamente relacionadas con la superficie interna del poro. La selección de los materiales, en particular los cementantes y del agregado grueso, parecen ser de particular importancia. Los resultados de los ensayes también indican que una diferencia relativamente pequeña al decidir sobre el tipo y dosificación de los aditivos o en el método de mezclado, pueden ser los responsables de grandes discrepancias en las propiedades del concreto endurecido. Por lo tanto, se debe tener especial cuidado al diseñar y fabricar el HCP para poder producir un material de alta calidad con propiedades repetibles y reproducibles.

Palabras clave: Concreto de alto comportamiento, poros capilares, resistencia a la compresión, cemento, microsílica, durabilidad al congelamiento, permeabilidad al ion cloro, ataque químico.

HIGH PERFORMANCE CONCRETE: MECHANICAL PROPERTIES AND DURABILITY ASPECTS

by

R. Pleau, M. Pigeon, A. Delagrave, and M. Azabi

Synopsis: The paper presents some findings of the research carried out recently at Laval University on the properties of high-performance concrete (HPC). Different aspects of HPC are covered including their capillary pore structure, their compressive strength, their resistance to freezing and thawing, the influence of ageing on their chloride ion permeability, and their resistance to chemical attack. The results reported here well illustrate the advantages offered by HPC not only as regards with the mechanical resistance, but also as regards with the durability of concrete structures. The properties of HPC were found to be closely related to their internal pore structure. The choice of materials, especially the cementitious materials and the coarse aggregate, appears to be particularly important. Test results also indicate that relatively small differences in the choice and dosage of admixtures, or in the mixing method can be held responsible for large discrepancies on the properties of hardened concrete. Consequently, a great care must be taken in the design and making of HPC in order to produce a high quality material with repeatable and reproducible properties.

Keywords: High-performance concrete, capillary pores, compressive strength, cement, silica fume, frost durability, chloride ion permeability, chemical attack.

Richard Pleau is an Associate Professor at the School of Architecture at Laval University in Québec City, Canada. His research interests are in the field of air entrainment, frost durability, high-performance concrete, and high-performance micro-reinforced cementitious composites.

ACI Fellow **Michel Pigeon** is a professor of Civil Engineering at Laval University, Québec, Canada. His research interests are mainly in the field of concrete durability, concrete repairs and high-performance concrete. He is a Principal Researcher of the Canadian Network of Centres of Excellence on High-Performance Concrete and a member of ACI Committee 201, Durability of Concrete.

Anik Delagrave is a Ph.D. student at Laval University. Her research interests are in the field of microstructure of cement paste, transport phenomenon, nuclear waste management and long-term durability of high-performance concrete.

Maha Azabi is a Ph.D. student at Laval University. Her research interests are in the field of durability of micro-reinforced high-performance cementitious composites.

INTRODUCTION

The advent of commercially available superplasticizer admixtures in the late seventies is mainly responsible for the rapid development of high-performance concrete (HPC) in the last decade [1, 2]. HPC was first used to take advantage of its higher compressive strength but it also offers many other advantages like a higher modulus of elasticity, a higher hydration rate, reduced drying shrinkage and creep deformations and, especially, a better durability [3, 4]. HPC is no longer considered as a laboratory material and its use is expected to grow rapidly in the next years for a wide range of applications.

This paper summarizes the results of some experimental studies carried out recently at Laval University on the mechanical properties and durability of a wide range of high-performance concrete mixtures.

CAPILLARY PORE STRUCTURE

High-performance concrete is obtained by reducing the water to binder[1] ratio (W/B) in order to produce a stronger and less porous cementitious matrix. The improved mechanical resistance of HPC and its higher durability are both closely related to the reduction of the capillary pore volume. But the lowering of the W/B ratio do not only reduces the total pore volume; it also yields to a more finely divided pore network. In ordinary cement pastes (W/B $\geq$ 0,45), many spaces located between cement particles are never filled with hydration products and thus remain as large capillary pores. For pastes having low W/B ratios, the cement particles are closer together and the reduction of the porosity is mostly achieved to the detriment of these larger pores [5].

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[1] The term "binder" includes Portland cement and other cementitious materials used as a partial replacement of Portland cement such as silica fume, fly ash, or blast-furnace slag.

A series of laboratory tests was carried out in our laboratory to study the influence of W/B ratio on the capillary porosity of cement paste. Four different binders were tested: a normal (ASTM Type I) and a high early strength (ASTM Type III) Portland cements with and without silica fume (6% in weight) used as a partial replacement for Portland cement. A granitic sand and a crushed limestone were used as fine and coarse aggregates in a 40/60 relative proportion (in weight). Concrete mixtures were made with three W/B ratios (0.45, 0.35, and 0.25), and their compositions were chosen in order to keep the volume of cement paste almost constant (approximately 35% of the total volume). All mixtures were submitted to the ASTM C 642 water absorption test method and to the AASTHO T227 rapid chloride ion permeability test method after respectively 7 and 28 days of moist curing at 23°C.

Table 1 shows the volume of permeable pores (i.e. pores containing evaporable water) obtained from the water absorption tests. This volume roughly corresponds to the total capillary porosity and, as Table 1 indicates, it decreases with the advance of cement hydration and the lowering of the W/B ratio. The values reported in Table 1 clearly show that Type III Portland cement is more reactive than Type I cement and thus yields a less porous cementitious matrix. Similarly, Table 1 indicates that the presence of silica fume also promotes the hydration process and reduces the total porosity, especially for concretes made with Type I cement and tested at early ages (7 days).

Table 2 summarizes the results of the rapid chloride ion permeability tests. It gives the electrical charge (expressed in Coulombs) passing through concrete specimens submitted to a 60 volt potential during a 6 hour period[2]. Table 2 shows that the chloride ion permeability also decreases with the advance of hydration and the lowering of the W/B ratio. Concretes made with Type III cement are less permeable than those made with Type I cement and the use of silica fume significantly reduces the chloride ion permeability, especially at later ages (28 days).

The chloride ion permeability per unit volume of capillary pores can be easily obtained by dividing the charge passing through concrete (Table 2) by its capillary pore volume (Table 1). This parameter is a good index of the fineness of the pore network because the chloride ion permeability is related not only to the total pore volume, but also to the spatial distribution of capillary pores. For a given pore volume, the mobility of ions, and consequently the chloride ion permeability, will be reduced if this volume is more finely subdivided. Figures 1 and 2 show the relationship between the charge per unit pore

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[2] One side of the concrete specimen is in contact with a 3% NaCl solution and the other side with a 0.3N NaOH solution.

volume and the W/B ratio for concretes made with and without silica fume after 7 and 28 days of moist curing. The curves shown on these Figures clearly illustrate the refinement of the pore network with the advance of hydration (7 vs 28 days of moist curing) and the lowering of the W/B ratio. It is interesting to notice that mixtures made with Type III cement have a slightly finer pore network than those made with Type I cement probably due to the higher fineness of the cement particles (The Blaine fineness was 545 m²/kg for Type III cement compared to 383 m²/kg for Type I cement).

The comparison between Figures 1 and 2 indicates that the presence of silica fume reduces very significantly the chloride ion permeability per unit capillary pore volume. This reduction can be due in part to the lower ionic content of the capillary pore water which increases the electrical resistance of concrete. But the main difference is most probably simply due to the refinement of the pore structure provided by the presence of the silica fume particles. It must be remembered that these particles are about 100 times smaller than cement grains which confer them a great ability to fill small spaces and act as nucleus for hydration products. It is interesting to notice that the presence of silica fume tends to increase the difference between 7 and 28 days of moist curing. It is also very significant to observe that, at 28 days, all silica fume concretes show extremely low values of permeability no matter the type of cement or the W/B ratio.

COMPRESSIVE STRENGTH

The compressive strength of HPC was the object of a large number of research studies reported in the technical and scientific literature. Among the many factors which influence the strength of HPC, the choice of materials is particularly important. Portland cements with a high fineness and a high early strength (such as ASTM Type III) generally provide the best results because they produce a less porous and more finely divided cementitious matrix as described in the previous section. Although it is not absolutely essential, most of the HPC contain silica fume because it significantly improves the quality of the cement paste as well as the quality of the bond at the paste-aggregate interface, and it generally have a beneficial effect on the rheological properties of fresh concrete. The choice of the coarse aggregate is certainly one of the key factor in the production of very high strength concrete. At very low W/B ratios the performance of HPC is often governed by the characteristics of the coarse aggregate which is rarely the case for normal strength concrete. The stronger aggregates are not necessarily the best ones because they are often too stiff and their high modulus of elasticity creates harmful stress concentrations at the paste-aggregate interface.

It is very difficult to assess the quality of an aggregate on the sole basis of its mineral composition. The best way (and certainly the safer) is to identify the good quality aggregates available in the local area and to test them in the laboratory. Such an exercise was done with a number of aggregates of the Québec City area. Concretes with low W/B ratios were made with 6 different coarse aggregates (with a 14 mm maximal nominal size) by using always the same materials in the same proportions. A Type I Portland cement containing 6 to 8% of silica fume was used in conjunction with a granitic sand and a naphtalene based surperplasticizer admixture. The composition and properties of concrete mixtures are given in Table 3. Table 4 summarizes the compressive strength obtained after 48 hours and 28 days of moist curing at 23°C (each number represents the average value obtained for three 150 x 300 mm cylinders). This Table indicates that the best results were obtained with a dolomitic limestone (aggregate C) and the worst ones with a granite (aggregate D) and a limestone (aggregate A). It is funny to notice that aggregate A is the most commonly used by the local concrete producers. For the 0.22 W/B ratio a difference of about 20 MPa is observed between the stronger and weaker aggregate which is quite a significant difference. Figure 3 shows the relationship between the compressive strength and the water-binder ratio for three of these aggregates (A, B, and C). This Figure shows significant differences between the strength obtained with these three aggregates. The reversed order of strength obtained with aggregates A and B at 48 hours and 28 days well illustrates the importance of the paste-aggregate interfacial bond at early ages (a lot of debonding and few broken particles were observed on the ruptured surfaces).

The materials which yields higher compressive strengths are not necessarily the more appropriate ones. Many other factors have to be taken into account such as the workability of fresh concrete, the loss of slump with time, the heat of hydration, the unit cost of concrete, bleeding and segregation, creep and shrinkage, etc. The compatibility between the cement, the superplasticizer, and the other chemical and mineral admixtures was found to be of a major concern and worth a particular attention from the concrete producer [6, 7]. When crushed particles are used as coarse aggregate, the reduction of the maximum nominal size often provides higher compressive strengths probably because the aggregate particles are less affected by the microcracking induced by the crushing process.

HPC is much more sensitive to little differences of mixture composition, mixing procedures, or weather conditions than normal strength concrete, and thus requires a more rigorous control. For example, a series of tests was carried out to study the influence of the superplasticizer (both its nature and its dosage) and the influence of the mixing sequence on the properties of a high performance concrete made with a W/B ratio of 0.35. Concrete mixtures having the same composition (i.e. about 150 kg of water, 425 kg of Type I Portland cement, 772 kg of granitic sand and 1090 kg of coarse aggregates for each cubic meter of concrete) were made with two coarse aggregates (a dolomitic limestone and a granitic gneiss) and three superplasticizers (a melamine and two naphtalene) used at different dosages. Two mixing sequences were tested. The first one consists to mix cement, sand, water, and superplasticizer until the obtention of an homogeneous mortar and to later add the coarse aggregates. In the second method dry materials (cement, sand, and coarse aggregates) were first mixed before the adding of the water and superplasticizer. The results obtained are summarized in Table 5. The 28 days compressive strengths reported in this Table range from 41 to 68 MPa which is quite a large difference for concrete mixtures supposedly similar ! The average resistance is around 60 MPa and it can be reached with all the three superplasticizers used with both coarse aggregates although the data are quite scattered (particularly for the two naphthalene-based superplasticizers). Let us consider, for example, the mixtures made with the dolomitic limestone and the superplasticizer N2. It is surprising to observe that two identical mixtures made with the same dosage of superplasticizer (6.5 L/m³) and the same mixing sequence (A) yields compressive strengths as different as 44 and 53 MPa. Mixtures made with the granitic gneiss and the superplasticizer N1 provide another good example. In that case, the compressive strength vary by about 17 MPa due solely to the mixing sequence used (the results were repeated twice for each mixing sequence). The reason for these discrepancies is still disputed. It was suggested that some internal bleeding beneath the coarse aggregates, or a poor distribution of water through the cement paste could be held responsible for some exceptionally low compressive strengths. This would be mainly observed for concrete mixtures with very high slump values. It is also interesting to notice that, in all cases, a much higher dosage of superplasticizer is required to obtain the same slump value when mixing sequence B is used instead of the sequence A. Further research is still need to better understand, and control, this particular behavior of HPC. However the results reported in Table 5 well emphasize the great care that concrete producers must give to the mixture design and the making of HPC in order to deliver a high quality construction material with always the same properties.

RESISTANCE TO FREEZING AND THAWING CYCLES

In Northern countries, repeated freezing and thawing cycles is one of the main cause of deterioration of concrete structures (along with the corrosion of steel rebars). Damages due to frost action can be divided in two categories: (1) internal microcracking caused by the hydraulic pressures resulting from the force movement of freezable water through the capillary pore structure during freezing, and (2) surface scaling due to freezing in presence of deicing salts [8]. When concrete is exposed to severe climatic conditions, air entrainment is required to protect concrete against freezing [9]. Air entrainment is achieved by adding an air-entraining admixture to the mixing water. Microscopical air voids (with diameters ranging from 10 μm to more than 1 mm) act as vessels to dissipate the internal pressures caused by the flowing of freezable water. These pressures are roughly proportional to the square value of the distance that freezable water must travel through cement paste [10]. The frost resistance is thus strongly related to the air-void spacing factor ($\bar{L}$) which is defined as the average maximum distance between the boundaries of two adjacent air voids. This spacing factor is determined by means of a microscopical examination of polished concrete surfaces according to the ASTM C 457 test method. The concrete is considered frost resistant if its $\bar{L}$ is smaller than a critical value called the critical spacing factor ($\bar{L}_{crit}$). For a given concrete mixture, the critical spacing factor is determined by submitting similar concrete mixtures with different $\bar{L}$ values to 300 repeated freezing and thawing cycles in laboratory (ASTM C 666 test method) and assessing the extent of frost damage by means of length change, resonance frequency, or pulse velocity measurements. The critical spacing factor is a function of many parameters such as the choice of materials, the mixture composition, and the curing period [11].

Table 6 gives the values of $\bar{L}_{crit}$ obtained for a number of concretes made with different binders (Type I and III Portland cement with and without silica fume) and different W/B ratios (0.50, 0.30, and 0.25) which were subjected to 300 freezing and thawing cycles in air or in water after 14 days of moist curing. Concretes having lower values of $\bar{L}_{crit}$ require a higher air content because the air voids must be more closely spaced in order to efficiently protect them against freezing and thawing cycles. Table 6 indicates that for low W/B ratios (0.30 and 0.25), the nature of cement is of a paramount importance. For concretes made with Type III Portland cement it was not possible to determine an $\bar{L}_{crit}$ value since all the mixtures tested were found to be frost resistant (the non air-entrained mixtures have an $\bar{L}$ of about 800 μm). In fact, even non-air entrained mixtures subjected to 750 freezing and thawing cycles in water only 24 hours after casting show no signs of deterioration. Concretes made with Type I Portland cement do not have such a good frost resistance and, for these mixtures, air entrainment is still required even for very low W/B ratios (although their air content may be significantly lower than the value generally recommended for normal strength concretes). The difference between cements is mostly due to their influence on the capillary pore structure (see the previous section devoted to this topic). The lower porosity and more refined pore structure obtained with Type III cement is such that the concretes made with this type of cement contain a negligible amount of freezable water which explains their better frost resistance.

For normal strength concrete, it is generally accepted that a spacing factor lower than about 200 to 250 μm is required in order to provide a satisfactory surface scaling resistance in presence of deicer salts [9, 11]. Most of the time, scaling is a condition more restrictive than internal microcracking since many concrete mixtures with $\bar{L}$ values substantially higher than 250 μm can sustain repeated freezing and thawing cycles without internal damage (see Table 6). Laboratory test results indicate that HPC generally exhibit very low weight losses due to scaling even after 150 daily freezing and thawing cycles irrespective to the W/B ratio, type of cement, and curing period [12, 13]. It thus seems that, contrary to the normal strength concretes, the frost resistance of HPC is mainly governed by the internal microcracking instead of deicer salt scaling. It is clear that it is possible to produce highly frost resistant HPC with lower air contents than normal strength concretes. In some cases it is even possible to produce frost resistant HPC without air entrainment. However, research is still needed in order to propose new guidelines about the characteristics of the air-void system which are required to ensure the frost durability of HPC.

INFLUENCE OF DRYING ON CHLORIDE ION PERMEABILITY

Penetration of chloride ions is one of the most important cause of deterioration of concrete structures and is associated to many detrimental phenomenon such as the formation of chloroaluminates, the corrosion of steel reinforcement, and the surface scaling. Because of its very low porosity, HPC is much less permeable to chloride ions which contributes to improve its durability very significantly. Recently, a research program was carried out in our laboratory to study the influence of drying on the rapid chloride ion permeability of concretes made with four binders (Type I and Type III Portland cement with and without 10% of silica fume) used at three W/B ratios (0.45, 0.35, and 0.25) [14]. All the concrete mixtures were subjected to a 7 day initial curing period and were air-dried for 90 days at different temperature (23, 38, and 110°C) prior to testing. Drying at 23°C and 38°C corresponds to normal conditions for field concretes. Drying at 110°C, however, is an extreme condition and was performed to better discriminate the expected relative performance of the different mixtures under very severe exposure conditions.

Figures 4 and 5 show the results obtained for concrete mixtures made with Type I and Type III Portland cement respectively. As discussed in a previous section, the lower chloride ion permeabilities associated to the lower W/B ratios and the presence of silica fume are due to the refinement of the capillary pore network as well as the reduction of the total pore volume. The results reported in Figures 4 and 5 reveal that drying can increase significantly the chloride ion permeability. This behavior can be partially explained by the internal microcracking resulting from thermal and moisture gradients. But a good part of the difference is most probably due to an important modification of the capillary pore structure. The removal of water from capillary pores is believed to open the thin water-filled spaces which connect the larger capillary pores to one another (this is known as the "ink bottle effect") [15, 16]. Drying is thus expected to yield a coarser pore structure with more interconnected capillary voids. The opening of the pore structure facilitate the migration of gas, liquids and ions through the cementitious matrix by means of different transport mechanisms such as capillary suction, ponding, permeability, osmose, and diffusion. Consequently, drying can be harmful to concrete durability in many different aspects such as deicer salt scaling, chemical attack, corrosion of steel reinforcement, etc.

Some important conclusions can be drawn from the Figures 4 and 5. First of all, concrete mixtures with smaller W/B ratio appear to be less affected by drying especially those made with Type III Portland cement. But even concretes with a 0.25 W/B ratio were affected by drying at 110°C when they do not contain silica fume. However, concretes containing silica fume and having a W/B ratio of 0.25 were not significantly affected by drying, even at 110°C. It is thus possible to make high-performance concretes which are extremely resistant to internal damage caused by drying providing that the mixtures have a very low W/B ratio (about 0.25) and contain silica fume. The beneficial effect of silica fume is probably due to the fact that its capillary volume is very finely divided (see Figure 2) which make him much less vulnerable to the opening of the pore structure.

RESISTANCE TO CHEMICAL ATTACK

The chemical attacks can be roughly divided in two categories: (1) the constituents of cement paste (such as calcium hydroxide) can chemically react with substances present in the surrounding solution to form new swelling hydration products (such as chloroaluminates), or (2) hydration products can be dissolved when exposed to an aggressive environment. Of course, both of these phenomenon can occur simultaneously. Although very few experimental data are available, HPC are expected to be much better resistant to the first type of chemical attack than normal strength concretes because their very low porosity constrain the intrusion of aggressive substances. The resistance of HPC to the second type of chemical attack has been investigated in our laboratory [15]. This resistance is particularly important for special applications such as the making of nuclear

waste containers where the very long term durability (after a few hundred years) is of a paramount importance.

In order to study the resistance of HPC to chemical attack, cement pastes were made with a Type III Portland cement, 6% of silica fume, and two W/B ratios (0.38 and 0.25). After six months of curing in a saturated lime solution, paste disks (70 mm in diameter and 4 mm in thickness) were soaked in three different pH controlled solutions for a period of up to 3 years. The three aggressive solutions were as following: 3% NaCl (by weight) maintained at a pH level of 8.5; 0% NaCl at 8.5 and 0% NaCl at 4.5). It must be remembered that capillary water have a pH value of about 13, and that even pure water (with a pH of about 7) is thus an aggressive, acid solution. At regular intervals, paste disks were removed from the solutions and submitted to scanning electron microscope examinations, X-ray energy dispersion analyses, and mercury intrusion porosimetry.

The results obtained from X-ray energy dispersion are particularly interesting. The concentration of calcium, chlorine, and aluminium were measured at 10 μm intervals along imaginary lines extending from the external surface in contact with the aggressive solution towards the internal part of the disks. The results obtained are shown on Figure 6. The horizontal line drawn on each diagram represents the concentration of calcium in the calcium silicate hydrates (C-S-H). The concentrations above this line correspond to the calcium contained in the calcium hydroxide ($Ca(OH)_2$), and the concentrations below, to the calcium of deteriorated C-S-H.

As can be seen in Figure 6, after three months of exposure to aggressive solutions, the calcium content far from the external surface was found to be similar to that in the control specimens (i.e. specimens tested immediately after the 6 months curing period). Near the external surfaces, however, the calcium content was much lower. The lower calcium content near the surface can be explained by the fact that calcium hydroxide and C-S-H are unstable at a pH lower than about 13. The decalcified zone was more pronounced for the mixtures having a W/B ratio of 0.38, than for the 0.25 mixtures. For example, after three months of exposure, the calcium content for the mixtures in contact with chlorides was lower in a zone covering approximately the first 375 μm for the 0.25 mixture, and 570 μm for the 0.38 mixture. The results also shows that the pH level of the corrosive solution plays an important role on the decalcification process. The influence of the pH level of the corrosive solution confirms the results obtained from previous studies at pH levels of 13 and 11.5 [16, 17]. Mercury intrusion porosimetry measurements indicate that the leaching of calcium also increases the capillary porosity. The pores having a diameter ranging from about 90 to 600 Å seems to be the most affected by this mechanism.

The results clearly indicate that the pH level of the aggressive solution is the most important factor controlling the durability of cement pastes subjected to chemical attack. The W/B ratio does not affect the deterioration processes but only influence the kinetics of these processes. The use of a lower W/B ratio slow down the leaching of calcium.

CONCLUSION

When compared to normal strength concretes, high-performance concrete does not only provides a higher compressive strength, but also an improved durability. The properties of HPC can thus be advantageously used for a large number of applications which required a high quality construction material. Nevertheless, the design of the mixture composition, as well as the batching, delivery, and placing of concrete is more touchy with an HPC than with a normal strength concrete. Consequently, although HPC can be easily and efficiently produced in concrete plants with commonly used equipments, it remains that its production requires a well qualified staff and a higher quality control.

This paper was dealing exclusively with high-performance concretes made with Portland cements and silica fume. But other supplementary cementitious materials, such as fly ashes or blast-furnace slags, can also be successfully used to produce HPC mixtures. Actually, little data are available on that topic although a number of major works have been reported recently [18, 19]. The use of mineral by-products in the making of HPC is expected to grow significantly in the next years. However, research is still needed to better understand the properties of these concretes, especially as regards with their durability.

ACKNOWLEDGMENTS

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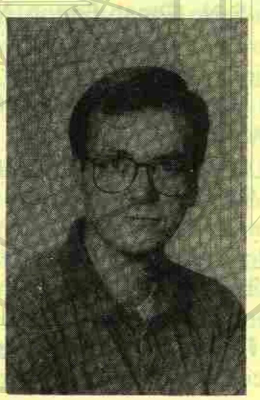


Richard Pleau, School of Architecture,
Laval University,
Quebec, Canada, G1K 7P4

Table 1 - Volume of permeable pores obtained from ASTM C 642 water absorp-tion test method.

		Type I cement			Type III cement		
	W/B	0.45	0.35	0.25	0.45	0.35	0.25
without silica fume	7 days	20.73	14.49	12.30	16.13	11.61	7.81
	28 days	18.14	12.43	12.25	16.53	9.84	8.01
with silica fume	7 days	16.19	13.35	10.07	14.99	12.63	8.09
	28 days	14.93	12.03	8.78	14.79	9.83	8.28

Note: Each number represents the mean value obtained on two concrete specimens

Table 2 - Charge passing through concrete specimens as obtained from AASHTO T 227 rapid chloride ion permeability test method (Coulombs).

		Type I cement			Type III cement		
	W/B	0.45	0.35	0.25	0.45	0.35	0.25
without silica fume	7 days	11 326	6 225	3 283	3 579	2 077	662
	28 days	5 045	2 737	1 809	424	276	164
with silica fume	7 days	8 447	4 903	3 060	3 610	3 503	741
	28 days	3 415	2 163	811	325	281	56

Note: Each number represents the mean value obtained on two concrete specimens

Table 3 - Composition and properties of concrete mixtures used for the comparative study on the influence of coarse aggregates on compressive strength of HPC.

W/B	0.22	0.25	0.30
Water (kg/m^3)	109 – 112	127 – 129	126 – 129
Cement (kg/m^3)	550 – 570	540 – 550	445 – 455
Fine aggregate (kg/m^3)	690 – 720	710 – 730	840 – 860
Coarse aggregate (kg/m^3)	1100 – 1130	1050 – 1070	990 – 1010
Superplasticizer (kg/m^3)	14.0 – 18.1	10.0 – 10.2	7.0 – 7.1
Slump (mm)	170 – 220	205 – 230	170 – 195
Unit Weight (kg/m^3)	2475 – 2541	2437 – 2493	2405 – 2472
Air content (%)	2.2 – 3.8	2.7 – 3.3	3.4 – 4.0

Table 4 - Compressive strength of similar concretes made with different coarse aggregates (MPa).

		Coarse aggregate					
		A	B	C	D	E	F
Age	W/B	limestone	granitic gneiss	dolomitic limestone	granite	peridotite	andesite
48 hrs	0.22	66.6	53.8	73.7	61.2	60.2	62.6
	0.25	62.1	50.6	73.5	--	--	--
	0.30	48.4	44.0	56.0	--	--	--
28 days	0.22	92.5	99.4	111.6	91.3	105.2	105.5
	0.25	88.5	93.6	106.2	--	--	--
	0.30	83.8	87.1	93.7	--	--	--

Note: Each number represents the mean value obtained on two concrete specimens.

Table 5 - Properties of concrete mixtures having exactly the same composition but made with different materials using different mixing techniques.

Coarse aggregate	Super-plasticizer admixture	Mixing sequence*	Dosage of super-plasticizer (L/m^3)	Slump (mm)	Air content (%)	28 days compressive strength (MPa)
dolomitic limestone	M melamine	A	6.5	90	2.1	59
		A	6.5	80	1.5	64
		B	6.5	50	2.4	64
		B	25.5	215	1.1	58
	N1 naphtalene	A	6.5	220	2.5	55
		A	6.5	220	2.1	59
		B	6.5	195	3.0	59
		B	13.6	205	2.1	62
	N2 naphtalene	A	6.5	220	1.8	44
		A	6.5	220	2.6	53
		B	6.5	155	3.3	59
		B	11.5	210	2.5	54
granitic gneiss	M melamine	A	6.5	165	2.3	61
		A	6.5	185	1.8	63
		B	6.5	60	1.5	63
		B	12.3	225	1.5	68
	N1 naphtalene	A	6.5	220	2.5	43
		A	6.5	215	2.8	43
		B	6.5	160	2.8	60
		B	11.0	220	2.5	61
	N2 naphtalene	A	6.5	235	2.4	41
		A	6.5	225	3.0	58
		B	6.5	165	2.8	60
		B	11.0	220	3.0	54

* Mixing sequence :

A : Cement, sand, water and superplasticizer were first mixed to obtain a homogeneous paste and the coarse aggregate were further added.

B: Cement, sand and coarse aggregates were first mixed to obtain a homogeneous mixture and the water and superplasticizer were further added.

Table 6 - Critical spacing factors obtained for different concrete mixtures subjected to 300 freezing and thawing cycles in air or in water (μm).

	Freezing and thawing cycles in water			Freezing and thawing cycles in air
W/B	**Type I cement**	**Type I cement with silica fume#**	**Type III cement with silica fume#**	**Type I cement with silica fume#**
0.50	500	250	—	400
0.50*	500	200	—	400
0.30*	400	300	> 800	450
0.25*	750	—	> 800	—

*: These mixtures were made using a superplasticizer admixture.
#: Silica fume was used as a partial replacement for Portland cement (10% in weight).

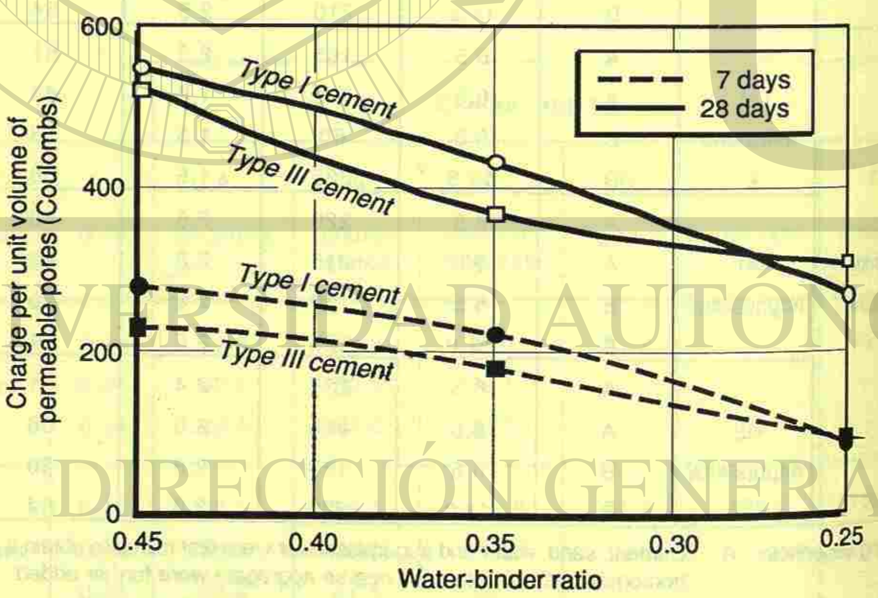
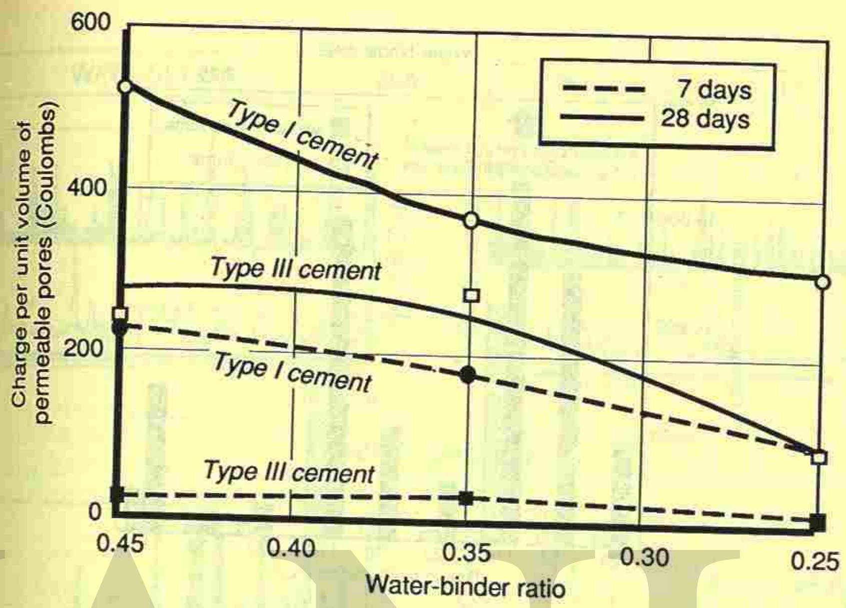
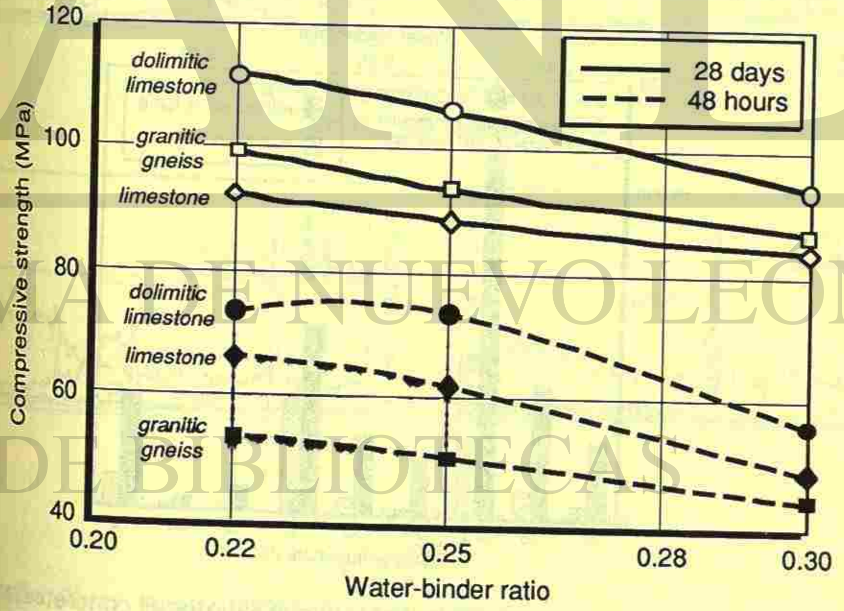


Figure 1 - Relationship between the charge passing through a unit volume of permeable pores and the water binder ratio for concretes not containing silica fume.



ure 2 - Relationship between the charge passing through a unit volume of permeable pores and the water binder ratio for concretes containing silica fume.



ure 3 - Relationship between the compressive strength and the water-binder ratio for similar concretes made with three different coarse aggregates.

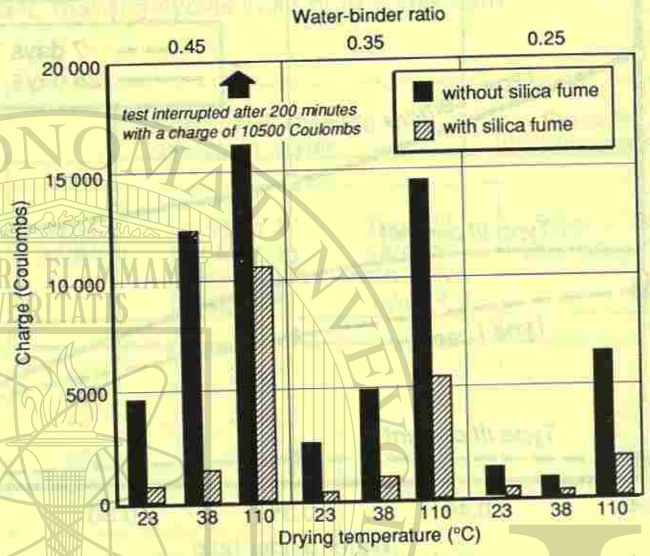


Figure 4 - Chloride ion permeability versus water binder ratio for all concretes made with Type I Portland cement (with and without silica fume) and dried for 90 days (at the temperature indicated) before the test.

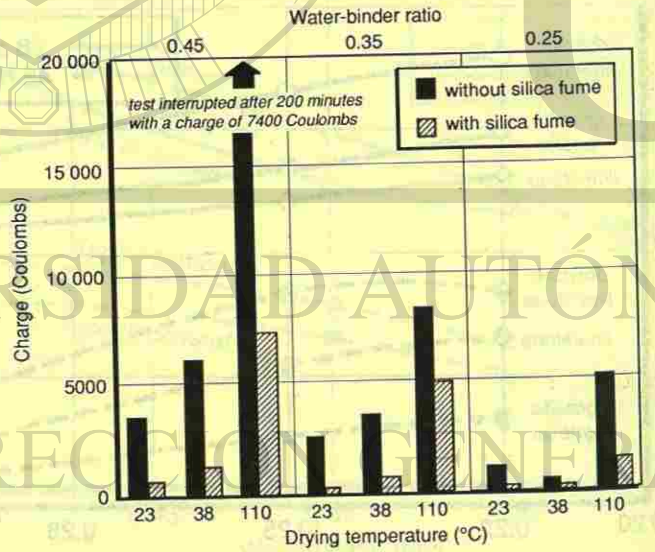


Figure 5 - Chloride ion permeability versus water binder ratiofor all concretes made with Type III Portland cement (with and without silica fume) and dried for 90 days (at the temperature indicated) before the test.

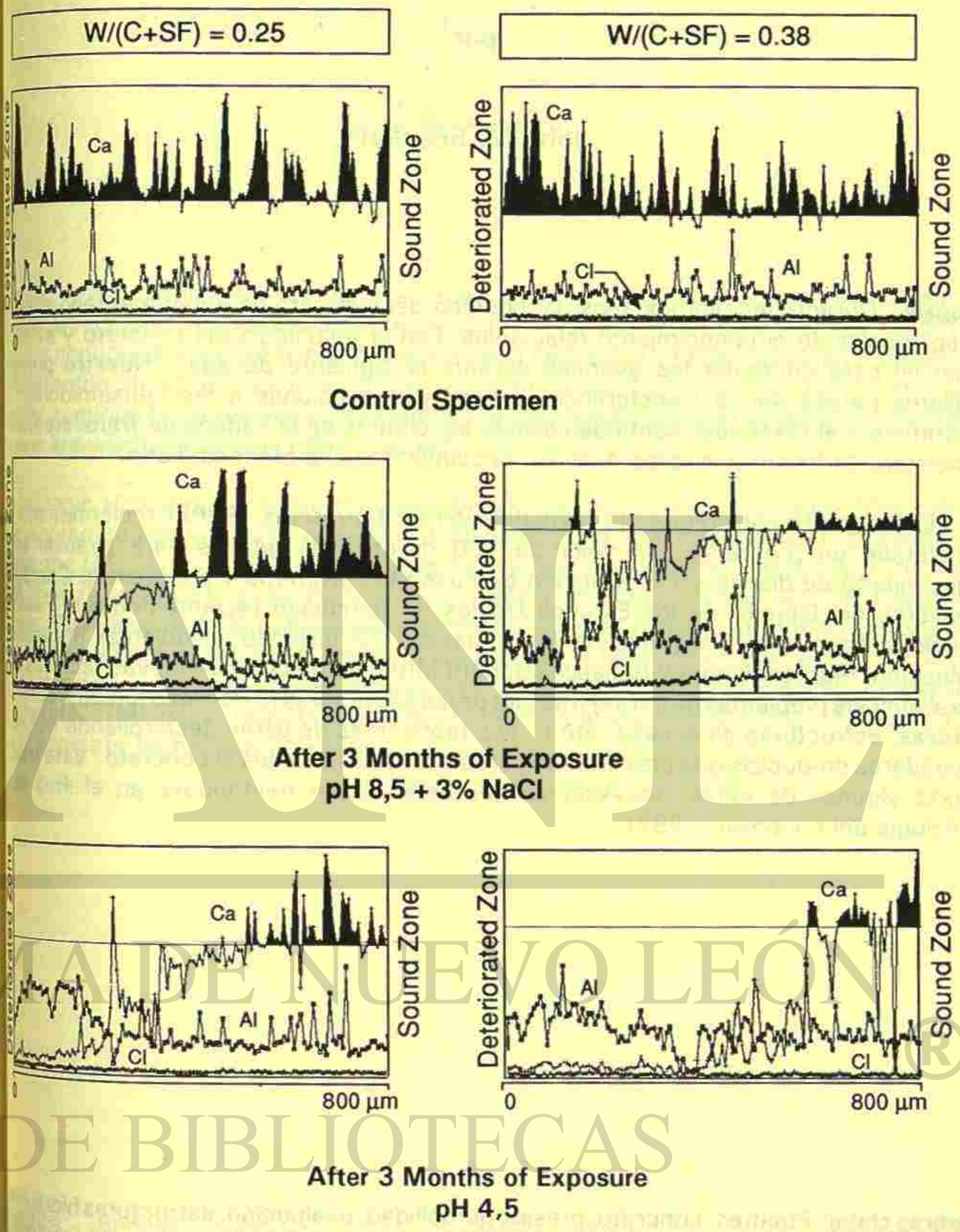


Figure 6 - X-ray energy dispersion measurements for cement pastes with low water-binder ratios (0.25 and 0.38) after three months of exposure to aggressive solutions (a pH of 8.5 with NaCl and a pH of 4.5).

INNOVACIONES EN LA TECNOLOGIA DEL CONCRETO

por

John M. Scanlon

Sinopsis: Durante los últimos años, la industria del concreto ha experimentado avances excepcionales en el conocimiento relacionado con la tecnología del concreto y existe el potencial para continuar los avances durante la siguiente década. Nuestro principal problema parece ser la transferencia de estas innovaciones a los consumidores. El Contratista y el Diseñador continúan siendo los últimos de la cadena de transferencia de tecnología, en incorporar estos nuevos conocimientos a la práctica diaria.

El Programa Estratégico de Investigación de Carreteras (SHRP) recientemente ha completado un programa con valor de 150 millones de dólares para avanzar en el conocimiento de diseño y construcción de nuestras carreteras y puentes; el Cuerpo de Ingenieros del Ejército de los Estados Unidos ha terminado recientemente la Fase I e iniciado la Fase II del Programa de Investigación (72 millones de dólares) Reparación, Evaluación, Mantenimiento y Rehabilitación (REMR) para desarrollar nuevas técnicas para sobrellevar los problemas de infraestructura de las estructuras hidráulicas (presas, cortinas, represas, estructuras de acceso, etc.). Los fabricantes de están desarrollando nuevos e innovadores productos que proporcionan propiedades especiales al concreto. Este trabajo reporta algunas de estas innovaciones que están experimentándose en el área de la tecnología del concreto (1993).

Palabras clave: Puentes, concreto, presas, durabilidad, evaluación, estructuras hidráulicas, innovación, alto comportamiento, pavimentos, desarrollo de productos, investigación, tecnología.

INNOVATIONS IN CONCRETE TECHNOLOGY

by

John M. Scanlon

nopsis: The concrete industry has been experiencing exceptional advances in owledge relating to concrete technology during the last few years and the potential sts for continued advancements during the next decade. Our major problems seems to e transferring these innovations to the users of concrete. The Contractor and the ecifier continue to be the last participant in the technology transfer chain to incorporate s new knowledge into every day practice.

e Strategic Highway Research Program (SHRP) has recently completed a 150 million lar program to advance the knowledge in designing and constructing our highways and dges; the U.S. Army Corps of Engineers has recently completed Phase I and initiated ase II of their Repair, Evaluation, Maintenance and Rehabilitation (REMR) Research ogram (72 million dollars) to develop new technologies in dealing with the infrastructure blems of hydraulic structures (dams, levees, locks, intake structures, etc.). Products nufacturers are developing new and innovative products to provide specialty properties f concrete. This paper reports on some of these innovations being experienced in the as of concrete technology (1993).

words: Bridges, concrete, dams, durability, evaluation, hydraulic structures, ovation, high performance, pavements, product development, research, technology.

John M. Scanlon is a Senior Consultant at Wiss, Janney, Elstner Associates, Inc., Northbrook, Illinois. He is a fellow of ACI and ASTM> Since retiring in 1987 as Chief, Concrete Technology Division, U.S. Army Engineer Waterways Experiment Station he has consulted on engineering and construction problems associated with concrete materials and structures.

INTRODUCTION

In the past, very little definitive research has been appropriately funded in relatively large amounts in the technical area of concrete technology. Concrete has been in such wide use as a construction material that the agencies and organizations involved in concrete design and construction have mistakenly felt that "Concrete was all that if could be" and that "No additional research was needed because concrete can't be improved".

While serving as Chief, Concrete Technology Division of the U.S. Army Engineer Waterways Experiment Station in 1983, the author with others originated a major research program to address the repair, evaluation, maintenance and rehabilitation of Corps of Engineers hydraulic structures. This program became a 12-year 72 million dollar research program with approximately 25 million dollars allocated for research in the area of concrete and concrete structures. If we have to spend such a large amount of funds to research repair of existing concrete it is obvious that "Concrete was not all that could be" and "Additional research on concrete should improve the technology".

The originators of the Strategic Highway Research Program (SHRP) recognized the potential for cost effective research in concrete technology and included appropriate funding in SHRP to take advantage of this potential. Hydraulic cement concrete is an excellent building material; the knowledge of concrete technology has drastically improved during the past 20 years; SHRP has expanded on that technology and as a result of the SHRP Concrete and Structures Research additional knowledge will come available in the near future because of the site specific projects that are being evaluated on a long-term basis.

It is very surprising how many accomplishments have evolved during this relatively short period of SHRP and REMR existence. It has always been the feeling of concrete technologists that concrete improvements based on research evolve over long periods of time and that it seems that "accelerated research doesn't accelerate concrete knowledge". SHRP and REMR have disproved that theory and as a result, we not only have advanced knowledge today, we can expect additional knowledge in the future due to these research programs.

SHRP ACCOMPLISHMENTS

The accomplishments of SHRP (Strategic Highway Research Program) in the area of concrete technology have been in three major overlapping areas, bridge physical condition ssessment, bridge protection and rehabilitation, and concrete materials.

Bridge Condition Assessment

Guidance to evaluate bridge components -- Assess the condition of concrete bridge components using this new "how-to" manual--a toolbox of new and existing survey methods. With reliable information on the level and rate of deterioration in hand, you can readily choose a cost-effective plan of preventive or corrective action to protect bridge components. The manual gives step-by-step guidance on methods to detect deterioration in reinforced and pre-stressed concrete structures. Evaluation steps include initial visual inspections, subsequent routine condition surveys, and detailed condition surveys. Evaluation steps are adjustable for local conditions, materials, bridge types, and agency policies. Thirteen of the tests described in the manual are effective methods currently in use. Seven new tests developed through SHRP research are now available as additional tools to obtain the data needed for a complete bridge condition assessment.

Sampling and testing for chlorides -- Before you protect or rehabilitate a concrete structure that is visibly deteriorating, you should first determine the extent of the chloride contamination. This commercially available chloride sampling and testing method is an inexpensive field procedure. Collect drilled powder samples from various locations and depths in the concrete, and then determine the amount of chloride ion in the concrete by using the specific chloride ion probe. Inspectors save time by immediately determining the amount of chloride present, thus making better sampling decisions. Field studies confirmed the test is accurate.

Corrosion rate -- If you know the history of the corrosion rates for the steel in a reinforced concrete structure, you can predict time to corrosion distress and select an effective rehabilitation strategy. This survey method describes a non-destructive field test for determining corrosion rates and details the use of three commercially available devices that measure polarization resistance for calculating the corrosion rate. All three devices performed well in field tests.

Measuring chloride corrosion -- Your field inspectors will produce more reliable bridge deck assessments using this new corrosion rate detector. This monitoring device detects corrosion using ultralow frequency impedance spectroscopy. Measurements are made by contact probes, and corrosion is quantified using customized and user-friendly computer software.

Measuring concrete permeability -- Three new tests were developed for measuring the permeability of concrete. They are:

- surface - air flow device

- forced air under pressure device
- AC impedance testing

Success of concrete sealers -- Two devices for measuring the effectiveness of concrete sealers were developed. They are:

- the electrical resistance test
- water absorption test

Membrane integrity -- Now you can check the integrity of chloride barrier membranes installed on concrete bridge decks without tearing up the asphalt overlay. A membrane that is perforated of that no longer bonds to the bridge deck will not prevent chloride penetration. Usually, you cannot visually detect this condition. When membranes fail, chloride-induced corrosion is often accelerated because the chloride attack is concentrated in a particular area of the concrete deck. This modified, non-destructive test method uses ultrasonic pulse velocity as an indicator of membrane condition. Using an ultrasonic pulse velocity meter, the transit time of the pulse is measured from the time the signal penetrates the surface of the deck until it is reflected off the membrane. The pulse transit time was statistically correlated to a visual condition membrane rating scale during laboratory and field validation.

Evaluate bridges in depth -- Is the concrete deteriorating under the asphalt overlay on your bridge decks? Ground-penetrating radar (GPR) finds delaminated concrete hiding under the asphalt overlay without your having to stop traffic or pay the high cost of removing the overlay. The GPR unit moves at speeds up to 25 miles (40 kilometers) per hour while mapping subsurface conditions. GPR mapping reduces concrete bridge repair costs by enabling you to focus on critical bridges and to plan cost-effective repairs. The new, more effective GPR unit uses short-pulse radar with a special antenna system and new data acquisition software designed for bridges.

Bridge Protection and Rehabilitation

Removing chlorides -- With this new "how to" guide, engineers can do a better job removing chloride electrochemically from concrete structures. Reinforced concrete bridge elements that are high in chloride (with or without visible deterioration) may be candidates for electrochemical chloride removal. This practical guide also describes the effectiveness of electrochemical chloride removal on a structure deteriorating as a result of alkali-silica reactivity. In cases of extensive chloride-induced corrosion, concrete removal should take place only to the extent of removing loose concrete and replacing it with an appropriate patching material. The remaining sound, but contaminated, concrete is treated electrochemically. Once an electric field is applied, the negatively charged chloride ions move away from the reinforcement steel, toward the surface of the concrete. Equally important, the concrete in effect "re-alkalies" around the bar and "re-pacifies" the steel. Subsequent sealing of the concrete helps to retard future chloride migration back to the reinforcement steel. Chloride removal may sometimes aggravates alkali-silica reactivity. This problem can be controlled by putting lithium ions into the electrolyte. The guide and two companion volumes describe laboratory testing and field trials undertaken to refine this protection technique.

Cathodic protection -- Learn how to stop chloride-induced corrosion in reinforced concrete bridges through use of this proven, cost-effective protection technique. Whether you are inspecting your existing cathodic protection installations or designing new systems, this manual provides you with up-to-date evaluations and recommendations. Using these guidelines on design, installation, and maintenance of cathodic protection systems, you can more effectively control chloride-induced corrosion. This manual guides you through the process of selecting the best cathodic system for the specific bridge component being protected. Useful information is provided on costs, durability of system elements, and monitoring and maintenance requirements.

Nulling currents -- To optimize operation of cathodic protection systems for controlling chloride corrosion, this guide provides reliable information on selecting control criteria. Using laboratory tests and mathematical modeling, corrosion rates were observed for various degrees of cathodic protection. This manual presents time-dependent criteria that optimize cathodic protection. The "corrosion null probe" concept is a technically valid, yet simple approach to meeting the criteria without relying on the long-term stability of embedded reference electrodes. The manual also provides criteria for the lowest electrical current level needed, the effect of intermittent application, and other key characteristics of cathodic protection systems.

Cost effective concrete removal and rebar cleaning -- When faced with cracked and deteriorated chloride-contaminated concrete, you may need to remove the damaged concrete. The extent of removal is based on the level of chloride contamination or corrosion measurements, the location of the damaged area on the bridge, and the selection of a protection system or rehabilitation technique. You can save money by reducing the amount of concrete removed while still protecting the steel from further corrosion. This guide explains how to select the most cost-effective, efficient, and damage-limiting methods from several traditional concrete removal techniques. The guide identifies the advantages and disadvantages of the three most commonly used methods--- pneumatic breakers, milling, and hydro-demolition --in terms of characteristics, productivity, and costs. Removal of surface contaminants and concrete up to a depth of half an inch (13 millimeters) can be accomplished with scrabbling, plaining, sandblasting, or shotblasting. When the concrete removal exposes the reinforcing steel, the bars need to be cleaned of chloride contaminates and loose concrete. Sandblasting, wire brushing, and hydro-demolition are the most feasible methods for cleaning the reinforcing steel. This new guide presents "how-to" information, along with a specification for hydro-demolition.

Rapid bridge deck rehabilitation -- When time is of the essence in repairing bridge decks, this new manual on field-validated, rapid rehabilitation techniques and materials can help you select the right strategy. Conventional and innovative techniques for protection, rehabilitation, and replacement are discussed and compared in the manual. The manual also covers cost, service life, and specific limitations of each method. Rapid protection systems include bituminous overlays on membranes, polymer overlays, and penetrating sealers. Temporary repairs include high early strength portland cement concrete patches, and other hydraulic cement concrete patches. The rapid rehabilitation of a deck includes the use of patching systems followed by either a protection system or a high-performance concrete overlay. Rapid replacement systems include site-cast high performance concrete or pre-cast concrete deck panels.

Repairing marine structures -- Arc-sprayed zinc coatings, originally developed for reinforced concrete marine structures, effectively prevent corrosion of concrete structures in warm, humid, and salt-laden environments. Arc-sprayed coatings perform well and generally cost less to apply and to maintain than conventional impressed-current cathodic protection systems. The coating is installed by first cleaning the steel bar and surrounding concrete surfaces, and then applying arc-sprayed zinc directly on the exposed area and onto the cleaned concrete. This creates a simple, self-powdered, sacrificial anode-protection system. Information is available on how to apply this innovative technique.

Solving corrosion-induced distress -- Using design guidelines and decision criteria, you can easily select the most appropriate solution for protecting or rehabilitating an existing concrete bridge on a life-cycle basis. Once you know the condition of a bridge, you can select a cost-effective solution. A manual or computer program gives you costs, service life, and technical viability of a wide range of techniques. You can use a decision model to predict future corrosion-related distress. The computer program covers bridge decks. The manual covers both decks and substructure elements and includes flow charts, figures tables, nomograph, equations, and other aids.

Concrete Materials

SHRP accomplishments relating to concrete materials technology can be categorized into six different areas:

1. Concrete Performance
2. Alkali-Silica Reactivity
3. Freeze and Thawing Conditions
4. Nondestructive Testing
5. High Performance Concrete
6. Optimizing Highway Concrete Technology

Concrete performance -- Concrete has been such an amazing building material, we sometimes forget that it's performance can be improved. Invariably it performs the way the designer expects it to; maybe the designers should expect it to do more. With improved performance we can expect designers to revise future design criteria in order to take advantage of this improved performance as they become more familiar with these capabilities.

- Improved Concrete Mix Designs (proportions) Handbook.

This new handbook will prove helpful in producing a dense, hardened concrete with high strength and low permeability, which improves durability and service life. The handbook describes a procedure for designing concrete mixes using optimum blends of sand and coarse aggregate to yield the maximum packing density of the concrete mix components. The procedure is based on a mathematical model that predicts the packing density of the concrete mix by taking into the account the size and specific gravity of the various mix components. The model output is expressed in the form of tables and triangular diagrams (isodensity lines), from which the engineer can determine the optimum amount of each mix element. For highway applications, there appears to be a narrow range of mix compositions that will produce the maximum packing density.

- Ensuring Crack-Free Curing of Concrete

Place concrete with confidence under varied temperature conditions. Using this set of tables, engineers can predict thermal effects during the curing of concrete pavements and assess the potential risk of damage to the concrete in a particular environment. This guide identifies a range of possible actions to ensure crack-free curing of concrete. For possible combinations of concrete temperature, air and ground temperatures, and type and amount of cement in the mix, the guide provides tables that show which combinations are safe and which require special care. Using these tables, you can predict the thermal effects for concretes generally used in concrete construction. The thermal effects tables warn you if too high a temperature will be reached during curing of the concrete and whether thermal cracking will result from variation in the temperature of the concrete and the temperature of either the air of the subbase.

- Diagnosing Performance Problems With Concrete

This guide provides a procedure for diagnosing problems with concrete field performance using optical microscopy. Problems with concrete performance and durability can be detected by optical microscopy, which reveals the microstructural features of concrete. Thin sections of concrete are treated with fluorescent dyes, making it easier to identify the microstructural features of the specimen, such as porosity, homogeneity, and air entrainment. The equipment is commercially available.

Alkali-silica reactivity -- Alkali-silica reactivity (ASR) causes irreversible damage to concrete pavements and structures. ASR occurs when silica substances in aggregates react with alkali in the cement and absorb water to the extent that the concrete expands and eventually cracks. It can lead to structural failure. ASR is difficult to distinguish from other causes of cracking in concrete. If concrete cracking problems are incorrectly identified and improperly rehabilitated, the ASR problem may be made worse. Inspectors and engineers need to be able to correctly identify ASR so they can choose the proper rehabilitation strategy and screen aggregates to avoid ASR problems in new construction.

- Handbook for Identification of ASR

Help is now available to assist highway engineers and bridge inspectors to identify alkali-silica reactivity (ASR). This new illustrated handbook of ASR-related damage in concrete pavements and bridge structures helps distinguish ASR-induced cracking from damage by other causes, such as improper curing, corrosion of reinforcing steel, or freezing and thawing. Concrete cracking is often caused by more than one factor. The color photographs in this handbook provide help in visually inspecting for ASR.

Once alkali-silica reactivity (ASR) has been visually detected in concrete, its presence must be confirmed. A new, simple, and fast chemical test for ASR requires no tedious sample preparation. Instead, the concrete surface is treated with a uranyl acetate solution and viewed under ultraviolet light. The uranyl acetate causes the ASR "gel" to fluoresce, imparting a greenish-yellow glow that can be clearly seen under the ultraviolet light. (The technique is described in the ASR handbook). The equipment and chemicals needed for this test are commercially available.

- Mitigating ASR in Existing Concrete

Current options for rehabilitating ASR-damaged concrete are limited. Two chemical treatments have recently been documented, and a new approach to mitigating ASR damage in existing concrete has been evaluated. High molecular weight methyl acrylate and silanes appear to be helpful in inhibiting further ASR damage in pavements and bridges. The chemicals penetrate and fill cracks and bond to the cracked concrete faces, preventing moisture from entering ASR-affected areas. Laboratory tests indicate that treating ASR-affected concrete with lithium hydroxide may also be an effective way to prevent further ASR-related damage.

- Tools for Screening Reactive Aggregates

The obvious way to avoid ASR-related damage in concrete is to refrain from using potentially reactive aggregates. Current standard test methods for screening aggregates - the Mortar Bar Test (ASTM C-227) and the Quick Chemical Test (ASTM C-289) - often underestimate the ASR potential, particularly for slow-reacting aggregates. A new laboratory test quickly and reliably screens potentially reactive aggregates. Mortar bars or concrete prisms are immersed in a sodium hydroxide solution maintained at a temperature of 80 degrees Celsius for up to fourteen days. If the bars or prisms expand by more than 0.1 percent of their original length, the aggregate is potentially reactive and should be avoided. The test may also be modified to determine the maximum acceptable alkali content in cement for use with a potentially reactive aggregate. The test has been submitted to AASHTO for consideration.

- Designing ASR-Safe Concrete Mixtures

Now you can design a concrete mix that will be safe from alkali-silica reactivity (ASR). Use this new specification guide to produce concrete that will resist ASR and the associated expansion and damage. The guide recommends specific combinations of cement, coarse and fine aggregates, and admixtures (both mineral and chemical) to resist ASR in new concrete. Adding pozzolanic materials such as fly ash and silica fume and chemical admixtures such a lithium hydroxide to the concrete mix will produce an ASR-safe concrete. Lithium salts and certain types of fly ashes have been found very effective in inhibiting ASR-related expansion in concrete.

<u>Freeze and thaw conditions</u> -- Freezing and thawing of concrete under saturated conditions has always been a major deteriorating condition for concrete. The test methods for determining the ability on specific concrete to resist freezing and thawing have not really represented existing field conditions.

- Concrete Resistance to Freezing and Thawing - Modified Test

A new procedure offers a third option for evaluating the resistance of concrete to freezing and thawing. Current practice is to use the standard AASHTO T-161 test (ASTM C666), which allows two alternative procedures to be used. Procedure A involves freezing and thawing concrete specimens in water, and Procedure B consists of freezing the specimen in air and thawing it in water. The SHRP alternative involves wrapping the test specimen in moist towels during both freezing and thawing phases, thus realistically simulating the natural freeze-thaw environment of concrete. The procedure is under consideration by AASHTO.

- Avoiding D-Cracking - An Aggregate Durability Test

Avoid cracks caused by freezing and thawing at the joints in concrete pavement (D-cracking) by using this new test to screen aggregates. The test method currently used to screen aggregates for D-cracking potential (AASHTO T-161 or ASTM C-666) takes too long (8-10 weeks). This new, simple laboratory test takes only 8-10 days. Pre-dried aggregates are submerged in water in a pressure chamber and subjected to a pressure of 1150 psi (about 8 MPa). Under pressure, water enters the aggregate pores and compresses the air within the pores. When the pressure is rapidly released, the air compressed in the aggregate pores forces the water out; if the aggregate fracture or fragments, it indicates a susceptibility to D-cracking. A total of 50 test cycles (10 per day) are recommended for each sample set. The fracture percentage indicates the D-cracking potential of the aggregate. Aggregates that pass this test can be readily approved for construction. Aggregates that fail should be subjected to the more rigorous. AASHTO T-161 test. The test method has been submitted to AASHTO for consideration.

- Avoiding Aggregates Susceptible to D-Cracking

Cracking at joints in concrete can be avoided by using aggregates that are not susceptible to being damaged by freezing and thawing. New concrete aggregate specifications based on the hydraulic fracture test (SHRP 2002) can identify such aggregates. Aggregates that are known to be durable based on highway performance records generally show less than 5 percent of fracture after 50 test cycles. The specifications can be used as the basis for developing criteria for selecting aggregates that resist the effect of freezing and thawing. The specifications will be submitted to AASHTO for consideration.

- Mitigating D-Cracking in Existing Concrete

The protection strategies described in this report can help avoid or minimize further damage of in-service concrete evidencing D-cracking problems. The permeability of the concrete is reduced by preventing water from entering and accumulating in concrete joints. Water-saturated aggregate increases the potential for the concrete to fracture during freeze-thaw cycles. Various protective options are reviewed, including penetrating sealers, low-permeability concrete bonding at joints, asphalt overlays, and other alternatives that may help to extend the service life of existing concrete.

- Improving Condition Evaluation - New Soundness Test for Concrete

Evaluating the condition of in-service concrete is now easier with this new laboratory test. Using impact-frequency techniques on the concrete specimen, the test detects changes in the modulus of elasticity of concrete due to weathering or other deteriorating influences. In addition to being rapid and reproducible, the test produces information on the damping characteristics of vibrations induced in the specimen by impact. As the condition of in-service concrete deteriorates through freeze-thaw damage, it causes a decrease of vibrational amplitude and an increase in the damping. The degree of damping indicates the extent of damage to the concrete caused by factors such as ASR, freezing, and thawing. The test can also be applied to predict durability factors of concrete specimens after subjecting them to only a few freeze-thaw cycles using the AASHTO T161 test. This test has been submitted to AASHTO for consideration.

- New Air Entrainment Specifications

This guide provides new specifications for using air entrainment to improve the resistance of concrete to freezing and thawing. Air entrainment is an established method of protecting concrete from frost damage. The increasing use of chemical admixtures in concrete, such as water reducers, superplasticizers and pozzolanic materials, including fly ash and silica fume, makes the revised and improved specifications necessary. The new specifications apply to concrete currently used in highway construction and cover volume, size, distribution, and spacing factors for air in concrete to make the concrete resistant to frost action.

Nondestructive tests for quality control of concrete -- Too often concrete pavements, piers and walls are removed because of a lack of knowledge about the concrete properties. When we have to wait for 28 days to verify compliance with a strength requirement, the structure may already be in service. Coring existing concrete to verify concrete properties invariably results in swiss cheese structure before everyone is satisfied or has enough information to make decision. NDT can prevent these problems.

- Detecting Concrete Flaws and Delaminations

Defects such as voids, cracks, or delamination in concrete pavements and structures can be located using a new, nondestructive test method that can be applied in the field. This technique, known as impact-echo (or pulse-echo), measures transient stress were signals within the concrete generated as a result of an external impact. The test detects differences in characteristics of reflected wave signals that depend upon the condition of the interior of the concrete. The commercially available test equipment costs approximately.

- Measuring Concrete Strength and Maturity in the Field

This new guide details method for estimating the early-age strength of newly constructed concrete structures in the field. The estimate will help you determine when to remove the construction forms, saving construction time an costs. The methods described in the guide are based on "pull out" and maturity tests. The pull-out test measures the force required to pull a specially shaped steel insert out of hardened concrete, which correlates with its compressive strength. The maturity test measures the temperature of the concrete caused by hydration of cement, which correlates with its strength development. Both these techniques are simple and easy to use and give results with acceptable accuracy.

- Measuring Air Entrainment with the Fiber Optic Air Meter

The right amount of air entrainment is very important to concrete quality in any construction work. This new meter meets the need for a rapid and reliable field method for monitoring the air content of concrete during construction. The fiber optic air meter measures the air content of fresh or plastic concrete at the construction site. The device uses a fiber optic sensor probe, which is inserted into the concrete. The intensity of light reflected from air bubbles correlates with the amount of air in concrete. With proper calibration, the light intensity measured by the probe determines the air content.

Mechanical behavior of high-performance concrete -- High-strength concrete was primarily developed for high-rise buildings in order to compete with structural steel, but now high-performance concrete can be developed to be used in structures such as pavements, bridges, training walls and even guard walls on parkways.

- Specifications for High-Performance Concrete

High-performance concrete, with augmented strength, enhanced durability, and extended service life, is increasingly used for a variety of special applications. This guide documents specifications for the production and use of high-performance concrete for highway applications. These special concrete blends are generally characterized by a low-water/cement ratio and may contain fly ash, silica fume, fibers, and chemical admixtures such a water reducers, superplasticizers, and accelerators. Guidelines and recommendations are provided for material selection, proportioning, mixing, placing, and curing three categories, of high-performance concrete (very early strength, high early strength, and very high strength). The guide focuses on specific highway applications, paying special attention to economic feasibility and the use of locally available materials. In addition, an extensive reference database is included in the guide on the mechanical properties and behavior of these concrete mixtures.

- Determining the Modulus of Rupture

This proposed modification to the current standard test method (ASTM C-78) for the modulus of rupture, or flexural strength, of portland cement concrete uses a reinforcing bar at the center of gravity of the test specimen and a device to measure deformation. The use of the reinforcing bar prevents the sudden failure of the specimen and any damage to the deformation measurement device. In addition to flexural strength, the test provides information on flexural strain capacity of concrete.

- Determining Interfacial Bond Strength of Concrete

Now available is a new laboratory test method for determining the interfacial bond strength of two concrete sections. For highway construction projects such as concrete overlays, it is important to determine the quality of bond between the old and new concrete. A good, strong bond will generally indicate a durable pavement or bridge deck with enhanced service life. The direct shear test uses a specimen with an interfacial bond area of old and new concrete. The test measures the strength of the interfacial bond and the relative slip between bonded faces.

- Measuring Compressive Strength of HP Concrete

High-performance concretes are generally stronger than conventional concretes. Testing high-performance concretes for compressive strength using sulfur capping sometimes results in the cap failing before the specimen. By modifying the current standard test method (AASHTO T-22 and ASTM C-39) to use steel caps with neoprene pads instead of sulfur caps, the problem is avoided. The AASHTO T-22 test currently includes the use of steel caps as a supplement to this method. This alternative has been further refined and the modification evaluated.

<u>Optimum highway concrete technology</u> -- The water content is the most critical ingredient of concrete; too little and the concrete may not be workable or capable of being consolidated; too much and the properties having to do with quality may be lost.

- Determining Water Content of Fresh Concrete

The amount of water in the concrete mix affects the quality of concrete during placement and the durability of the hardened concrete. Excessive amounts of water can cause the concrete to be more prone to damage by freezing and thawing and alkali-silica reactivity. Also, too much water in the mix will make the concrete more permeable, allowing salt to penetrate, which will promote corrosion of reinforcing steel. A simple and rapid field test method has been developed for determining the water content of fresh concrete based on microwave drying. This test consists of drying a representative sample of fresh concrete in a microwave oven. The water content is calculated by the weight loss of the sample after drying. The test takes only 15 minutes.

- Testing for Consolidation of Concrete

Proper consolidation of concrete increases its density and results in a number of benefits, such as reduction in undesirable air voids, decreased permeability, decreased drying shrinkage, and improved bonding to reinforcing steel. This field test method determines the degree of consolidation, or in-place density of plastic concrete, using a twin probe nuclear density gauge. The measurement of in-place density or thickness of concrete and other construction materials has been made much easier by the development of nuclear gauge technology. The technology measures the decrease in radiation intensity (gamma rays or photons) between the nuclear source and a detector situated across the material. A decrease in radiation intensity correlates with the thickness and density of the material. The twin-probe gauge has an advantage over other nuclear gauges in that it can estimate the density of plastic concrete at various depth and across a larger area.

- HWYCON - An Expert System of Concrete Durability

This computer program serves as a useful advisory tool to assist highway engineers in making decisions related to the diagnosis, material selection, and repair or rehabilitation of concrete pavements and structures. It is a user-friendly, menu-driven program whose knowledge base contains current information on concrete durability from various sources, such as published literature, SHRP research, and other concrete research. Photographic images stored in the computer provide visual information about various types of concrete distress and test methods and procedures. The system uses explanatory tests to assist the user in interpreting the facts, photographs, and procedures. Recommendations for repair and rehabilitation strategies are based largely on the extent of concrete deterioration and the type of distress. HWYCON is designed to run a 386-based desktop computer.

REMR ACCOMPLISHMENTS

The U.S. Army Corps of Engineers Repair, Evaluation, Maintenance and Rehabilitation (REMR) Research Program was initiated in 1984 with a planned six-year expenditure of 36 million dollars of which ten (10) million dollars would be used to research concrete materials and structures infrastructure. The REMR research program also included other areas of infrastructure research:

	• Geotechnical	$ 8 million
	• Hydraulics Structures	4.5 million
	• Coastal Structures	4.7 million
	• Electrical & Mechanical	2.5 million
	• Environmental Impact	1.5 million
and	• Operations Management	2.2 million

The first phase of this research program was completed in 1991, and was so successful that the Congress approved an additional Phase II with similar funding for six additional years starting in 1992.

Significant accomplishment during Phase I developed so many cost effective materials and techniques for repairing existing concrete hydraulic structures that the savings attributed to the first six months of research exceeded the total cost of the six-year research program. The major accomplishments developed in the Concrete and Structures Research Area are listed below:

- Development of a precast concrete stay-in-pace forming system for lock wall rehabilitation
- Development of procedures and devices for underwater cleaning of hydraulic structures
- Extension of the technology of nondestructive testing systems for in-situ evaluation of concrete structures
- Evaluation of the effectiveness of a) pressure injection, and b) polymer impregnation repair techniques
- Development of mathematical models for analyzing concrete cracking in lock wall resurfacing
- Development of concrete mixtures for use in underwater repairs
- Development and evaluation of underwater repair techniques using abrasion-resistant concrete
- Evaluation of vinylester, polyester, and epoxy resins, and cement grouts for embedding anchors in hardened concrete
- Development of procedures for repairing wet concrete surfaces
- Development of a user's guide: Maintenance and Repair Materials Data Base for Concrete and Steel Structures
- Development of new stability and safety evaluation procedures for gravity earth retaining structures founded on rock
- Identified in-situ repair techniques to be used in rehabilitating cracked concrete hydraulic structures
- Extended the technology of silica fume concrete

- Extended the state-of-the-art in using anti-washout admixtures for underwater concreting
- Developed procedures for producing service life in cases of deterioration due to freezing and thawing.

ADDITIONAL INNOVATIONS

SHRP and REMR were two of the most successful research programs that have been ertaken during the past 20 years but there have been many innovative ideas and ucts developed by the concrete industry. A few of these innovations are:

- Extended Retardation Chemical Admixtures
- Corrosion Inhibiting Admixtures
- Freeze Protection Chemical Admixtures
- Alkali-Silica Resistant Admixtures
- Anti-Washout Admixtures
- Specialty Pre-packaged Repair Materials

Yes, the technology of concrete is being vastly improved and we have just begun, et us dream about the future; we will probably see many innovations within the next ade.

- Greater utilization of mineral admixtures
- Silica fume properties materials being marketed made out of rice-hull ash
- Greater dependency on prepackaged materials providing specific properties for particular projects
- Tighter controls on the chemical properties of cements so that they may be used with greater efficiency with chemical and mineral admixtures.
- Increased ASTM C494 requirements of existing commodity chemical admixtures.
- Independent QA auditing of materials producers with greater reliance on fresh concrete evaluation techniques for final acceptance.

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John M. Scnlon,
Senior Consultant at Wiss, Janney, Elstner Associates, Inc. Norhbrook, Ill. U.S.A.

CONCRETO EN AMBIENTE MARINO

por

Theodore W. Bremner

Sinopsis: Se revisa la historia de las estructuras marinas de concreto comenzando con los puertos romanos hasta las plataformas mar adentro de hoy en día y se evalúa su comportamiento en términos del grado en que hayan cumplido las expectativas del diseñador. El medio ambiente marino puede tener un efecto destructivo sobre el concreto y se discuten las diversas reacciones físicas, mecánicas y químicas en términos de los posibles mecanismos de deterioro. Se presentan las normas actuales para hacer al concreto resistente a las acciones destructivas y se discute su efectividad. Se dan ejemplos de estructuras que fallaron al no comportarse adecuadamente, se proporcionan las posibles razones para su pobre comportamiento y se presentan procedimientos para la reparación. Se describen los proyectos actuales que involucran los avances en la tecnología marina del concreto. Se enumeran las investigaciones que se están desarrollando en todo el mundo y se enfatiza la necesidad de coordinar internacionalmente un programa de investigación sobre el concreto marino.

Palabras clave: Concreto, marino, agua de mar, químico, deterioro, investigación, refuerzo, corrosión, álcali-agregado, abrasión, impacto, puzolana, ligero, expansivo, vesicular, barcos, puertos, muelle, desembarcadero.

CONCRETE IN MARINE ENVIRONMENT

by

Theodore W. Bremner

nopsis: The history of concrete marine structures is reviewed starting from the Roman rbors up to the offshore platforms of the present time and their performance is evaluated terms of how well they met the designers expectations. The marine environment can ve a destructive effect on concrete and the various physical, mechanical and chemical actions are discussed in terms of possible deterioration mechanisms. Current code quirements to make concrete resistant to these destructive actions are presented and eir effectiveness discussed. Examples of structures that have failed to perform properly e given and possible reasons for their poor performance are given and repair procedures e presented. Current projects that involve advances in concrete marine technology are scribed. Research being done on marine concrete around the world is listed and the ed for an internationally coordinated research program for marine concrete is stressed.

eywords: concrete, marine, seawater, chemical, deterioration, research, reinforcing, rrosion, alkali-aggregate, abrasion, impact, pozzolan, lightweight, expanded, vesicular, ips, harbor, piers, wharf.

Theodore W. Bremner, Professor of Civil Engineering, University of New Brunswick, Fredericton, N.B., Canada, E3B 5A3. Dr. Bremner teaches undergraduate and graduate courses in materials and conducts research on lightweight concrete and on the performance of concrete in severe environments. His students are working in the areas of moisture mechanics, permeability of lightweight and normal weight concrete, effectiveness of epoxy-coated reinforcing steel in salt water and seawater, durability of concrete in seawater, chemical admixtures for concrete and resistance of concrete to freezing and thawing. Before returning to university, he was a structural design engineer. He acts as a consultant on the design, maintenance and repair of marine facilities.

INTRODUCTION

Historians focus on disasters, wars and national calamities and spend scant attention on the successes of the working public especially if they are builders and especially so if the facility is a success. In ancient times ports were of great importance as most of the trade and communication was by sea and one of the best preserved is the Port of Cosa in Italy built about 200 B.C. The main feature of the harbor was the concrete piers, three of which are still visible above the water. The marine concrete served its function well until changing trade patterns two centuries later made it obsolete (1).

The piers at Cosa built by the Romans are believed to be the first known examples of harbors built using pozzolans. It seems they recognized early in the second century B.C. that pozzolans, a powdery volcanic ash, when mixed with lime and water, sets and endures when placed in seawater or fresh. Pozzolans, hydrated lime and water were used by the Cosa concrete engineers to form the matrix or binder that was combined with the particulate phase composed of limestone, tuff and amphora fragments to produce concrete. Over the years these massive blocks wasted away to some extent but nevertheless the structure is still useable at the end of the second millennium for its intended purpose if the need exists. Unfortunately some modern structures being attacked by seawater in a similar way have been known to require repairs before the facility was commissioned and others need continual maintenance to keep them in a serviceable condition. These processes of deterioration that affect both ancient and modern concretes will be examined and means of minimizing their effect will be discussed.

A further complication arose when steel was incorporated into concrete for marine work. Significant increase in structural efficiency was possible when the tensile forces could be assigned to this embedded steel but these more highly stressed structures developed a new form of deterioration - spalling of concrete cover over embedded steel. The material specialist in some aspects has yet to fully cope with this problem which has rendered seemingly elegant structures into ones that challenge our ability to maintain them in a serviceable condition.

In recent years new materials and procedures have been introduced that offer significant improvement both to the economy and to the performance of structures and those procedures and materials appropriate for marine work will be presented.

NATURE OF MARINE EXPOSURE

Effect of Geographic Location

The degree to which concrete remains durable depends on a number of factors including mix constituents, mix design, climatic conditions, exposure duration and workmanship. Of prime importance is geographic location as this influences whether or not the concrete will be subjected to cycles of freezing and thawing. Prior to the use of an air entraining agent, most of the concrete marine structures deteriorated rapidly from this effect. With a properly entrained air void system the concrete is immune to this type of attack and other forms of attack become of prime importance.

Temperature affects the reaction process in that increasing temperature results in increasing rates of chemical reaction between the seawater and the concrete; however some of the reaction products - gypsum in particular - are more soluble in cold water than in hot water. Also the cold seawater absorbs more oxygen and so can cause greater rates of corrosion of steel in submerged concrete. In general the maximum air temperatures in July are surprisingly similar at the major ports around the world. The wide variation in minimum January temperature at these various ports reflects the fact that deterioration from chemical interaction is very slow in the winter at some locations which means that exposure intensity is different but probably not substantially so from one marine location to another for the concrete exposed to the atmosphere.

What can be different is the temperature of the surface water which can range from 28°C at the tropics to -1°C in the polar regions (2). Useful design information about the effect of these temperature extremes on concrete can be obtained from work done by the U.S. Corps of Engineers at their two exposure sites for concrete on the east coast of North America (3). Their cold water site is at Treat Island, Maine near the Canadian-U.S. border where the temperature ranges from a low of 1°C in the winter to a high of 40°C in the summer. The warm water site is at St. Augustine, Florida, where the surface seawater temperature is 21°C year round. Chemical attack would tend to dominate in Florida while deterioration due to freezing and thawing would be the main factor at Treat Island for concrete specimens located at a midtide wharf. Summer air temperatures at Treat Island can approach extremes experienced at St. Augustine and concrete prisms at midtide level of size 250 mm square by 750 mm long reach ambient air temperature at the end of the dry period of the tidal cycle and therefore a chemical interaction between the seawater and the concrete also should be expected in northern waters as the concrete remains fully saturated during the period when the tide is low.

Nevertheless the potential for chemical attack is substantially less in cold regions than in tropical regions. Mean tidal range at Treat Island and St. Augustine are 5.5 and 1.4 m respectively. The wide variation in temperature between water and air plus the more rapid change in water level would make the thermal shock effect more severe in Treat Island as compared to St. Augustine. However the tendency for disruptive expansion to occur would be lessened at Treat Island by the increased solubility in cold water of some of the reaction products.

As shown previously, ambient air and water temperature plus local tidal ranges affect the concrete's interaction with seawater. Of lesser importance is the local variation in the chemical composition of the soup that constitutes seawater for a particular ocean body (4). However, the salinity and content of various salts in different seas vary as can be seen in Table 1 (5, 6). The salinity in seas with a small inflow of fresh water and high evaporation rates such as the Mediterranean and Dead Seas is higher than that of the oceans (5). Seas whose intake of fresh river water or snow and ice melt water is high, can have greatly reduced salinities. Local evaporation of sea spray plus possible contamination of the seawater with fish processing wastes can result in severe local concentrations of salts. In northern regions deicing agents such as sodium and calcium chloride can be used to deice concrete surfaces which will have disastrous results for marine structures already critically saturated with salt from spray.

In terms of aggressive attack of concrete by seawater it would appear that all concrete materials engineers concerned with the design and repair of marine facilities will be building in a soup that chemically tends to be more similar than different and that the maximum summer temperature also will tend to be similar. This allows some simplification in formulating international design codes in that design is normally based on the worst possible case conditions regardless of the geographic location of the structure. The only exception is that design codes recognize that in higher latitudes where freezing and thawing is likely to be a problem, air entrained concrete must be used.

Zones of Reaction

To illustrate the various zones of attack, reference will be made to one of the most common concrete marine structures which is cylindrical piles (columns) used to support a beam and slab deck. Such a structure can be seen in Figure 1. The numbers refer to areas and components most likely to deteriorate with the lowest number assigned to the area where deterioration is likely to become sufficiently severe to require attention first. The higher numbers reflect areas that are less likely to deteriorate and that may survive indefinitely without requiring repair. The zones and various deterioration mechanisms listed below are based on observations and testing of various types of structures in Canada and observations of a total of 11 international ports.

Description

Zone

A. **Submerged Zone.** Fully below the low tide level and continuously underwater and which usually is not affected significantly by the seawater.

B. **Lower Tidal Zone.** Between midtide level and low tide level with significant chemical attack possible at midtide level and with essentially no deterioration at low tide level.

C. **Upper Tidal Zone.** Between midtide and high tide level with significant deterioration in cold regions of concrete with an inadequate air void system. Chemical interaction between seawater and concrete has the most severe effect in this region. Also corrosion of steel reinforcement is evident at midtide level and becomes more serious above midtide level being especially severe at high tide level.

D. **Splash Zone.** From high tide level to a height above high tide level equal to the annual maximum wave height as measured from trough to crest. Deterioration due to freezing and thawing is usually maximum in this region. Minor signs of chemical attack from seawater are evident.

E. **Atmospheric Zone.** From the Splash Zone upward and being of greater effect where onshore winds are present. The effect is reduced going inland but still can have a significant effect in terms of chloride content of concrete at 1 km from shore.

The numbered components of the structure in Figure 1 are listed in Table 2 in escending order of those most likely to deteriorate and for each component the types of eterioration are listed in order of decreasing effect or decreasing probability of occurrence.

Within the five zones there can be microzones where deterioration patterns are ignificantly different. Generally on a wharf deck the top concrete surface tends to eteriorate to a significantly greater depth than does the underside which tends to be oist and where the diffusion of carbon dioxide is significantly lessened. The opposite ccurs in bridge decks over seawater. Generally the underside of the bridge deck is much ner than the deck and the depth of carbonation on the bottom is usually twice that of the op. In general the second and third bays in from the front of the wharf will show greater orrosion damage than does the outer bay. Nevertheless sufficient chloride can be present om the seawater spray to cause the cover over the bottom layer of steel to fall off evealing a network of corroded steel reinforcement. Repairs to this type of structure are xtremely difficult. Beams under the wharf that support the deck slabs are frequently the st to show deterioration. Salt water running down the face of the beams, evaporating nd leaving the salt behind can account for exceptionally high levels of salt in very calized regions.

Marine concrete can be subjected to the same type of deterioration in all regions but he effects appear more pronounced in some zones. Alkali-aggregate reaction is usually vident first in wharf faces or parapet walls above high tide level. It usually is evident next n the top of the wharf deck. Based on experience in Atlantic Canada alkali-aggregate eaction has a much greater effect on highway bridges inland than it does on marine tructures when the same materials were used in both instances. Apparently the salt ater carries away the reactive compounds lessening the effect.

Abrasion of the concrete due to lateral transport of sand and gravel past the structure an have an effect especially at the splash zone and to a lesser extent below the water vel. Damage due to hard berthing can also be a problem and damage of a purely mechanical nature is frequently evident and may be a greater indicator of poor seamanship than of poor concrete design and construction.

Typical Deterioration Patterns

In cold northern waters, concrete structures of the type shown in Figure 1 have been known to exhibit scaling of the concrete deck with a veneer of concrete coming off the concrete surface in the first year. Also in the first year loads of concrete that failed to have an air-entraining admixture added to the mixture become evident and can result in complete disintegration of the concrete in several years. Current practice normally requires that every load of concrete be tested for entrained air and that the existence of an acceptable air void system be verified by testing the hardened concrete using candidate materials before construction starts. Nevertheless, many structures made without entrained air continue to serve a useful purpose in cold northern waters. Normally the part of the structure saturated and exposed to freezing and thawing will deteriorate on the surface first and then progress inward. The deteriorated form has the characteristic hour glass shape. It starts at midtide level and becomes more severe reaching a maximum at high tide level and decreasing at an increasing rate in the splash zone. Repairs can be surprisingly effective if all concrete showing freeze-thaw damage and areas likely to deteriorate in the foreseeable future are removed and replaced with air entrained concrete. Inevitably deterioration will eventually occur at the old to new concrete interface and this must be accepted as part of the ongoing maintenance process. In certain instances, particularly in small sections of concrete that can easily respond to temperature fluctuations, failure can be more dramatic with the concrete failing in mass rather than by progressive loss of a surface layer, and then repair and reinstatement on a local level will probably not be successful unless the as-yet-undamaged concrete can be encased with a structurally and thermally protective layer of significant thickness.

When freezing and thawing is not a problem the deterioration is a maximum at midtide and is symmetric above and below the midtide level proceeding at decreasing rates above and below midtide level and being negligible at high tide and low tide levels (7). In most instances where freezing and thawing are not a problem surface deterioration due to chemical attack is slow with trivial loss of surface mortar taking several decades. Polymer coatings can be highly effective in restoring the appearance of the structure and preventing future chemical attack (8).

Concrete below low tide level is surprisingly resistant to seawater. Some surface softening will occur in low strength concrete, however, this is not likely to be significant in concrete of a strength normally used for marine structures. Mehta and Hayes tested 67 year old concrete blocks (1 x 1.7 x 1.7 m) that initially had been partially submerged and then fully submerged at the San Pedro Breakwater in Los Angeles Harbour. Only the 20 MPa concrete showed any surface softening (9).

DETERIORATION MECHANISMS

Deterioration Mechanism for Plain Concrete

As was discussed previously, lack of resistance to freezing and thawing causes remature failure usually before the normal chemical interaction of concrete with seawater as taken place. Only in concrete exposed in regions where freezing and thawing is not problem or where an effective air void system is present can the chemical interaction be valuated. In concrete not at risk to freezing and thawing damage, the interaction with eawater involves various surface effects that are easily visible and result in a progressive ughening of the surface. Also elements in the seawater diffuse into the concrete and eact with the hydration products. This according to Moskvin causes an initial densification and then an expansion that after many years of seemingly good performance produces severely spalled surface (6).

Anderson examined concrete from the Bay of Fundy and from the English Channel sing a scanning electron microscope equipped with energy dispersive x-ray equipment 10). He found that the chemical interaction zone was no more than 25 mm thick in good uality concrete even after five and six decades of exposure at midtide level. Also he und that specimens having severe surface deterioration still retained strength comparable what would be expected if they had been continuously moist cured in fresh water.

The chemical reactions between marine concrete and seawater have been summarized y Regourd (11) who lists such reaction products forming as $MgSO_4$, $MgC1_2$ and $CaC1_2$ as eing the dominant reactants coming from the seawater. The tricalcium aluminate (C_3A) s the main reactant with the seawater to cause expansion of the concrete. Also she notes at CO_2 cannot only react with the $Ca(OH)_2$ to produce a precipitate that acts as a coating milar to carbonation but also reacts with the ettringite ($C_3a.3\ CaSO_4.32\ H_2O$) to produce aumsite ($CaCO_3.CaSO_4.CaSiO_3.15H_2O$). During the course of current research this latter oduct has been found in entrained air bubbles in large quantities which in turn renders r entrained concrete no longer air entrained and therefore vulnerable to damage due to ezing and thawing.

Surface expansion due to the growth of chemical compounds resulting from the action between the seawater and the cement paste occurs in the outer 25 mm layer of e concrete and must be considered a positive factor assuming that freezing and thawing sistance in not being degraded. Unfortunately concretes with otherwise good rformance tend to shed a veneer of concrete about 25 mm thick after about 50 years posure to seawater. Also workmen engaged in repair note a very dense surface layer at tends to be very strong but is only of limited thickness. During demolition this thin neer tends to come off in slabs. This surface deterioration may be due to many causes d is deserving of further research.

A review of the test data from long term testing at the Technical University of Norway ne over a twenty five to thirty year period and reported by Gjorv (12) revealed a nificant increase in flexural strength with time up to 15 years at which time there was a regression of strength for concrete with high C_3A contents. Apparently these chemical surface effects on the 40 x 40 x 300 mm and 100 x 100 x 750 mm had what appears to be an initial positive effect on the tensile strength until a system of fine cracks formed in the interior of the concrete that adversely affected the tensile strength (12).

The deterioration mechanisms noted above are not likely to have a significant effect on concrete of a water to cement ratio prescribed by most codes as the reactions are only of a surface nature and are dependent on the tricalcium aluminate content of the cement. By limiting this component the reaction can be controlled.

Deterioration Mechanism for Concrete Containing Embedded Steel

Marine structures of all types that are of steel or have steel components are at risk from the rapid oxidation of the iron in the chloride laden seawater. Concrete is a protective material in which to encase steel in that an initial reaction takes place which rapidly goes to completion and forms a thin protective oxide layer over the steel. This protective whitish layer ($Fe(OH)_2$) can be readily seen by the naked eye. This protective layer can break down for one of two reasons, and the $Fe(OH)_2$ turns to $Fe(OH)_3$ and $Fe(OH)_3.3H_2O$ with a substantial increase in volume that results in the concrete over the steel being spalled off exposing the steel to direct corrosion from the seawater with a resulting loss of structural capacity.

The protective oxide layer can be maintained by keeping the pH of the concrete above about 11.5 which is not difficult if the concrete is wet as in the splash zone or below as seawater is normally deficient in CO_2 and it is the CO_2 gas which reacts with the $Ca(OH)_2$ to produce $CaCO_3$ which then lowers the pH and results in the protective layer being destroyed. Above the splash zone and particularly on the underside of marine decks where the structure is periodically subjected to salt laden moist air during storms followed by extended periods of exposure to dry air, the concrete will dry out, air will diffuse in and the concrete will carbonate rendering the pH less than 11.5.

The protective oxide layer can also be destroyed even when the pH is above 11.5 when chloride ions diffuse into the concrete from seawater, salt spray, and salt laden winds. Salt levels of 0.6 to 0.9 kg of Cl per cubic metre of concrete are normally the threshold level to cause corrosion to start. A combination of chloride levels above the threshold level plus carbonation provide especially severe conditions for concrete and the periodic nature of high wind speeds and high waves plus subsequent air drying accounts for most of the failures of concrete structures to perform their intended purpose. This applies only where distress cannot be attributed to poor workmanship, inadequately air entrained concrete in cold regions, inadequate structural design, and where concrete mix materials and mix proportions failed to meet current standards.

Corrosion of steel can be controlled by using one or more of the following approaches:

i improving the quality of the concrete;
ii using protective coatings at the concrete surface;
iii using a corrosion inhibiting admixture such as calcium nitrate;
iv using a coating at the steel/concrete interface as a barrier;
v suppressing the electro-chemical process of embedded steel by implementing cathodic protection.

The above five approaches are listed in order of decreasing ease of implementation and in increasing probable degrees of effectiveness given available technology.

Quality of concrete in this context means low permeability to chlorides and oxygen and in order to achieve this, researchers have dealt with such parameters as composition of cement, mix proportions, water to cement ratio and the thickness of the concrete cover. Unfortunately concrete capillaries vary from 15 to 1000 A° in diameter and chloride ions are less than 2A° in diameter and easily pass through concrete. Also concrete subjected to alternating conditions of wetting and drying in a marine environment provide conditions such that the chloride and oxygen ions will eventually permeate through relatively high quality concrete. Therefore enhancing the concrete properties to prevent corrosion of the embedded steel is not always economical or possible.

One of the most effective methods of improving the performance of concrete is to prestress it to prevent cracking. Unfortunately there have been problems with this technology in recent years where prestressing strands pass through ducts in the concrete. Apparently these ducts have been improperly grouted in many instances and the cables have corroded. Currently the UK Department of Transport is preparing a new standard for this work and until it is available they will not commission any new bridges using grout-duct post tensioning (13). The use of protective coatings has proved to be surprisingly effective based on coatings applied to unreinforced concrete prisms at Treat Island by the U.S. Corps of Engineers. A variety of polymeric coatings were applied and have remained intact for about a decade. Whether or not the coating is sufficiently crack free to hinder chloride diffusion is not known nor is it known if the coatings would be as effective at high tide level where corrosion is a major problem. An epoxy coating has been used on the King Faud causeway from Saudi Arabia to Kuwait in the splash zone for the main piers and has given good service. On test piles that were not coated, salt scaling has eroded the surface to a depth of 3 to 5 mm (14).

The third method listed; use of a corrosion inhibiting admixture, shows considerable promise provided that the concrete can be made sufficiently impermeable that the admixture, calcium nitrate, does not bleed out. Long term experience is lacking, but short term results are promising.

Coatings applied at the concrete/steel interface have been used for several decades, however recent incidents of failure have been reported. In 1986 the first reported signs of corrosion of epoxy-coated bars were observed in the Long Key Bridge Structure in

Florida which was built in 1979-1981. Further investigation revealed problems with other bridges built about the same time. Although some of the spalling of the cover over the reinforcement was in areas where the cover was inadequate the majority of the corrosion was observed in locations with up to 10 cm of sound concrete cover. In December 1988, the Florida Department of Transportation stopped specifying the use of epoxy-coated rebars in bridge substructures as they felt it would not provide suitable long term protection against corrosion in a marine splash zone environment (15). Nevertheless bars coated with fusion bonded epoxy and cast in concrete prisms and tested both in the field at Treat Island and in accelerated tests have demonstrated their ability to give significant improvement in performance over that of uncoated bars (16).

Suppression of the electro-chemical process by implementing cathodic protection has been demonstrated to be effective in parking garages and bridge decks but long term data on its successful use in marine structures is limited and its effectiveness to deal with the various exposure zones has yet to be established. Gerwick reported on a seawater cooling canal in eastern Saudi Arabia where sacrificial anode cathodic protection has been effective below and 1 m above water level (14).

In the question of corrosion of embedded steel in a concrete marine structure it is apparent that at best any one system, if employed properly, will give a useful service life. However, marine structures tend to have long term uses and a combination of two of the above listed protection methods would appear mandatory and that only the first method, improving the quality of concrete, enjoys full support of the engineering profession in terms of certainty of results and method of implementation.

DESIGN CODES

Marine Structures are covered by general codes such as those formulated by technical associations like the American Concrete Institute and the Federation Internationale de la Precontrainte. Also there are various national codes written specifically for marine work. In recent years it has been the practice to write standards for specific applications such as for offshore (17), floating (18) and Arctic (19) structures.

The purpose of these codes is to assure that concrete does not fail based on the available information, and as information becomes available these design codes are revised. Generally they reflect the state-of-the-art at the time they are written and differences between various documents can be explained by noting the date of publication. A case in point is the allowable level of tricalcium aluminate (C_3A) in Portland cement for use in marine concrete. Before 1980 codes normally limited C_3A to 5% or less as it is the primary reactant in the cement to cause expansion when in the presence of seawater (11). The C_3A was later found to react favorably with the chlorides in the seawater producing an insoluble chloro-aluminate which in turn lowered the permeability of the concrete and also reduced the potential of the steel reinforcement to corrode (20). The 1989 Canadian Standard for Offshore Structures requires a tricalcium aluminate of "not less than 4 nor more than 10% by mass of the total cementing material used" (17). Gerwick, in giving an overview of his international experience, stated that "there is a growing indication that 10 to 12 percent may be optimum" (14). Formulation of standards and guides to good practice must be a continuing process if lessons learned from current laboratory research and field observations are to be incorporated into these standards in a timely way. Means must be found to have standards that are used internationally reflect the broad spectrum of local environmental conditions rather than reflect the conditions near the head office of the so-called international institutions. With globalization of trade this process will become less difficult with time as the major international financial groups have a vested interest that extends world wide.

PERFORMANCE RECORD OF MARINE STRUCTURES

Marine structures are usually designed for a specific application for which they are deemed cost effective if they can be maintained operational for their financial design life with acceptable maintenance costs. Using this as a measure of their performance record concrete marine structures have been extremely successful with only a few structures requiring extensive maintenance to keep them serviceable. These problem structures usually involve either improperly understood technology such as the hollow-core piles for the Rodney Terminal in Saint John, New Brunswick, Canada (21) or the use of unsuitable materials such as seawater as mix water for heavily reinforced marine concrete as used in the middle east (22). In the former, the best minds of the day failed to anticipate the problems and in the latter the basic principles, known for decades, were ignored.

Marine structures used by the Allied and Axis forces during the last world war were almost without exception extremely durable and continue to act as an example of how concrete should perform. With the exception of the submarine pens in Trondheim, Norway, the structures have not found an alternative use and their good durability is in fact a disadvantage. Not so in most other concrete marine structures. Owners come and go but the marine commercial facilities endure continuing to meet the needs of successive owners. Usually bought for distressed prices they are modified to meet the new owner's requirements and frequently make business ventures viable that would not be so otherwise. A case in point is the non air entrained concrete wharf at Eastport, Maine, USA which was built in the 1920's by Continental Can, sold to various fish processing firms and then bought by Mearle Corporation for their fish scale business. Finally it was sold to a Canadian firm with the unlikely name of "Maine Pride Incorporated" who currently put it to good purpose in their salmon aquaculture business. Only limited areas of this non air entrained concrete wharf needed repairs at several times in its life but these were easily done at modest cost. Unfortunately one small part of the facility failed to receive timely repairs and collapsed; however, the remaining part has been revamped to meet current needs and serves its intended purpose admirably.

Most government and municipal marine wharves continue to be serviceable in spite of benign neglect. In Canada all wharves made prior to 1950 were non air entrained and most of them, in various states of distress continue to serve their intended purpose. The main reason for this is their massive nature in an environment where deterioration is a surface phenomena. The exception is when the alkalis in the cement react expansively with the aggregates and this can, at least in Atlantic Canada, cause serious problems with the structures after about three decades of service. Although no testing of these structures appears to have been done below low tide the problem might be less here as cracking although severe in the splash zone becomes less apparent at low tide.

The offshore structures located between Norway and Scotland used to extract oil from the sea bed have been in place for over two decades and have performed exceptionally well (23). Equally well performing but less well known are the concrete platforms in the Gulf of Mexico. Some of these have been in place for over four decades. Many of them are made from lightweight concrete and demonstrate the good performance likely to be obtained with this material. Mather (24) reviewed the literature on the first two marine structures (now over 75 years old) made from lightweight concrete - the "Atlantis" and the "Selma" and noted the good performance of these ships. Recently Hoff (25) reported on a comprehensive series of tests on lightweight concretes that are relevant to marine structures.

Many examples of good long term performance exist for marine concrete. The exceptions are few and can be attributed to many reasons including the fact that unique designs were attempted for which all factors were not taken into account. Even in these rare cases the structure can, in most instances, be made serviceable as was Rodney Terminal and the non air entrained wharf at Eastport.

CONCLUSION

Marine concrete structures have proven to be both so versatile and durable that they can serve in the most severe climatic conditions in the world. Instances of poor performance are so few that the public normally expects extremely long term service. This means that the structure may be asked to serve several uses during its long service life and this presents a special challenge to designers to produce a structure that will have minimum maintenance costs. Our current concern with how best to build durable marine structures focuses attention on those aspects most needing investigation, and the problems with the corrosion of steel embedded in concrete is probably the one most in need of action. Because corrosion is influenced by the presence of flexural cracks it would appear desirable to do testing of stressed beams in seawater in both cold and hot climatic conditions. A research program to address this problem is now being implemented at Treat Island and in Florida and also possibly at a coastal site in Mexico. Because these tests must run for several decades, considerable care is needed to make sure that the test frames and instrumentation will have the necessary long term stability. Work to validate the testing equipment and procedures is now essentially complete and full scale beam tests are scheduled to start in the Fall of 1994.

The performance of structures is influenced, according to Skalny, "... by our education system, quality of research and development, knowledge transfer, quality of workmanship,

onomic factors such as life cycle costing and research funding ..." (26). To cover this the level Skalny suggests, warrants a study on an international bases that covers the spectrum of human expectations with respect to marine concrete and discussions ng these lines perhaps should start at this conference.

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Theodore W. Bremner,
Professor of Civil Engineering,
University of New Brunswick,
Fredericton, N.B., Canada

Table 1. Concentration of Major Ions In Seawater Concentration (mg/ℓ)

Ion	Form	Atlantic	North Sea	Persian Gulf	Baltic Sea	Caspian Sea
Chloride	$C1^-$	20 000	16 850	23 000	3 960	4 173
Sodium	Na^+	11 100	12 200	13 100	2 190	2 482
Sulphur as Sulphate	$SO_4^=$	2 810	2 220	4 000	580	2 349
Magnesium	Mg^{2+}	1 410	1 110	1 480	260	570
Calcium	Ca^{2+}	480	430	500	50	270
Potassium	K^+	400	550	670	70	66

Table 2. Components Most Likely to Deteriorate and Component Deterioration

Number	Component (in decreasing order of most likely to deteriorate)	Cause for individual component deterioration (in decreasing effect or decreasing probability of occurrence)
1	Upper Tidal Zone Support	Freezing and thawing, chemical reaction between seawater ad concrete causing dissolution or disruptive expansion
2	Wharf Face	Mechanical damage, alkali-aggregate reaction, corrosion
3	Deck Slab (Top)	Plastic shrinkage cracking, freezing and thawing, alkali-aggregate reaction, corrosion
4	Deck Beams	Freezing and thawing, corrosion
5	Lower Tidal Zone Supports	Chemical reaction between seawater and concrete causing dissolution and disruptive expansion
6	Deck Slab - Soffit	Freezing and thawing, corrosion
7	Back Wall	Corrosion, freezing and thawing
8	Buildings On or Near the Wharf	Corrosion
9	Structures Away from the Sea	Corrosion
10	Submerged Zone Supports	In exceptional circumstances - surface softening due to destructive interaction between seawater and cement paste matrix. Usually found where concrete has been placed improperly underwater and the cement past fraction has segregated from the fine and coarse aggregates.

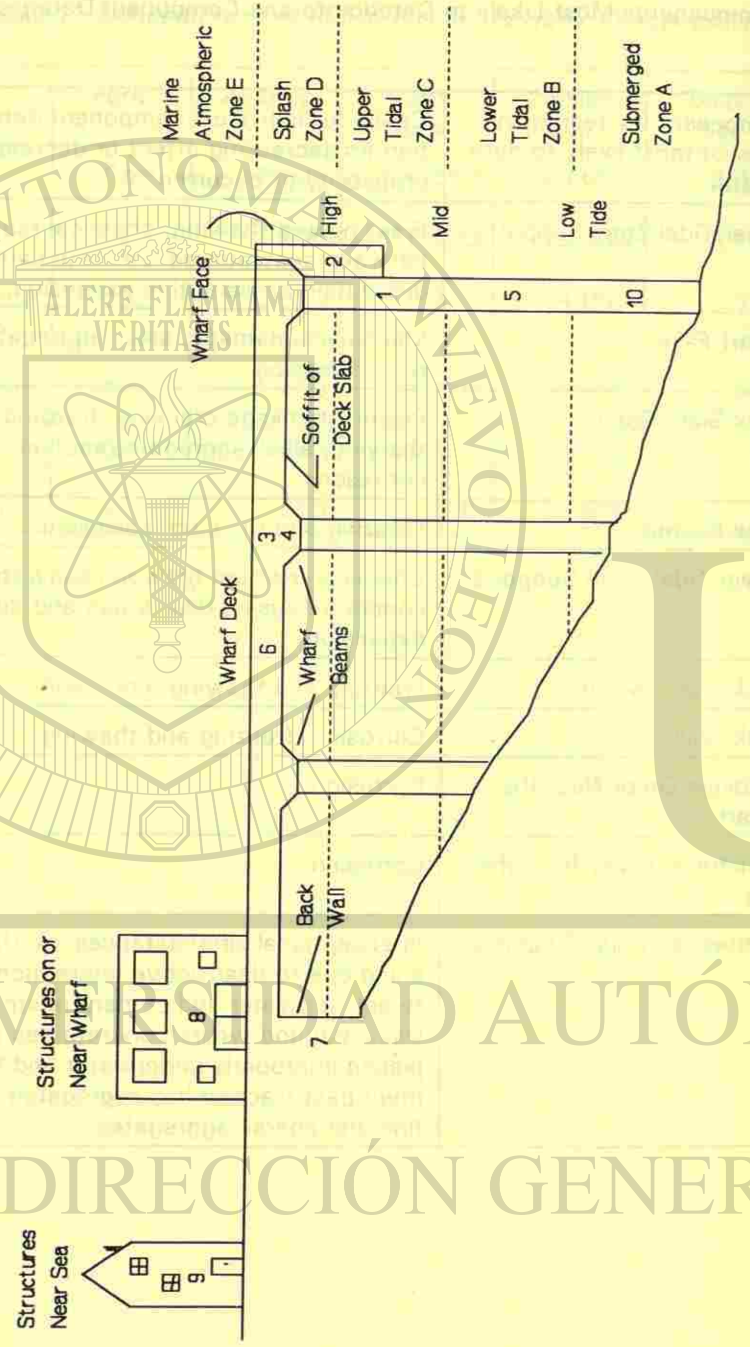

Figure 1. Concrete Structural Elements of a Harbor Structure

CONCRETO EN ESTRUCTURAS MAR ADENTRO

Por

George C. Hoff

Sinopsis: Este trabajo da a conocer los antecedentes relacionados con el uso de concreto en estructuras mar adentro para la explotación de hidrocarburos. Se describen estructuras cimentadas en el fondo, flotantes y otras más especializadas. Se discute el uso de materiales para concreto y la mano de obra de la zona para producir concreto con resistencia moderada para estas plataformas. Se describen las diversas propiedades de tanto el concreto fresco como endurecido que son esenciales en el diseño y construcción de estructuras mar adentro. Se toman en cuenta los diseños especiales y las normas de seguridad. No se requiere de prácticas de construcción especiales. Las estructuras pueden construirse en diques secos, en deslizadores o barcazas sumergibles. Partes de las estructuras pueden ser de concreto prefabricado. Todas esas estructuras involucran alguna maniobra marina. Requieren de muy poco mantenimiento aun en ambiente marino severo y, por consiguiente, los costos de su ciclo de vida tienden a ser bajos.

Palabras Clave: Construcción en barcaza, materiales de construcción, propiedades del concreto, calidad del concreto, prácticas de construcción, diseño, construcción en dique seco, estructuras de concreto flotantes, maniobras marinas, estructuras de concreto en mar adentro, concreto prefabricado, seguridad, construcción en deslizadores.

CONCRETE FOR OFFSHORE STRUCTURES

By

George C. Hoff

ynopsis: The paper provides background information related to the use of concrete in ffshore structures for the development of hydrocarbon resources. Bottom-founded tructures, floating structures, and other more specialized structures are described. The se of local concrete materials and labor to produce moderate strength concrete for these latforms is discussed. The various properties of both the unhardened and hardened oncrete that are essential in the design and construction of offshore structures are escribed. Special design and safety considerations are noted. No special construction ractices are required. The structures can be built in dry docks, on skidways, or on ubmersible barges. Portions of the structures can be precast concrete. All of these tructures involve some marine operations. The structures require very little maintenance ven in the severe marine environment and thus tend to have low life-cycle costs.

eywords: Barge construction, concrete materials, concrete properties, concrete quality, onstruction practices, design, dry dock construction, floating concrete structures, marine perations, offshore concrete structures, precast concrete, safety, skidway construction.

George C. Hoff received his Doctorate in Civil Engineering from Texas A&M University. He has worked in the field of concrete and construction materials for 30 years and joined the Engineering Department of Mobil Research and Development Corporation in 1982. He is a member of the American Concrete Institute Committee 357, Offshore and Marine Concrete Structures, and the Federation de 'la Precontraint Commission on Sea Structures.

INTRODUCTION

Offshore concrete structures are generally understood to be those structures exposed to an open-sea environment (1,2). They are designed to remain permanently or semi-permanently fixed to the sea bed by gravity, piles, or anchors, or to remain afloat and moored. They are often associated with the exploration and production of hydrocarbons but may have many other specialized uses.

Like most other types of concretes, concretes for use in offshore structures are usually made with local materials by local labor in conformance to local guidelines or specifications. Thus, they can vary widely in quality. Depending on their particular application, their strengths can vary from 25 to 65 MPa (3600 to 9500 psi). They are all required to be extremely durable. Once a concrete structure is placed in the sea, maintenance becomes very difficult due to the hostile environment and is very expensive. Some offshore concrete platforms have design lives of 50 to 70 years.

The use of concrete in marine structures goes back to the ancient Romans and Greeks. The use of concrete as a hull construction material for commercial vessels began at the end of the 19th century (3). Initial applications were generally world-wide and consisted of concrete barges and pontoons. The first reinforced concrete sea-going ship was the "Namsenfjord", constructed in Norway in 1917. The first concrete platform for oil and gas production in the Gulf of Mexico was installed in 1950. Since that time, more than 1000 related concrete structures have been built in that area (4) with the first concrete gravity base structure in U.S. waters being installed in 1978 (5). The first large offshore concrete platform for the North Sea (Ekofisk Tank) was installed in 1973. Three concrete platforms, functionally similar to those of the North Sea, have been built in Brazil for South American offshore waters (6). Concrete has a long history and a significant and successful presence in offshore and marine applications.

Throughout the paper, reference will be made to things that are "onshore", "inshore", and "offshore". Onshore is on the land. Inshore means that the location is away from the land but is close enough to the shore to be in protected waters with respect to the open sea. Offshore means that it is located in the open sea. The term "owner" is also frequently used. The owner of an offshore structure can be a single company, or it can be a collection of companies who retain varying percentages of the operation but who have designated a single company to operate and maintain the facility.

TYPES OF CONCRETE STRUCTURES

Offshore structures used in conjunction with hydro-carbon exploration and production can generally be grouped as either being bottom-founded or floating. Many of the bottom-founded structures are also required to float at various stages of their life. The following descriptions of the various types of platforms are very brief but are intended to give the reader a feeling for the enormous versatility that can be realized when concrete is used.

Bottom-founded Structures

Bottom-founded structures can be further identified as:

1. Gravity Base Structures.
2. Concrete Cylinder Pile Supported Structures.
3. Floatable/Bottom-founded Concrete Hull Structures.

Examples of each are shown in Fig. 1, 2 and 3.

The gravity base structure (Fig. 1), commonly called a GBS, maintains its position on the sea bottom due to its very large weight. The sliding force and over-turning moment due to the maximum environmental loads are resisted by the weight of the concrete, the operating weights on the structure, and any additional ballast weight that is contained within the structure. This type of structure is common where produced oil must be temporarily stored before being removed to a tanker or pipeline. The practical range of water depths for these platforms is 40 to 350 m (130 to 1150 ft.). These structures are built at onshore or inshore locations and floated out to their final location. They can also be re-floated when platform removal is required (7). More detailed descriptions of these types of platforms can be found in (1), (2) and (8).

Concrete cylinder piled structures (Fig. 2) were the earliest type of concrete offshore platform used. The first Gulf of Mexico platform of this type was installed in 1950. More than 1000 of these platforms have been installed in Lake Maricaibo in Venezuela (4). They consist of an array of prestressed concrete piles which are driven into the seabed. The piles are arranged so that a prefabricated template deck can be placed over the array to form the working surface of the platform. The decks can be made of concrete or any other suitable construction material. Concrete jackets are often placed around the piles in the splash zone and boat impact region of the platform. Steel cross bracing between piles may also be used to stiffen the overall arrangement when the piles become fairly long. The practical range of water depths for these platforms is from 5 to 20 m (16 to 65 ft.). The use of concrete cylinder piles is also common for support of docks, wharves, bridges and roadways over water.

The floatable/bottom-founded concrete hull platforms generally consist of a barge-like concrete hull which is designed to float. Extending upwards from the hull are posts or columns which act as the support frame for the platform (Fig. 3 and 4). These posts or columns can be made of concrete or steel. The hull is floated to its desired location and then water- ballasted down until it sits on the seabed. It is then "pinned" to the seabed by spud piles around its perimeter. These piles maintain the platforms position and help resist sliding and overturning as the platform does not have sufficient on-bottom weight by itself. Once the hull is piled into position, the topsides deck and equipment are usually added using a crane barge. This type of platform has many variations. It can accommodate some subsea storage of produced oil in the hull. The practical range of water depths for these platforms is from 4 to 30 m (13 to 98 ft.). Platforms of this type that are in use in the Gulf of Mexico have, on numerous occasions, been refloated and reused at different locations. Table 1 is a listing of this type of structure constructed by one firm for the Gulf of Mexico and shows typical concrete hull dimensions.

Floating Structures

Floating structures are those structures which will perform their operational function while in a floating mode. These structures will require a permanent mooring system. In general, the current family of floating concrete structures includes:

1. Concrete Tension Leg Platform (TLP).
2. Deep Draft Concrete Floaters (DDCF).
3. Concrete Production/Storage Barges.

Examples of each are shown in Figs. 5, 6 and 7. Large concrete buoy-type floating structures have also been conceptualized.

Concrete Tension Leg Platforms (TLPs) (Fig. 5) derive their name from the fact that they are fastened to large anchors on the seabed by long tethers which have a predetermined amount of tension in them. These tethers, which originate at the corners of the platform, keep the floating platform in a very precise position. The platform itself can have various configurations but generally resembles the semi-submersible drilling rigs which are common throughout the offshore petroleum industry. It consists of an arrangement of base pontoons, shafts or columns which extend upward from the pontoons, and a deck which sits on top of the shafts or columns. The entire hull (pontoons and shafts) and the deck can be made in concrete. The practical range of waterdepth for use of this type of platform is from 300 to 1500 m (1000 to 5000 ft.). The size of the TLP is generally dictated by the amount of operational weight to be carried. Current designs have ranged as high as 50,000 tonnes (55,000 tons).

The Deep Draft Concrete Floater (DDCF) (Fig. 6) is similar in principal to the TLP but uses a conventional mooring system rather than tension tethers. It maintains its positioning during operations due to its extremely deep draft (greater than 130 m (425 ft.)), large weight, low center of gravity, and mooring from the lower portions of the hull. These factors tend to make the structure relatively insensitive to the motions of the sea.

ke the TLP, its configuration can have many variations, but in general, it resembles a TLP ut with a very deep hull. Similarly, the pontoons, columns, deck, and any bracing can e made in concrete. The practical range of waterdepth for use of a DDCF is from 300 900 m (1000 to 3000 ft.). Like the TLP, the size of the DDCF is generally dictated by e amount of operational weight to be carried. Current designs have ranged as high as 0,000 tonnes (55,000 tons).

A variation of the DDCF is the Spar Buoy Platform (Fig. 8). It also takes advantage f the low center of gravity and heavy weight of the concrete to be relatively insensitive the motion of the sea. It can accommodate crude storage, if desired. It also needs a ooring system (9).

Concrete Production Barges (Fig. 7) are custom-built prestressed concrete barges that ovide a support surface for the process equipment, work and storage areas, and living arters needed for offshore oil and gas production. Drilling is usually not done from these rges but is done from special drilling vessels or jack-up rigs. The production wells are ually located on a nearby unmanned fixed platform. The entire barge or selected rtions of the barge can be built in concrete. A mooring system must be provided for the rge. Storage of the produced oil and other partially processed fluids can be accommoated in the barge. Large floating concrete oil storage facilities have been built in Japan. he size of the barges is influenced by the sea states in which it must operate and the mount of working area it must provide. The water depth in which a barge can operate s a function of the draft of the barge and the operational sea states. A notable oduction/storage concrete barge is the Ardjuna Sakti liquified natural gas (LNG) barge rrently on station in the Java Sea (10). More detailed information on concrete barge-like uctures and concrete hulls can be found in (1), (3), and (11).

her Structures

Concrete subsea oil storage tanks (12,13,14) (Fig. 9) have been proposed for use in aterdepths ranging from 20 to over 400 m (65 to over 1300 ft.). These tanks can be ilt like the base of a GBS but are fully submerged to the seabed where they function as gravity base structure.

Concrete wall caissons (Fig. 10) have been used to provide the retaining wall for earth ed islands. These islands provide the working surface for the oil and gas exploration nd/or production. The caissons are built as floating units, towed to location, joined into unit, and then ballasted to the sea floor. The framework of the caissons then forms the rimeter of an island. A hydraulic fill is usually used to fill the interior. When the use of e caisson retained island is complete, the caissons can be refloated, disassembled, and ture allowed to reclaim the island. A notable application was the Tarsuit Caisson etained Island (15) where the caisson was made of lightweight aggregate concrete. ncrete caissons for an artificial island are a strong contender for development of the ytch Farm prospect in offshore southern United Kingdom (16). The use of caissons for ificial islands is generally limited to waterdepths of less than 15 m (50 ft.)

Concrete has been used for the base of flare towers and offshore loading buoys. An entire flare boom tower made of concrete will be used for the Sliepner platform in Norway. Concrete anchors (17) for the Snore TLP have been built. The Maureen offshore development uses a concrete offloading buoy. Concrete subsea wellhead protectors for Oseberg II in the North Sea have been built (18).

The potential for concrete use is great and is limited only by the ingenuity of the concrete designer and constructor.

CONCRETE QUALITY

There is a perception that all concrete used in offshore platforms is something unique and special and requires a technology that is beyond "normal practice" for concrete construction. If "normal practice" means the practice applied to residential construction, the perception is correct. If "normal practice" means the practice applied to any major civil engineering structure such as a building or bridge, then the perception is wrong. There is nothing unique or special in the application of proper batching, delivery, consolidation, and curing of properly proportioned concrete mixtures. In general, the recommended practices for concrete construction, including materials selection and mixture proportioning, that exist in the various building codes, specifications, and standard practices of most developed countries are entirely sufficient for use in the offshore concrete industry. Somewhat different values for water-cementitious ratio, cementing material content, and concrete cover over reinforcing bars may be required because of the marine exposure but these values are well documented. Examples are shown in Tables 2 and 3.

The concrete provided for offshore North Sea platforms has seen a gradual evolution of cube compressive strength from 50 to 70 MPa (7200 to 10100 psi) (19). Table 4 shows the strength development for platforms built by one North Sea contractor. The unique environment in which this concrete is used demands this high quality of concrete. The Ravenspurn North platform (20,21) is in a more moderate environment in the southern part of the North Sea and required only 50 MPa (7250 psi) concrete which was delivered from local ready-mix suppliers. The early concrete platforms made in the Gulf of Mexico used concrete with cylinder compressive strengths from 25 to 35 MPa (3600 to 5000 psi). Recent samples from some 33 year old platforms in the Gulf of Mexico showed an increase in strength from 50 to 69 MPa (5000 to 10000 psi) over the life of the structure (22,23). The actual strength required for a given structure depends on a large number of factors but is significantly influenced by the environmental and operating loads. When these are small, the strength of the concrete can usually be consistent with that which is commonly made in the region of the construction.

CONCRETE MATERIALS

As noted earlier, the constituents of the concrete can be local materials. They must be evaluated, however, to ensure that they have the proper concrete making characteristics and will be durable in the environment in which they are used. Most offshore concrete platforms have a service life of 20 years or more. Because of their offshore location, they are not easily accessible for remedial work when problems occur. To eliminate the high cost of future offshore repair work, the materials used and the resulting concrete must be virtually maintenance free for the service life of the structure.

The durability of offshore and marine Portland cement concrete is generally defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration while retaining its original form, quality, and serviceability when exposed to its environment. This includes resistance to deterioration from freezing and thawing action, chemical attack by the constituents of the seawater, physical abrasion due to wave action, floating or suspended solids and debris, and floating ice, corrosion of steel or other metals imbedded in the concrete, and chemical reactions associated with aggregates in the concrete (22). When considering all these deteriorating actions collectively, it is easily deduced that the most aggressive exposure a concrete can routinely experience is in a tidal zone in freezing weather. For most offshore structures, the most prevalent of the destructive mechanisms is the corrosion of reinforcing bars associated with ingress into the concrete of chlorides from the seawater.

Portland cements should have as low of a tricalcium aluminate (C_3A) content as is practical with the local cement production. This helps to reduce the possibility of attacks from sulphates. The total alkali of the cement, calculated as sodium oxide, should not exceed 0.60 percent to minimize any potential for reactivity with the aggregates. The cement should have some finely divided siliceous material added to it (23). This includes natural pozzolans, fly ash, granulated slag, or condensed silica fume. These products contribute to the formation of a denser binder which inhibits the migration of the seawater into the concrete. They also combine with the alkalis to reduce the amount of available alkalis.

Coarse aggregates can be either normal density gravel or crushed stone, or good quality lightweight aggregate. The aggregates should be evaluated with respect to their potential for reactivity with the alkalis in the cement. Those aggregates which are potentially reactive should not be used. Aggregates from areas in close proximity to the sea, should be checked for concentrations of sea salts. These salts must be washed from the aggregate before it is used. Fine aggregates can be either natural or manufactured sands. They, too, must be non-reactive and free from deleterious materials.

In no instance should seawater or brackish water be used to make the concrete. All mixing water should be potable. Washing of aggregates should also be done with potable water.

Chemical admixtures are essential for the production of durable marine concrete. Air entrainment is needed when cycles of freezing and thawing can occur.

High-range water reducing admixtures (HRWRA), commonly called superplasticizers, are required for both consolidation assistance and for improved durability. An HRWRA will allow mixing water reductions up to 30 percent without sacrificing workability. This water reduction significantly reduces the permeability of the concrete and contributes to a densification of the binder fraction of the concrete.

CONCRETE PROPERTIES

Of importance to structural designers are the properties of the materials of construction at an age when appreciable loads are applied to the structure. For most offshore structures, the maximum loadings occur when the structure is put into service. This can vary from 1 to 5 years from the start of construction depending on the size and complexity of the structure and its ultimate use. The properties of hardened concrete that are used by the designers of offshore concrete platforms are:

a. compressive strength,
b. tensile strength,
c. modulus of rupture,
d. modulus of elasticity,
e. Poisson's Ratio,
f. stress-strain relationships,
g. fatigue strength,
h. absorption,
i. shear strength,
j. creep and shrinkage,
k. shear friction capacity,
l. bearing strength,
m. and, thermal properties such as the coefficient of thermal expansion, thermal conductivity, specific heat, and diffusivity.

The numerical value of each of these properties is generally not critical because the design process can usually use whatever values the selected concrete produces. The specific properties may not always be complimentary, however. For example, a very high compressive strength concrete (e.g., 65 MPa (9400 psi)) may allow compressive structural members to be reduced in cross-section for a given loading. If, however, the corresponding increase in the modulus of elasticity of that concrete allows cracking to occur at lower strain levels, then additional reinforcement may be required to reduce the cracking. Because the cross-section has now been reduced, the additional reinforcing steel adds to the congestion within the wall and makes the concrete placement more difficult. The cost of the in-place reinforcing steel may also be more than the reduction in cost due to using less concrete. Trade-off's between the various properties of the concrete should be attempted, where possible, to achieve the most efficient and cost-effective design.

All of the hardened concrete properties should be determined at advanced ages for the specific concrete to be used in an offshore structure. Unfortunately, this is not always possible and early age properties (e.g., at 28-days age) are often used. This gives the design a conservative flavor but it may add substantial costs to the structure. There is a risk associated with extrapolating early age data, particularly with high strength concretes, because the improvement of concrete properties with age may not always follow assumed trends.

Other properties of the concrete are of concern to the constructor rather than the designer. These include:

a. workability,
b. pumpability,
c. unit weight,
d. air content,
e. consolidation,
f. thermal gradients,
g. and, finishing.

The inter-relationship of these properties is a complex problem. Of utmost importance is the unit weight of the concrete. For a structure of given dimensions and configuration, and that may also be required to carry a fixed amount of dead load while floating, variations in the concrete unit weight may adversely affect the floating stability of the structure, causing it to sink or overturn. The in-place unit weight, in turn, is affected by the mixture ingredients, their proportions, and the void content, which is both a function of the entrained air content and the entrapped air or voids remaining after consolidation. If the mixture does not have adequate workability to surround the high levels of reinforcing bars that may occur, additional voids could result in the concrete. The absorption values determined on the hardened concrete are applied to the hardened density of the concrete to establish what the concrete density is when the structure is in the water. If the actual density varies significantly, so will the actual absorption values which will be different than those used in the design process.

As described later, the typical structural members in an offshore platform are quite thick. Because most offshore codes require fairly high cement contents (see Table 2) for durability purposes, the possibility of significant heat development within the concrete exists. Limiting values for the maximum placing temperature and the maximum heat rise are contained in the Codes. Even when meeting these requirements, care must be exercised to minimize thermal gradients so that thermal cracking of the structural members does not occur.

The finish of the concrete surface of an offshore structure may seem like a non-critical item, but a poor finish can have several undesirable effects beside appearance. For most offshore structures, the governing design load is caused by the forces from sea waves acting on the surface of the structure. Rough surfaces tend to gather more wave forces and thus reduce the factor of safety planned for a structure. In cold climates, an initially rough surface tends to degrade faster when subjected to cycles of freezing and thawing because there are receptacles in the surface of the concrete for water to collect and freeze. In ice-infested waters, ice moving against and past a structure tends to abrade rough surfaces faster than smooth surfaces (23).

Other properties of the concrete that are usually not of concern to either the designer or the constructor, are the durability properties. These are of concern to the owner as the offshore structure is usually part of a profit making venture that has a prescribed life-time. Some of the durability aspects of the concrete, such as freezing and thawing resistance, are addressed in the Code requirements. Matters such as the air void system in hardened concrete, as defined by spacing factor, specific surface, and voids per millimeter (inch), have specific requirements which must be met. Guidance is also provided in the Codes to prevent or mitigate such deleterious effects as sulfate attack and alkali-aggregate reactivity through proper materials selection.

Chloride-ion permeability of the concrete should also be evaluated to insure that a satisfactory concrete is being provided to resist reinforcing bar corrosion. Although minimum concrete cover over the reinforcing bars is specified for a given exposure zone (see Table 3), this may have to be increased if the concrete to be used in the platform does not have adequate resistance to chloride-ion penetration.

The abrasion resistance of the concrete to water-borne sediments, debris, floating objects, and ice is usually not specified as it is a rather site-specific phenomenon. In offshore areas where significant abrasion can occur, such as ice-infested waters, the resistance of the concrete to the abrading medium must be evaluated and loss rates for the concrete surface determined (24). Once these rates are known, measures to accommodate or eliminate the losses, such as additional concrete cover or steel plates in the abrasion zone, respectively, can be implemented.

An evaluation of all the concrete properties noted above for a specific concrete for a specific structure is the ideal situation, but it has not often been done. When actual numerical values are not available, conservative approximations are chosen and these result in a satisfactory, but not necessarily cost efficient, design. One study that addressed most of the properties noted above, was performed on high strength lightweight aggregate for use in offshore Arctic structures and is described in (25) to (27).

DESIGN CONSIDERATIONS

As noted above, concrete offshore structures can be bottom-founded or floating. With the exception of a structure which has its base made entirely from prestressed concrete piles, most of the other bottom-founded structures are in a floating mode at some time in their early life. These structures must then include design provisions for both bottom-founded operational loads as well as those loads associated with the structures behavior as a ship.

Design codes and guidelines for offshore concrete structures have been developed by various regulatory agencies and standards groups. A listing of some of the major codes and regulations is shown in Table 5. These are constantly being upgraded as the technology advances. In general, detail design of the individual elements of an offshore concrete structure for such things as shear, tension, flexure, compression, eccentric loads, etc., is not significantly different than for any other type of concrete structure. It is only the types of loads, their frequency and duration, and their magnitude that differs from ordinary civil engineering structures.

The principal loads the offshore structure encounters are permanent loads, variable functional loads, environmental loads, accidental loads, and deformation loads. These various loads are combined in realistic manners to determine their net effect.

Permanent loads include the weight of the structure, any permanent equipment, ballast that will not be removed, and the external hydrostatic sea water up to mean sea level. Variable functional loads are the loads associated with the normal operation of the structure. Loads in this category that are unique to offshore structures include variable ballast, installation and drilling loads, vessel impact, fendering and mooring, weight of petroleum products temporarily stored in the platform, helicopter loads, and crane operations.

Environmental loads include waves, wind, current, ice and snow, and earthquake. Accidental loads include fire, explosion, ship impact, unintentional flooding, unintentional ballast distribution, and changes in presupposed pressure differences. Examples of deformation loads include prestressing, concrete shrinkage, and thermal gradients.

The geotechnical considerations offshore are much more complex than onshore. For bottom-founded structures, this is an extremely critical area of design. The anchors and moorings of floating structures are also significantly influenced by the subsea soil conditions. Seismically active areas warrant special consideration. Specialists in subsea foundation problems, not onshore foundation specialists, should always be used to work this part of the design problem.

For the initial offshore concrete structure in a country or region that has never used one before, it is desirable to use the design expertise of companies or firms that have prior experience with these structures. Such firms exist in North America, Europe, Scandinavia, United Kingdom, and Japan. By involving local design firms in partnerships with these experienced firms, the philosophy and mechanics of the design process can be transferred to the local regions.

SAFETY CONSIDERATIONS

Modern offshore concrete platforms are designed with sufficient redundancy to resist major accidental loads. Concrete has exceptionally good impact resistance and only a few isolated instances of structural damage due to ship impact have been reported. Sufficient ductility can be designed into structural concrete elements to eliminate the problem of progressive collapse. The fire resistance of concrete is well known with concrete often being used to protect steel from fires in many major structures. A summary of the service record of concrete platforms in the North Sea can be found in (28).

Floating concrete structures are designed for one compartment damage stability which means that any local damage which causes leakage of the sea into the hull will not cause the floating structure to be at risk. Similar criteria are applied to the bottom-founded structures so that they are not at risk while in their floating mode. Bottom-founded structures, such as that shown in Fig. 1b, have the unique capability of having each of its shafts or towers operate independently of the others. All living quarters and other major personnel areas can be isolated on top of one shaft and kept removed from the more dangerous areas where drilling and processing of hydrocarbons takes place. All areas can be connected by bridges. This is a significant advantage over a structure where most of the supporting structural members are tied together in some fashion and collectively support all the operations of the platform. In the event of a major fire or explosion in or on the shafts where the hydrocarbons are present, the living quarters would not be affected and successful evacuation of the GBS type platform could take place.

CONSTRUCTION PRACTICES

There are no unique construction practices needed to build an offshore structure such as those described above. The good practices employed for any major civil engineering project are sufficient for building an offshore concrete platform. This gives the owner flexibility in selecting construction contractors.

The large offshore concrete platforms of the North Sea have been predominately constructed using slipforming (29). It has been demonstrated that the North Sea slipforming techniques can be satisfactorily applied to offshore concrete construction in the hotter climate of the Gulf of Mexico (30). The small barge-like platforms in the Gulf of Mexico are cast-in-place (3). The Hibernia platform (31) for the East Coast of Canada, which is scheduled for completion in 1995, will be jump-formed. Precast elements have been used in some structures (32). The method of construction can be anything that works and should be left to the discretion of the contractor.

The concrete can be delivered to the form by pump, boom, conveyor, bucket, buggy, barrow, or, again, anything that works. Sophisticated distribution systems normally are not needed. The distribution of the concrete into the slipforms used on North Sea structures has been by wheel-barrow.

Proper and sufficient consolidation of the concrete is essential. The equipment and procedures to do this already exist. Adequate curing must be provided. This includes protection from early freezing in regions where this is a possibility.

The two items of offshore concrete construction that differ from most onshore concrete construction, are the thickness of the concrete elements and the amount of reinforcing steel that is used. The thinnest concrete walls are usually 350 mm (13 in.). The thickest walls can be several meters (greater than 6 feet) thick. Typical wall thickness's are 500-600 mm (20-24 in.). Temperature control of the concrete in these thick walls is essential to eliminate thermal cracking problems. Reinforcement densities typically average 400 kg/cu m (676 lb/cu yd). Extremes in critical areas have been as great as 1100 kg/cu m (1859 lb/cu yd). The proportioning of the concrete and the consolidation methods must be tailored to insure the reinforcing steel is completely encapsulated by the concrete.

In general, the maximum size aggregates used in these structures has been 19 mm (3/4 in.) or less to allow the concrete to move around the large amounts of reinforcing bars. The use of high-range water reducers (superplasticizers) is a necessity.

Prestressing is also used in almost every structure. This is to insure water tightness of the concrete. The amounts of prestressing are structure and location specific. Standard prestressing materials and practices can be used for these structures with a view that the work will be done in a marine environment. Special care must be taken to protect end anchorages of the prestressing from the corrosive environment of the sea.

CONSTRUCTION LOCATIONS

Bottom-founded and floating offshore concrete structures can be constructed, either partially or completely,

1. In dry docks or graving docks,
2. On submersible barges,
3. On skid ways, or
4. In precast facilities onshore.

Precast facilities are used principally for the precast concrete piles and template decks. These facilities can also be used to prefabricate substantial portions of a structure. The prefabricated elements are then transported to and assembled at the other locations. The construction of the concrete Arctic drilling structure Glomar Beaufort Sea I, also known as the CIDS (Concrete Island Drilling System), had most of its interior concrete elements made in a prefabrication facility (32). The actual method selected will depend greatly on the existing site facilities where construction is planned and on the economics of the project.

Dry Dock Construction

The principal method of construction used for the large North Sea concrete platforms is to begin the construction in a dry dock or graving dock. Fig. 11 shows the construction sequence. Once the base of the structure becomes stiff enough and has sufficient buoyancy, the dock is flooded and the base allowed to float. It is then towed from the graving dock to a deeper water location where it is temporarily moored. Construction then continues at that location. Depending on available water depths, this location can be close to the shore or a great distance away. If the structure is located close enough to shore to be reached by a floating or fixed bridge, materials and personnel can reach the structure directly from the shore. Office facilities and concrete production facilities can remain onshore. If the structure is located some distance away from the shore, it will require adjoining barges which support offices, materials laydown areas, concrete batching plants, and other essential equipment. Concrete materials are obtained directly from supporting ships. Work crews are shuttled back and forth from the shore by crew ships.

Once the concrete construction is complete, the structure may be moved to an even deeper water location where it is mated with the topsides equipment and hook-up of the equipment is begun.

It generally costs less to construct a GBS entirely in a dry dock rather than partially in the dry dock, at a wet dock or at a jetty mooring. Productivity is generally higher in the dry dock and specialized equipment is minimized. Construction risk is usually lower. For smaller concrete platforms, the platform can be built entirely in the dry dock with sufficient buoyancy to be floated out and towed to its final location. The Ravenspurn North platform (19,20) and all the barge-like platforms (Figs. 3 and 4) built for the Gulf of Mexico are constructed in that manner. Fig. 12 shows an exploded view of various components of the Ravenspurn North platform (33). It is reported (34) that in Europe, concrete GBS's for water depths from 100 to 150 m (330 to 490 ft.) can be constructed entirely in a dry dock. The limit is governed by a combination of gate width and sill draft. As noted above, the total support for complete dry dock construction is land-based and very similar to constructing a concrete building. Very large platforms can also be built in this manner but will require the use of auxiliary buoyancy compartments for floatation. These compartments can be designed to be removed after platform installation or can be left in place. Some small platforms have also used additional buoyancy compartments to satisfy installation requirements.

A major expense in the construction of a concrete platform is the development of a dry dock if one does not exist, or the modification of existing dry dock facilities to accommodate the construction. This expense can include land procurement, excavation, cofferdam construction, dewatering systems, dredging of channels for float out, construction of supporting quays, docks and wharves, and the overall upgrading of the infrastructure to improve project support (roads, bridges, power supply, water supply, sewage treatment, etc.). These costs can easily reach 80 percent of the project cost in remote areas.

Construction on Barges

To eliminate much of the costs associated with the construction of a dry dock, the construction of some or all of the platform on submersible barges is a practical solution. This eliminates the need for a dry dock and thus greatly expands the potential for construction site locations. Fig. 13 shows a typical barge construction scenario. Any location with sufficient water depth for barge and supply ship operation is a possible construction site. Preferably this site should be in sheltered waters and not subject to severe seas. The number of barges required will depend on the size of the structure and the capacity of the barges. Special built barges are always a solution but it may be more economical to weld together a sufficient number of smaller, standard size barges to accomplish the same objective. Once the barge or barges have been assembled to provide the working platform, construction then proceeds on the barge(s) as if it was on land. The construction support will depend on the water depths available. Initially, the construction can probably begin with the barges moored adjacent to the shore so that all construction support can come directly from the land. Fig. 14 shows the concrete tether anchors for the Snore Tension Leg Platform (TLP) which were constructed entirely on barges with direct land support. If the platform is small, the entire platform can be built on the barges at one location. If the platform is large, only a portion would be built on the barges. That portion would be floated off the barges, temporarily moored, and construction completed while in a floating mode.

Skidway Construction

Skidways exist in almost all marine fabrication yards. A skidway is basically a structural slab having a small slope that extends from a construction area down into the sea. Structures that are built on the skidway can be gravity assisted as they are moved down the slope and into the sea. The structures can be self-floating or can be skidded onto barges.

For a concrete GBS, the structure can be built either in part or in its entirety on the skidway (35). Small structures (Fig. 15) are the most likely candidate for this type of construction. Because the weight of a complete large concrete structure is so large, there is probably an upper limit where any given skidway could not be used without significant structural upgrading. This upgrading may not be cost effective but warrants consideration. The base of most concrete structures can probably be constructed on the skidway to a level where it would be self-floating. It can then be skidded into a floating mode and towed to a deeper water site for mooring and completion while floating.

Site Limitations

The principal limitation for any of the construction sites, is its access to the open sea or to maintained ship channels. As most of the structures will float at some time during their construction and installation phases, water of sufficient depth for them to float and to be towed to their final location must exist in close proximity to the construction site. Dredging of new channels is very expensive and may not be possible due to physical, environmental or political reasons. Most large structures with deep drafts have been completed while in a floating mode. When this is done, the deep water requirement is further constrained because of a need to have this deep water location in a relatively mild environment with respect to wind, waves and current. It is also desirable to have this deep water location close enough to shore so that materials supply and construction crew changes can be made without having to stockpile large amounts of material offshore or to provide temporary offshore quarters for personnel. Both of these aspects of construction will add greatly to the cost of the project.

If a dry dock is required and must be built, the supporting infrastructure must be carefully studied before a site is selected. If the site is not convenient to major population areas, the site development will have to include such things as accommodations for work crews, dining facilities, water supply, sewage disposal, recreation facilities, communication facilities, transportation and all the other things required for a small city.

As noted above, the existence of skidways in operating shipyards does not necessarily mean they can be used for concrete structures because they may be undersized or structurally inadequate. The dredged channels leading away from skidways most likely will not be of sufficient depth for the draft of larger concrete platforms and some additional dredging may be required.

MARINE OPERATIONS

The subject of marine operations is outside the scope of this paper, but it should be noted that offshore development and marine operations go hand-in-hand. The construction and installation of any type of offshore concrete platform will involve significant marine operations (36). The structure itself must float, be towed, and perhaps submerged for final installation.

The structure may have its topsides equipment installed in a mating operation. In this operation, the structure is ballasted down until it is almost submerged. The assembled topsides is then floated over the submerged structure. Deballasting is initiated, and as the structure rises, it picks the topsides up off the barges. Deballasting is continued until the structure reaches its final draft for towing operations. Hook-up of the topsides to the structure is then done. This mating is a critical marine operation.

Marine operations are expensive and may require special vessels. Fortunately, many good specialist firms or consultants with offshore structure installation experience exist around the world and they should be consulted on this aspect of the use of concrete platforms.

COST CONSIDERATIONS

The costs of an offshore structure are usually dictated by the complexity of the structure, the location where it will be built, and the methods of construction used. These structures have the potential for low capital costs because the materials are local and the

bor skill level usually does not have to be high. When properly made, concrete can be rtually maintenance free thus producing an attractive life-cycle cost. Detailed cost scussions are beyond the scope of this paper but a cost philosophy for these structures described in (37) and (38).

SUMMARY

The above information is intended to provide a general overview of what is required the use of concrete for offshore structures. A significant amount of information on the bject is distributed throughout the literature but has not yet been assembled in a useful xt book form.

Concrete is the material of choice for permanent constructions in all regions of the orld. It can be produced with local labor and local materials. The potential exists for use the offshore oil and gas industry in developing regions of the world and in existing fshore regions where steel structures have been predominately used before. The use of ncrete for these structures generally must be sold to owners or developers who are used dealing with steel structures.

For use in offshore structures, the concrete technology is well established. No special aterials are required, the concrete quality is not unusual, design requirements are well tablished, competent offshore concrete design firms exist in many regions of the world, special construction practices are needed, flexibility exists in construction site selection, pital costs can be low, and life-cycle costs can be very attractive. Because of the vere environment that exists offshore, the durability aspects of the concrete will require ecial attention.

The potential for the use of concrete in offshore structures is great and is limited only the ingenuity of the concrete designer and constructor and their ability to sell their ideas the owners.

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TABLE 1 -- SUMMARY OF FLOATABLE/BOTTOM-FOUNDED CONCRETE HULL STRUCTURES FOR THE GULF OF MEXICO CONSTRUCTED BY ONE CONTRACTOR BETWEEN 1984 AND 1989 (4)

Year Installed	Location	Water Depth m (ft.)	Hull Dimensions width x length x height (m x m x m)	Hull Dimensions width x length x height (ft x ft x ft)
1984	Eugene Island Block 45	6.1 (20)	17.7x17.7x3.7	(58x58x12)
	Bayou Sorrel	2.4 (8)	9.4x11.0x3.7	(31x36x12)
	Cox Bay North	2.1 (7)	19.5x46.9x3.0	(64x154x10)
	Cox Bay South	2.1 (7)	19.5x33.5x3.0	(64x110x10)
	Vermillion Block 72	7.0 (23)	17.7x17.7x3.7	(58x58x12)
	Eugene Island Block 45	7.0 (23)	18.3x18.3x3.7	(60x60x12)
1985	Brenton Sound[1] Block 1	2.7 (9)	18.3x26.8x3.7	(60x88x12)
	Lease Platform	NK	8.5x30.5x3.7	(28x100x12)
	Eugene Island Block 44	6.4 (21)	17.7x17.7x3.7	(58x58x12)
	South Pass	2.4 (8)	21.9x49.4x3.7	(72x162x12)
1986	Gordon Island Bay	NA	10.7x19.8x3.7	(35x65x12)
	Lease Platform	NK	8.5x30.5x3.7	(28x100x12)
	Brenton Sound[2] Block 2	3.7 (12)	18.3x24.4x3.7	(60x80x12)
	Quarantine Bay	NK	21.3x38.7x3.7	(70x127x12)
1987	Delta Dock Fld	NA	12.2x30.5x3.7	(40x100x12)
	Pt. Ala Hache	NK	21.3x40.8x3.4	(70x134x11)
	Pt. Ala Hache	NK	28.0x32.9x3.4	(59x108x11)
	W. Lake Verret	NK	15.2x29.9x3.7	(50x98x12)
1988	Chandeleur Snd.	NA	4.6x7.6x3.0	(15x25x10)
	Atchsduya Bay[3]	N/A	15.5x15.5x4.3	(51x51x14)
	S. Marsh Island Block 253	NK	18.3x18.3x3.4	(60x60x11)
1989	Main Pass Block 69	NA	12.2x18.3x3.7	(40x60x12)
	Eugene Island Block 30	4.3 (14)	21.9x25.0x4.0	(72x82x13)
	West Bay		21.9x42.7x3.7	(72x140x12)

not known.
not applicable. Barge structure.
Designed for 5000 bbl of storage.
Designed for 4500 bbl of storage.
Designed for 2500 bbl of storage.

TABLE 2--SUMMARY OF RELEVANT CODE REQUIREMENTS FOR WATER/CEMENT RATIO, CEMENT CONTENT, AND COMPRESSIVE STRENGTH

Codes (See Table 3)	Exposure Zones		
	Submerged	Splash	Atmospheric
ACI 357			
Maximum water/cement ratio	0.45	0.40	0.40
Minimum cement content, kb/cu m (lb/cu yd)	356 (600)	356 (600)	356 (600)
Maximum cement content, kg/cu m (lb/cu yd)	415 (700)	415 (700)	415 (700)
Minimum 28-day cylinder compressive strength, MPa (psi)	35 (5000)	35 (5000) 42 (6100) if subject to abrasion	35 (5000)
CSA S474			
Maximum water/cement ratio	0.45	0.40	0.40
Minimum quantity of cementing material, kg/cu m (lb/cu yd)	360 (610)	400 (675)	400 (675)
Minimum quantity of portland cement, kb/cu m (lb/cu yd)	300 (510)	300 (510)	300 (510)
Minimum cylinder compressive strength, MPa (psi),			
28 days	30 (4350)	40 (5800)	40 (5800)
91 days	35 (5000)	45 (6500)	45 (6500)
FIP			
Maximum water/cement ratio	0.45 (less than or equal to 0.40 preferred)		
Minimum cement content, kb/cu m (lb/cu yd)	320 to 360* (540 to 610)	400 (675)	320 to 360* (540 to 610)
Maximum cement content, kb/cu m (lb/cu yd)	500 (845)	500 (845)	500 (845)
Minimum 28-day cylinder compressive strength, MPa (psi)	32 (4650)	32 (4650) 40 (5800) if subject to abrasion	32 (4650)

(continued)

TABLE 2 -- CONTINUED

Codes (See Table 3)	Exposure Zones		
	Submerged	Splash	Atmospheric
DnV			
Maximum water/cement ratio	0.45 (less than or equal to 0.40 preferred)		
Minimum cement content, kb/cu m (lb/cu yd)	300 (510)	400 (675)	300 (510)
Maximum cement content, kg/cu m (lb/cu yd)	---	---	---
Minimum 28-day cylinder compressive strength, MPa (psi)	---	---	---
BS 6235			
Maximum water/cement ratio	0.40	0.40	0.40
Minimum cement content, kg/cu m (lb/cu yd)	320 to 360* (540 to 610)	400 (675)	400 (675)
Maximum cement content, kb/cu m (lb/cu yd)	---	---	---
Minimum 28-day cylinder compressive strength, MPa (psi),**	32 (4650)	32 (4650) 40 (5800) if subject to abrasion	32 (4650)

320 kg/cu m (540 lb/cu yd) for a maximum aggregate size of 40mm (1.57 in.); 360 kg/cu m (610 lb/cu yd) for a maximum aggregate size of 20mm (0.78 in.).

Cylinder strength assumed to be 80 percent of specified cube strength.

TABLE 3 --SUMMARY OF RELEVANT CODE REQUIREMENTS FOR MINIMUM CONCRETE COVER

Codes (See Table 5)	Splash or External Atmospheric	Other
ACI 357		
Untreated reinforcing bars	65 mm (2.6 in.)	50 mm (2.0 in.)
Prestressing tendons	90 mm (3.5 in.)	75 mm (3.0 in.)
Cover of stirrups	13 mm (0.5 in.)	less than above
CSA S474		
Untreated reinforcing bars	65 mm (2.6 in.)	50 mm (2.0 in.)
Epoxy coated reinforcing bars	50 mm (2.0 in.)	35 MM (1.4 in.)
Prestressing tendons	90 mm (3.5 in.) A cover of 75 mm (3.0 in.) may be used in atmospheric zone.	75 mm (3.0 in.)
Cover of stirrups	15 mm (0.6 in) less than above	
FIP		
Untreated reinforcing bars	65 mm (2.6 in.)	50 mm (2.0 in.)
Prestressing tendons	90 mm (3.5 in.)	75 mm (3.0 in.)
DnV		
Untreated reinforcing bars	50 mm (2.0 in.)	40 mm (1.6 in.)
Prestressing tendons	100 mm (4.0 in.)	80 mm (3.2 in.)
BS 6235		
Untreated reinforcing bars	75 mm (3.0 in.)	60 mm (2.4 in.)
Prestressing tendons	100 mm (4.0 in.)	75 mm (3.0 in.)

TABLE 4 --STRENGTH DEVELOPMENTS FOR NORTH SEA OFFSHORE CONCRETE STRUCTURES (18)

Platform (Year)	Concrete in Cell Walls* m^3 (yd^3)	Specified Concrete Grade	Obtained 28-Day Cube Strength MPa (psi)	Typical Slump mm (in.)
Ekofisk I (1972)	---	C40*	45* (6530)	100 (3.9)
Beryl A (1974)	17100 (22370)	C45	55.0 (7980)	120 (4.7)
Brent B (1974)	40600 (53100)	C45	53.0 (7690)	120 (4.7)
Brent D (1975)	34000 (44470)	C50	54.2 (7860)	120 (4.7)
Statfjord A (1975)	47400 (62000)	C50	54.9 (7920)	120 (4.7)
Statfjord B (1979)	56700 (74160)	C55	62.5 (9070)	160 (6.3)
Statfjord C (1982)	63700 (83320)	C55	67.5 (9790)	210 (8.3)
Gullfaks A (1984)	63400 (82930)	C55	65.2 (9460)	220 (8.7)
Gullfaks B (1985)	45000 (58860)	C55	80.8 (11720)	220 (8.7)
Oseberg A (1986)	43000 (56240)	C60	76.7 (11120)	230 (9.1)
Gullfaks C skirts (1986)	17400 (22760)	C70	83.8 (12150)	240 (9.4)
Gullfaks C (1989)	115000 (150420)	C65	79.0 (11460)	230 (9.1)

Only the slipformed concrete in the cell walls except where noted. Does not represent the total concrete in the structure.

TABLE 5--SUMMARY OF RELEVANT CODES FOR OFFSHORE CONCRETE STRUCTURES

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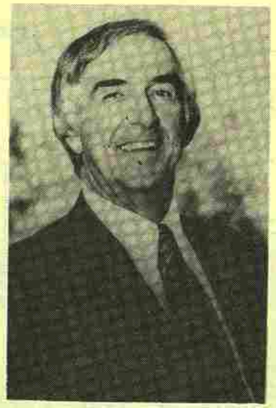


George C. Hoff
Engineering Department
of Mobil Research and
Development Corporation
Dallas, Tx., USA.

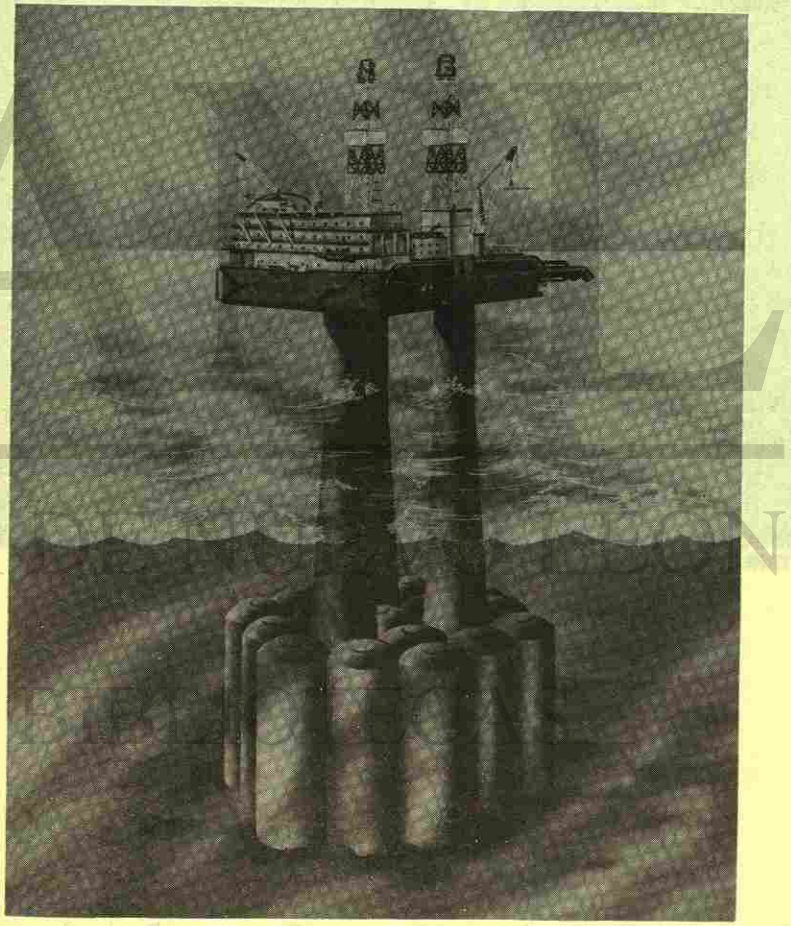


Fig. 1. Typical Gravity Base Structure.

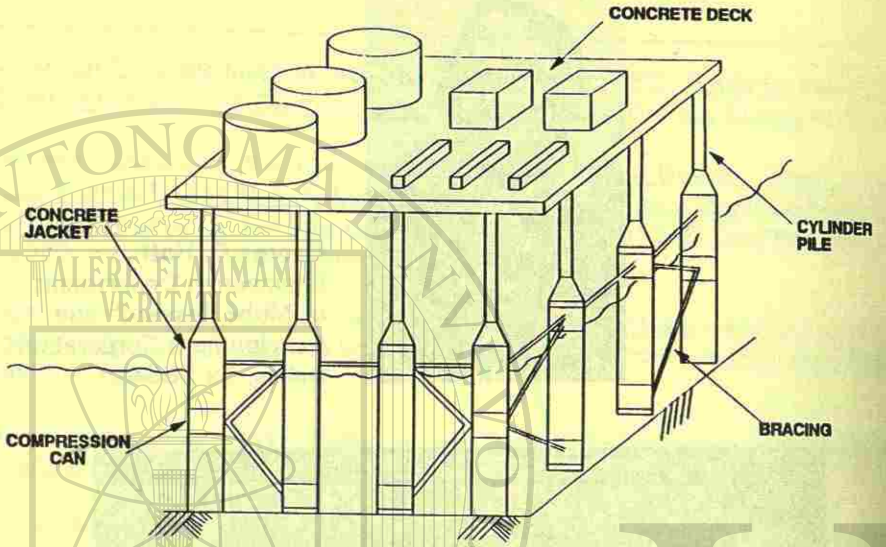


Fig. 2. Typical Cylinder Pile Supported Plataform.

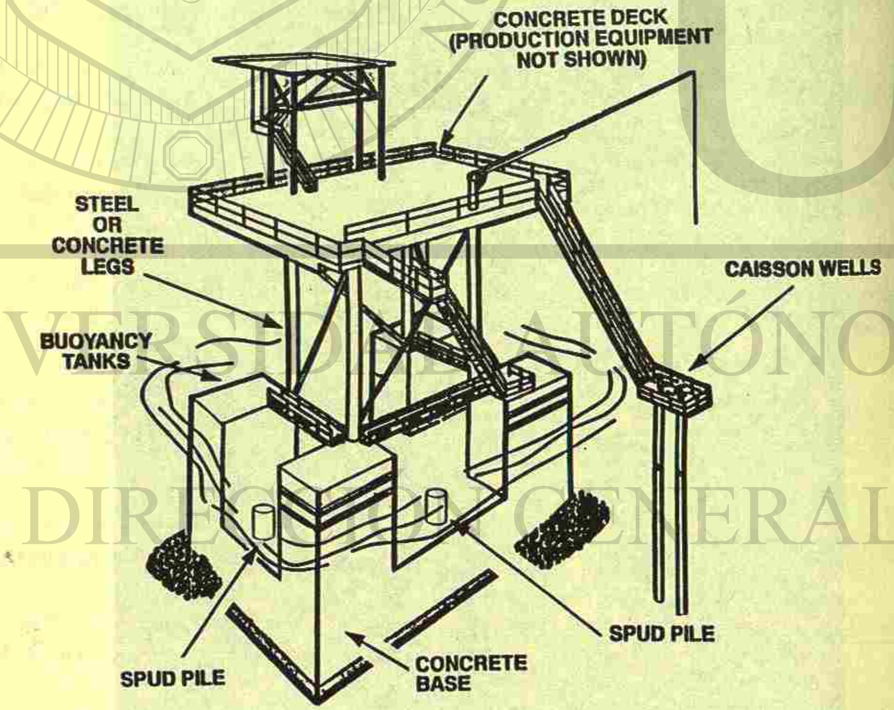


Fig. 3. Flotable/Bottom-Founded Structure (Courtesy of Production Management Structural Systems).

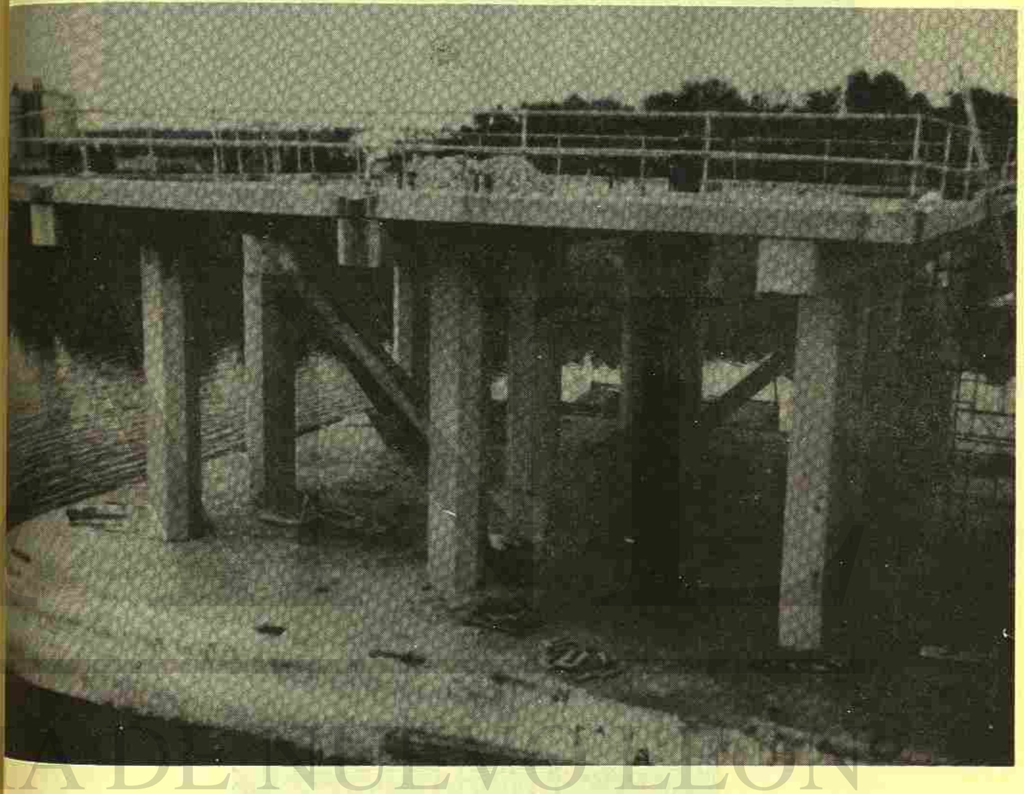


Fig. 4. Eighteen-year Old Flotable/Bottom Founded Platform After Refloating and Relocation to Wet-Dock for Equipment Modifications.

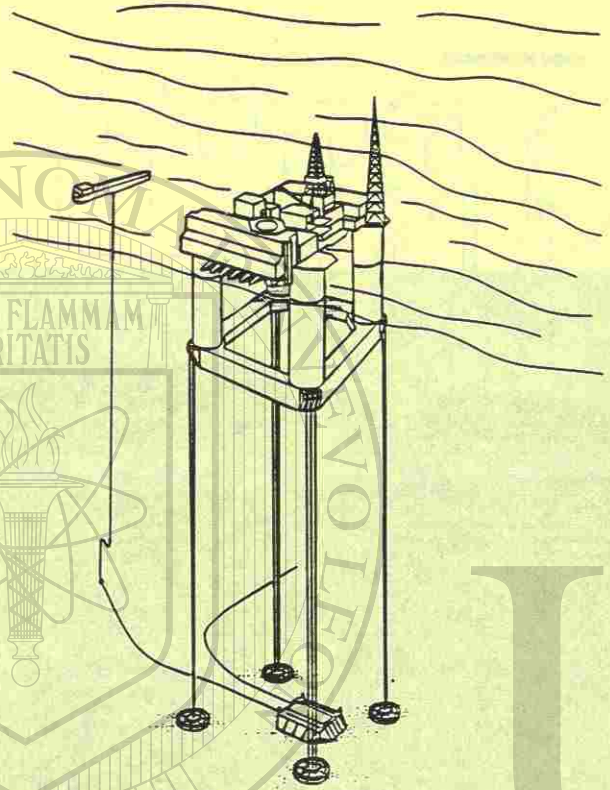


Fig. 5. Concrete Tension Leg Platform (TLP).

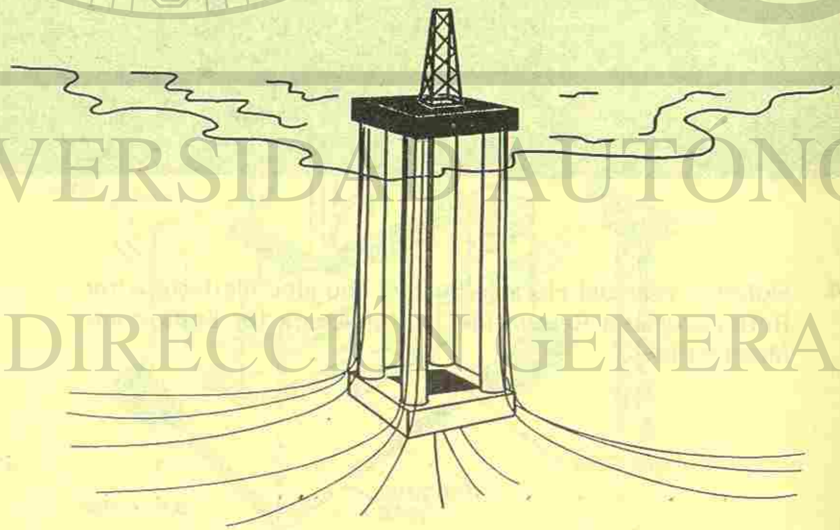


Fig. 6. Deep Draft Concrete Floater (DDCF)

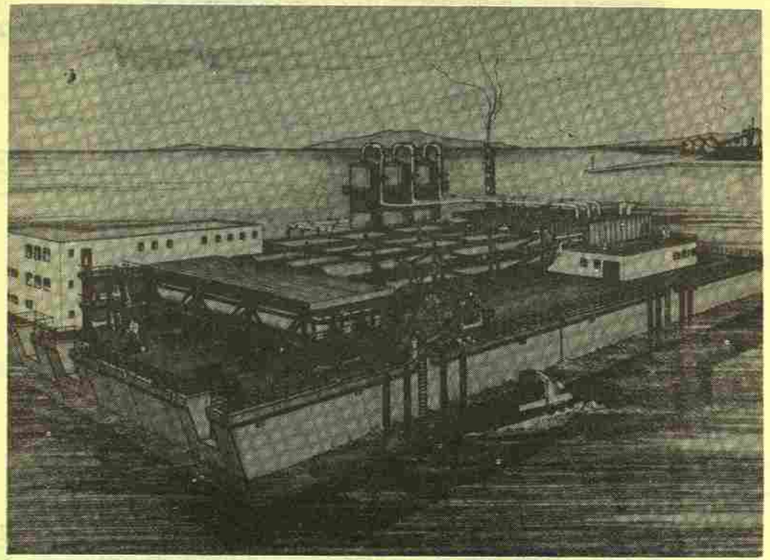


Fig. 7. Concrete Production Barge (Courtesy of Ed. Zublin AG)

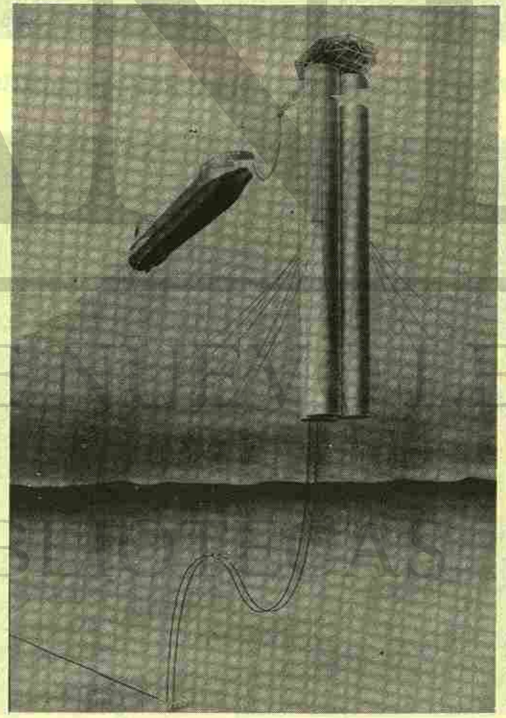


Fig. 8. Spar Buoy Platform (Courtesy of Norwegian Contractors.

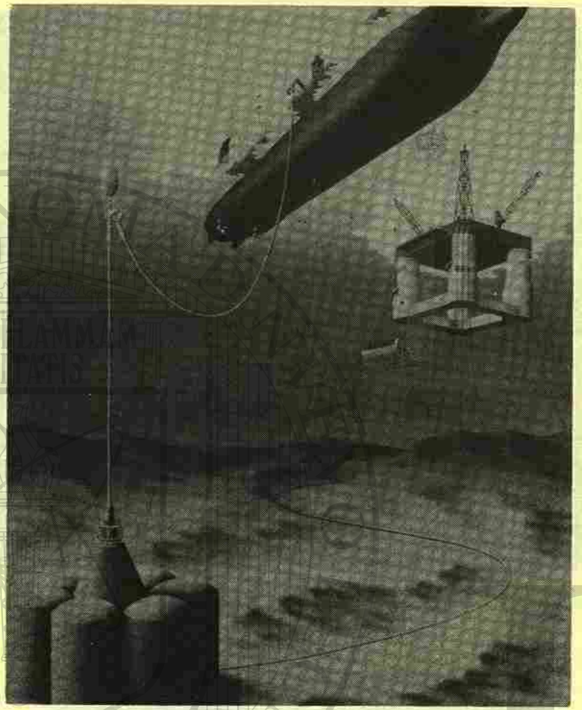
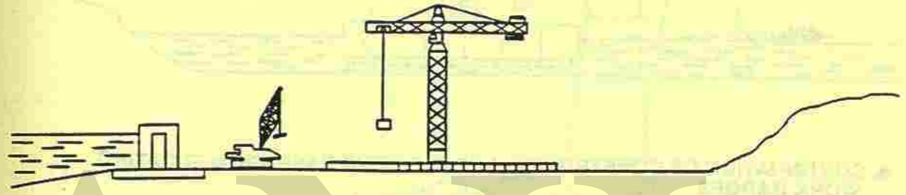


Fig. 9. Concrete Subsea Storage Tank.

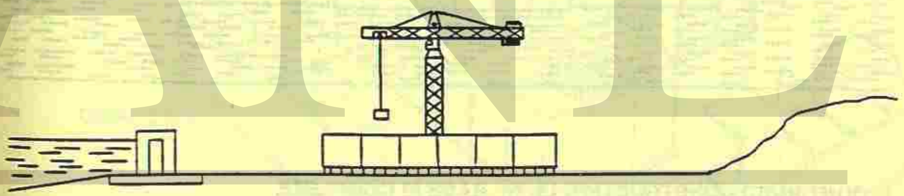
Fig. 10. Concrete Retaining Wall Caissins (14)

1. EXCAVATION OF CONSTRUCTION AREA

2. CONSTRUCTION OF SUBBASE OR BASE SLAB OF PLATFORM



3. CONSTRUCTION OF CONCRETE PLATFORM TO SUFFICIENT HEIGHT FOR TOW-OUT



4. FLOODING OF DRY DOCK

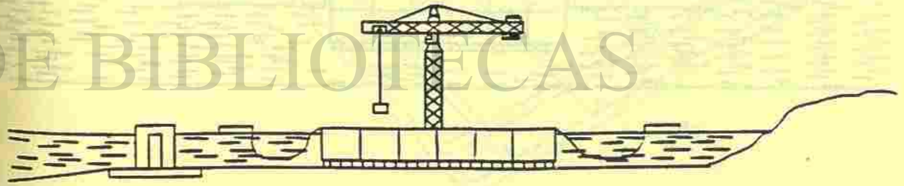
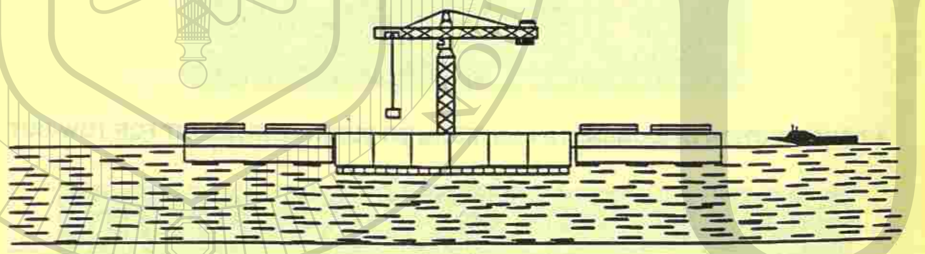


Fig. 11. Dry Dock Construction Scenario
(continued)

5. TOW-OUT OF PLATFORM TO DEEPER WATER CONSTRUCTION SITE

6. CONTINUATION OF CONSTRUCTION OF FLOATING BASE FROM FLOATING WORK BARGES



7. CONTINUED CONSTRUCTION UNTIL BASE IS COMPLETE

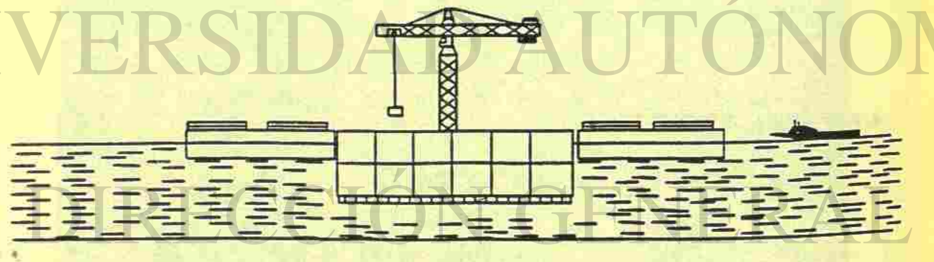


Fig. 11 (cont.) Dry Dock Construction Scenario

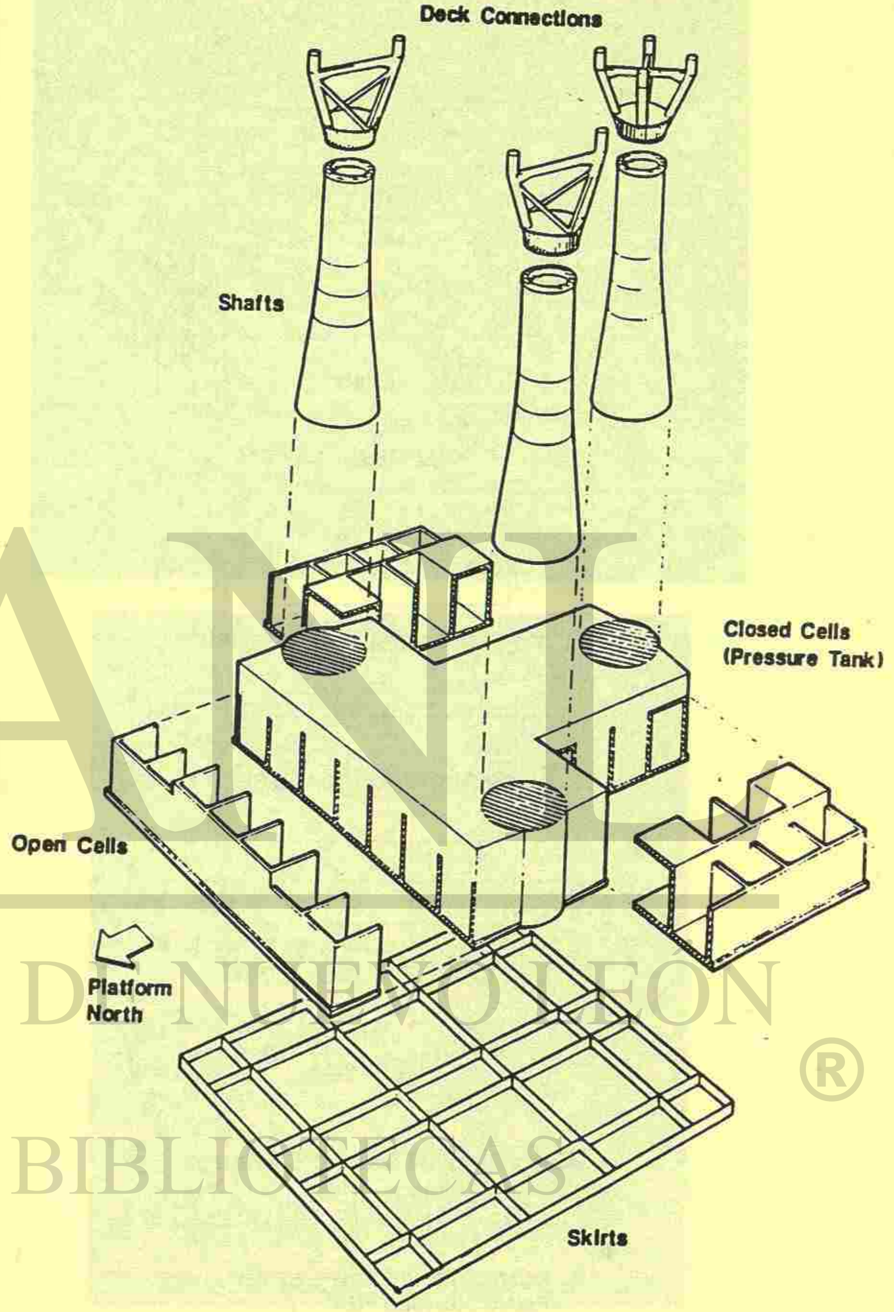


Fig. 12 Principal Components of the Ravenspurn North Concrete Platform (Courtesy of Ove Arup & Partners).

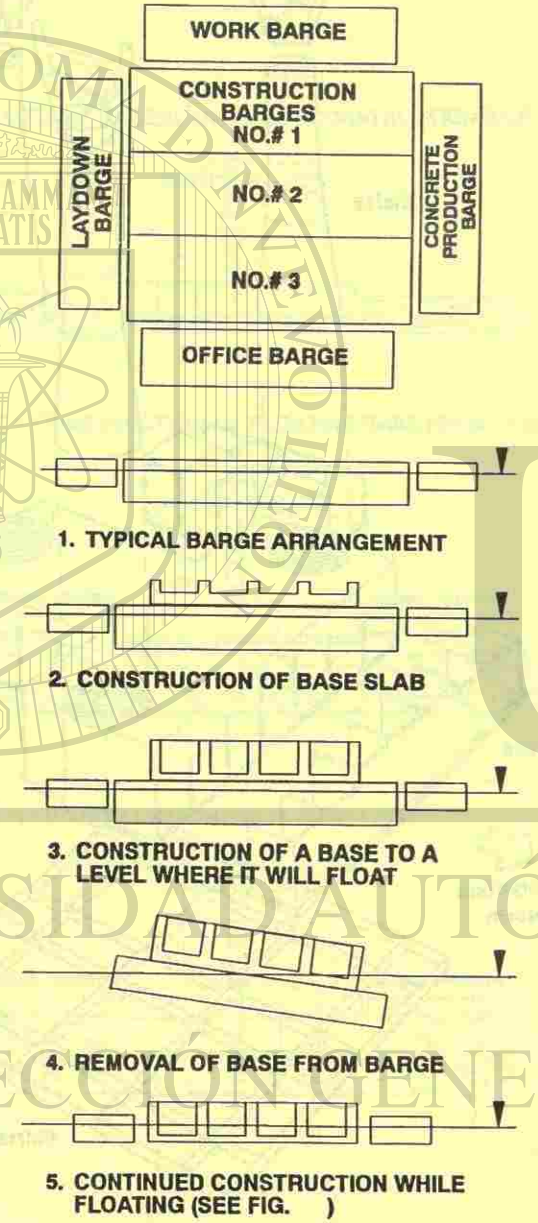


Fig. 13. Barge Construction Scenario

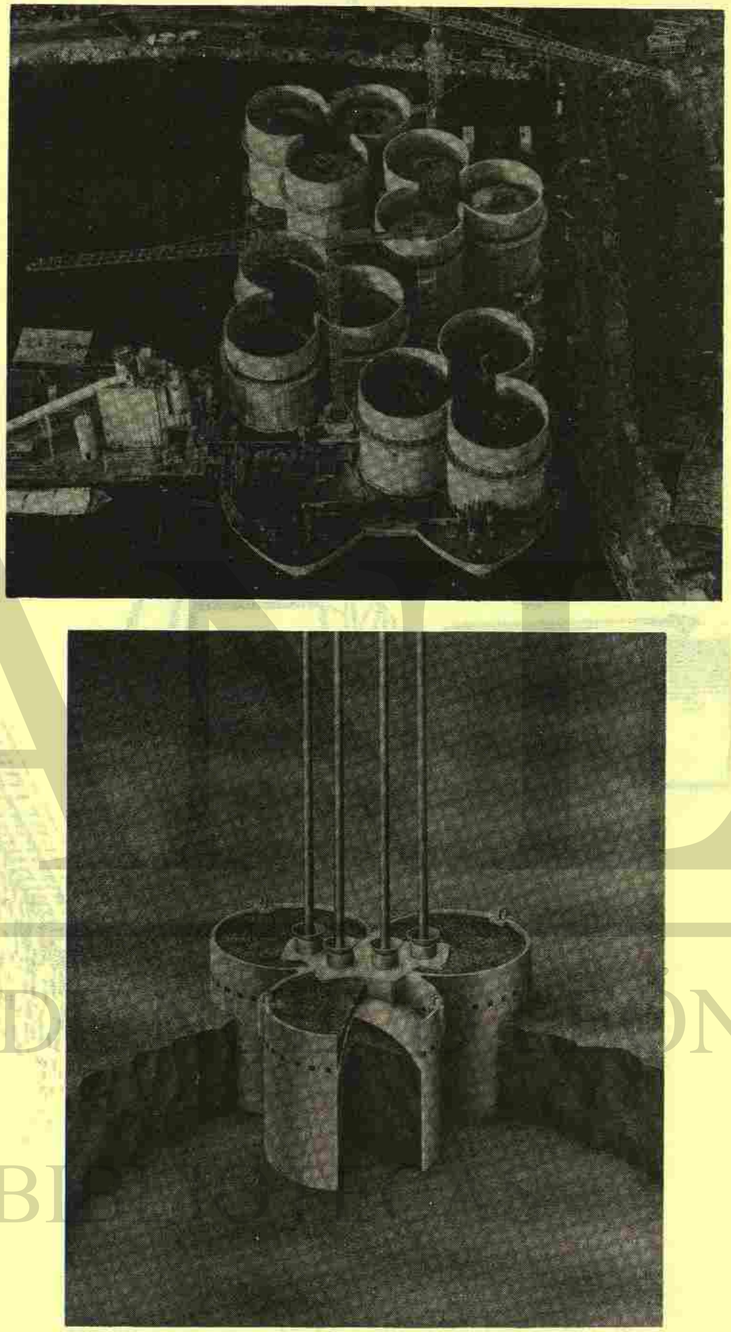


Fig. 14 Snore TLP Foundation Anchors (Courtesy of Norwegian Contractors, a/s).

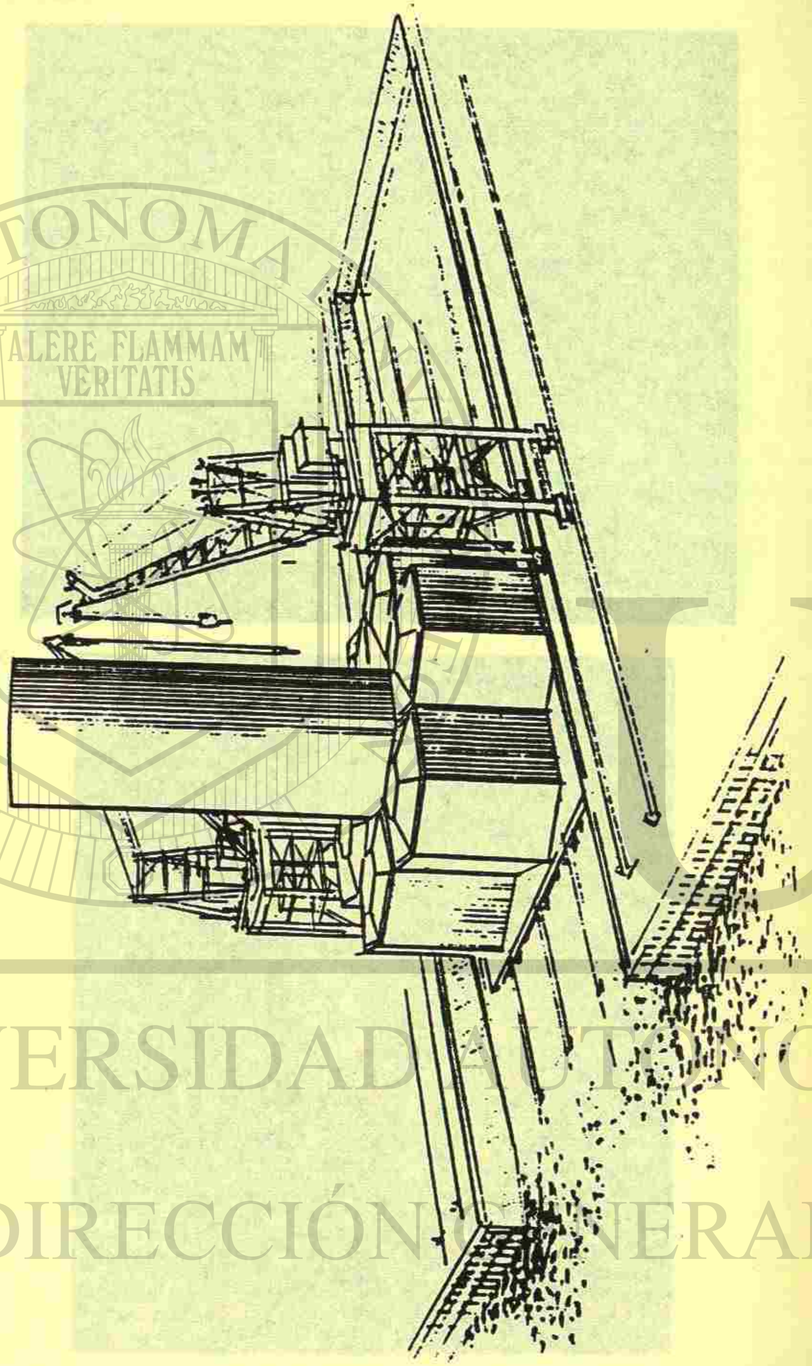


Fig. 15 Skidway Construction (Courtesy of Ben C. Gerwick, Inc.)

CONSIDERACIONES QUE AFECTAN EL PROPORCIONAMIENTO DE LA MEZCLA DE CONCRETO CONTENIENDO CENIZA VOLANT

por

R.L. Carrasquillo y P.J. Tikalsky

Sinopsis: Se presentan los resultados de tres años de estudio sobre las propiedades del concreto que contiene ceniza volante. Se reportan las propiedades de tanto el concreto fresco como el endurecido usando cemento Tipo I, arena de río, arena natural y cenizas volantes de diferentes y variadas fuentes. [illegible] para que [illegible] Se determinó que el concreto puede ser diseñado con [illegible] reológicas buenas, igual o mayor durabilidad que [illegible] Con la ceniza volante Clase C y F [illegible] a más de 1,200 [illegible] congelamiento, resistencia a la flexión, resistencia a la [illegible] resistencia a la abrasión. [illegible] contenido de ceniza volante de [illegible] del cemento portland con cenizas volantes de la Clase C y Clase F. Se presentan [illegible] para la selección de materiales y [illegible] proporcionamientos para producir concreto conteniendo ceniza volante que cumplan con [illegible] carreteras de concreto.

Palabras clave: Ceniza volante, propiedades del concreto, durabilidad, resistencia a la flexión, flujo [illegible] resistencia a la abrasión.

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Palabras clave: Ceniza volante, propiedades del concreto, durabilidad, resistencia a la flexión, flujo, retracción, resistencia a la abrasión.

CONSIDERATIONS AFFECTING MIX PROPORTIONING OF CONCRETE CONTAINING FLY ASH

By

R. L. Carrasquillo and P. J. Tikalsky

nopsis: The results of a three year study are presented on properties of concrete ntaining fly ash. Both the fresh and hardened concrete properties are reported using pe I cement, river gravel, natural sand and fly ashes from several different sources. The xes are proportioned to have equal slumps and a constant cementitious contents, by eight. It is shown that concrete can be designed with equal strength, good rheological operties and equal or better durability when a suitable ASTM C618 Class C or F fly ash used appropriately. Test data on over 1600 laboratory field specimens tested for eeze-thaw resistance, flexural strength, compressive strength, creep, shrinkage and rasion resistance are presented. Fly ash contents ranging from 0% to 35% by weight portland cement are used with both Class C and Class F fly ashes. Guidelines for the election of materials and their proportions for producing concrete containing fly ash to eet existing highway specifications for concrete are presented.

ywords: Fly ash, pozzolans, concrete properties, durability, flexural strength, creep, inkage, abrasion resistance.

Ramon L. Carrasquillo, Ph.D. is Professor of Civil Engineering at the University of Texas at Austin. He is the Austin Industries Endowed Faculty Fellow.

P. J. Tikalski, Ph.D., PE is Assistant Professor of Civil Engineering at the Santa Clara University.

INTRODUCTION

Fly ash has been used in concrete for over 50 years in the United States. Initially fly ash was used in mass concrete to reduce the heat of hydration and cracking at early ages. More recently fly ash has become what some consider a necessary component of high strength concrete. While the chemical and physical characteristics of fly ash vary greatly between sources of ash, many individual sources of fly ash are collected with uniformity through good quality control techniques. The uniformity in the chemical and physical composition of the fly ash allows mix designers to predict the behavior of concretes containing different fly ash-portland cement combinations through analysis of fly ash characteristics, trial batching and performance records.

The use of fly ash as a component of structural and pavement concretes has become increasingly important in recent years. Technical and economic considerations will continue to expand the use of fly ash in these grades of concrete. In the past, several methods have been developed to proportion concrete containing fly ash and include factored volume and weight replacement schemes. These methods use a base mix design containing portland cement, which meets the appropriate job specification, and replaces a prescribed amount of cement with a factored amount of fly ash. In general, these methods do not directly consider the individual characteristics of different fly ash sources or the effects of different fly ash contents on the strength and durability of the concrete containing fly ash.

When designing a concrete mix containing fly ash, emphasis should be placed on the performance requirement of the concrete in the field. The mix designs must be proportioned so that the concrete will meet current job specifications, which include rheological properties, strength requirements, and long term durability.

This paper shows that, provided a good quality uniform fly ash source is available, fly ash can be used to produce concrete with the following properties:

(1) flexural and compressive strengths equal to or greater than concrete without fly ash.

(2) adequate workability and cohesiveness to provide easy placement, consolidation and finishing.

(3) long term durability to provide material and structural performance throughout the life of the structure.

EXPERIMENTAL PROGRAM

Test Procedures

A three year laboratory study was performed to determine the effects of fly ash on the mix proportioning procedures for concrete required to satisfy rheological, strength, and durability specifications. In 5 test series, over 1600 specimens were cast from concrete containing between 0 and 35 percent fly ash by weight of cementitious material. Each test series had a unique cementitious content; i.e. cement + fly ash, by weight. A list of mix designs are presented in Table 1.

The cementitious contents ranged from 517 to 658 pounds (5.5 to 7.0 sacks) per cubic yard of concrete and two separate and distinct sources of fly ash were used throughout the laboratory study. The concrete mixes were batched to satisfy a slump range requirement of 3 1/2 +-1/2 inches. Since the water demand of concrete containing fly ash is different from that of portland cement concrete, mixes in each series were adjusted to maintain a constant yield by varying the fine aggregate content appropriately. Although the w/(c+p) ratios varied between mixes, the workability of each was nearly identical.

Beam specimens, 6"x 6"x 20" (152mm x 152mm x 508mm), were cast and tested for flexural strength according to ASTM C293 with center point loading. Compressive strength of 6"x 12" (152mm x 305mm) cylinders was determined according to ASTM C39. Freeze- thaw resistance was determined by testing 3"x 4"x 16" (76mm x 92mm x 366mm) specimens according to ASTM C666 Method A. Abrasion resistance was tested according to ASTM C944, and creep according to ASTM C512. Shrinkage was measured using a mechanical strain gauge between two positively affixed 8 inch gauge points on opposite faces of 3"x 4"x 16" (76mm x 92mm x 366mm) prismatic specimens. All tests considered at least 3 identical specimens which were measured or tested at the same time.

In addition to the laboratory study, commercially produced high strength ready-mix concrete containing fly ash was studied. This limited test series had cementitious contents ranging from 837 to 960 pounds (8.9 to 10.8 sacks) per cubic yard of concrete. Only flexural strength and compressive strength were determined in this series according to ASTM Procedures C293, with third point loading and C39 on 6"x 6"x 20" (152mm x 152mm x 508mm) and 6"x 12" (152mm x 305mm) specimens, respectively.

Fresh concrete properties of both the laboratory and ready- mix concrete were recorded for each batch. Properties of the concrete tested included slump, air content, and unit weight according to ASTM Procedures C143, C231, and C138, respectively.

Material Properties

The physical and chemical properties of the cement and fly ash used in concrete can alter the rheological characteristics as well as the strength and durability of the concrete. A single source of Type I cement was used in the laboratory portion of this study, and was used as a constant reference throughout this study. In addition, a single source of Type IP cement was used containing 20 percent low calcium, ASTM C618 Class F fly ash which was interground with the portland cement clinker. Both the Type I and Type IP cements had a Blaine fineness greater than 3000 cm²/gm.

Two primary sources of fly ash were used in this study: a low calcium ASTM C618 Class F fly ash and a high calcium ASTM C618 Class C fly ash. The chemical and physical characteristics of the fly ashes are shown in Table 2.

Carbon in fly ash may have a detrimental effect on the durability of concrete containing fly ash in as much as organic air entraining agents have an affinity for the carbon. LOI values greater than 3 percent may be considered unacceptable according to ASTM Specification C618. The loss on ignition (LOI), which is a measure of the carbon content in the fly ash, was below 0.45 percent for both fly ashes.

A major difference between the two sources of fly ash was the distribution of oxides. The Class F fly ash had a calcium oxide (CaO) content of only 10 percent and a Silicon + Alumina + Iron (Si + Al + Fe) oxide content of 78 percent, whereas the Class C fly ash had a CaO content of 38 percent and a Si + Al + Fe oxide content of 57 percent.

The calcium oxide content of the fly ash will determine the time and type of reaction of that fly ash in concrete. Class C fly ashes, which characteristically have high CaO contents, will generally exhibit both cementitious and pozzolanic properties in concrete. This is due to the availability of a limited amount of calcium for reaction with the silicates in the fly ash to form calcium silicate hydrates upon the addition of water. However, the quantity of available CaO in Class C fly ashes is insufficient for complete hydration of the silicates; therefore, the excess silicates react at later ages with the calcium hydroxide which is produced as a by-product of the hydration of portland cement. This later reaction is known as a pozzolanic reaction, whereas the former is a cementitious reaction.

Class F fly ashes, on the other hand, have lower CaO contents than Class C fly ashes, therefore Class F fly ashes generally react in a mostly pozzolanic manner. The pozzolanic characteristics of fly ash may decrease the early strength gain of concrete, effectively lowering the early heat of hydration of the concrete. Where the early concrete temperature is of concern, the use of fly ash may be considered as a means of control against an excessively high heat of hydration of concrete.

The two ashes used in the laboratory portion of this study were chosen because they are readily available locally and are uniformly produced, high quality sources of fly ash. Single sources of 3/4 inch nominal maximum size silicious river gravel and natural sand were used throughout the laboratory portion of the study. Two types of admixtures were used in the laboratory: air entraining agents and a water reducing admixture. The water reducing admixture was used at the manufacturer recommended dosage rate and the air entraining agents were added to obtain specified air contents.

In the high strength concrete mixes, water reducers and a superplastisizer were used addition to crushed limestone coarse aggregate and natural sand, however no air training agents were added.

LABORATORY TEST RESULTS

The comparisons made in this section are for the two particular sources of fly ash and e cement used in this study. The fly ashes in this study are typical of those found in xas. They represent fly ashes with good chemical and physical characteristics, and it these characteristics which determine how a fly ash will perform in concrete.

xural strength

A primary concern in designing concrete for use in highway application is the flexural ength of the concrete. Concrete with low flexural strength will crack excessively due differential shrinkage, temperature variations and applied loads, leading to rapid terioration of pavements or highway structures. The flexural strength or modulus of pture of concrete is an indirect measure of the tensile strength and is the test most often ed by highway departments in evaluating concrete for pavements [8].

In this study, the minimum acceptable 7-day modulus of rupture from a center point ading was 600 psi. Cement was replaced by an equal weight of fly ash at the levels of , 20, 25 and 35 percent for cementitious, c + p, contents of 5.5, 6.0, 6.5 and 7.0 sacks r cubic yard.

The flexural strength of concrete containing Class F fly ash decreased slightly as the centage of fly ash was increased for all c + p contents. The concrete containing Class ly ash consistently met the minimum 7-day flexural strength requirement of 600 psi, gardless of c + p content or percent replacement. Test results are shown in Figure 1.

Flexural strength of concrete containing Class C fly ash equaled or decreased slightly the percentage of fly ash was increased. For all but one case, the concrete containing ss C fly ash met the minimum flexural strength requirement. The exception was the se of concrete containing 35 percent Class C fly ash with a c + p content of 5.5 sacks r cubic yard of concrete which reached a 7-day flexural strength of only 550 psi. Test ults for Class C fly ash are presented in figure 2.

A comparison of the 7 and 28 flexural strengths is shown in Figure 3. The average crease in flexural strength in concrete containing 25 percent Class C or F fly ash was percent at 7 days. Flexural strengths at 28 days were within 7 percent of the strength concrete without fly ash. Concrete produced with Type IP cement consistently met the nimum flexural strength requirement and exceeded the strength of the 5.5 sacks per bic yard mix without fly ash.

In general, fly ash concrete met the minimum specified strength and showed a higher rate of strength gain between 7 and 28 days than did concrete without fly ash. However, normal strength concrete containing either Class C or F fly ash displayed reduced 7-day flexural strengths when compared to mixes with the same cementitious content and no fly ash.

Compressive strength

Structural concrete is commonly specified by minimum compressive strength at 28 days. However, the compressive strength of concrete, at early ages is an important consideration during construction and at later ages for long term behavior.

The typical compressive strength development of concretes containing fly ash with age is shown in Figures 4 and 5. The cementitious content (c+p) was varied from 5.5 to 7.0 sacks per cubic yard of concrete in this study, with similar strength development characteristics.

After 28 days concrete containing Class C fly ash displayed varying compressive strengths with respect to cementitious content and percentage replacement of cement by fly ash as shown in Figure 6. Concrete with a c+p content of 5.5 sacks per cubic yard and 15 percent Class C fly ash yielded a 13 percent increase in compressive strength over concrete without fly ash, whereas 25 and 35 percent replacement by weight decreased the 28-day compressive strength by 7 percent.

At a c+p content of 6.0 sks/cu.yd. concrete containing between 20 and 35 percent Class C fly ash exceeded the compressive strength of concrete without fly ash by 8 percent. Mixes with a c+p content of 6.5 sks/cu.yd. showed strengths equal to that of the control concrete, regardless of the Class C fly ash replacement. At c+p contents of 7.0 sks/cu.yd. the 28-day compressive strength of concrete containing 15 percent fly ash was again higher than that of concrete without fly ash, while 25 and 35 percent diminished the strength.

Concrete containing Class F fly ash, displayed compressive strengths greater than that of portland cement concrete in mixes with a c+p content of 6 sks/cu.yd. The 28-day compressive strength of concrete containing Class F fly ash with c+p contents of 5.5, 6.5, and 7.0 was less than that of concrete without fly ash. Test results are presented in figure 7. These figures reinforce that for a given fly ash, an optimum replacement level exists for each different cementitious content. The required strength can be attained by using trial batching and performance records to determine the amount of fly ash needed at given cementitious contents. This is the same manner that commercial concrete mix designs are verified.

At 7 days the compressive strength of concrete without fly ash was higher than that of concrete containing fly ash, regardless of the type of fly ash or percentage replacement. However, the long term compressive strength of concrete containing fly ash exceeds that

of concrete without fly ash. For most concrete tested in this study the compressive strength was equal to or greater in concrete containing fly ash than in concrete without fly ash after 91 days.

In general, a specified 7-day or 28-day compressive strength can be met using concrete containing fly ash by considering the strength gain characteristics of the fly ash in the mix design and adjusting the c+p content or percentage of fly ash or both. When strengths are specified at later dates the required cementitious content will decrease while the percentage of fly ash will increase.

Effect of curing and mixing conditions

The mixing temperature and time, and the curing conditions under which concrete is subjected affect the compressive and flexural strengths. These effects were investigated in this study for concrete containing fly ash.

It was found in this study that concrete containing fly ash is more sensitive to prolonged mixing than is concrete without fly ash . Mixes containing fly ash exhibit a accelerated slump loss when mixed excessively. The increased slump loss is caused by the reduced water demand of concrete containing fly ash. The spherical particles in fly ash provide additional workability to the mix as long as the mix is fluid. Once the hydration of C_3A and C_3S begins and water starts evaporating from the mix the fly ash concrete with the lower water content becomes stiff and less manageable.

The flexural and compressive strength of concretes containing fly ash subjected to different curing conditions is shown in Figures 8 and 9. Concretes containing fly ash cured under hot dry conditions (100 F, 32% RH), moist cured (75 F, 100% RH) or cold cured (40 F, 55% RH) developed flexural and compressive strengths in the same relation as concrete without fly ash under the same conditions. The same relationships were observed at 35 percent fly ash and in 6.5 sack mixes. Compressive strength development of concrete containing Class F fly ash was slightly accelerated by the hot-dry condition, and diminished by cold curing.

Figures 10 and 11 show that fly ash content had a relatively minor effect on the strength development of concrete cured with a membrane curing compound at 100 F and 32% RH. However, the compressive strength development of concrete containing fly ash was substantially impaired by membrane curing at 75 F and 55% RH. The pozzolanic reaction in concrete containing fly ash will realize greater strengths at ambient temperatures when a moist environment is present; the membrane curing compound used was an inadequate moisture barrier for concrete containing fly ash.

When proportioning concrete containing fly ash, site conditions and temperatures should be considered. Excessive delays in the placement of the concrete may result in severe handling problems. It should be noted however that the increased paste-aggregate ratio in most concretes containing fly ash may improve the finishability of the concrete surface and reduce the permeability of the concrete.

Freeze-thaw resistance

The resistance to freeze-thaw action is a necessary property for concrete exposed to winter environments. Concrete which is not durable under freeze-thaw conditions will spall and crack, and later become unserviceable.

The most important elements in freeze-thaw resistant concrete are an adequate entrained air system and strength. Strength considerations have already been discussed. Fly ash affects the air system in concrete in that organic based air entraining admixture may be adsorbed by carbon particles in the fly ash. An increased dosage rate of air entraining admixture may be needed in concrete containing fly ash to compensate for the loss of admixture. The additional dosage in this study ranged from 0 to 2 times the standard dosage of air entraining agent, depending of the Class of fly ash used and the percentage replacement. Trial batching and previous performance records are the best way to determine the dosage rate of air entraining agents for fly ash concretes.

Tests were conducted on 135 specimens for freeze-thaw resistance according to ASTM C666, Method A. The cementitious content was varied from 5.5 to 7.0 sacks per cubic yard. Both Class C and F fly ashes were used as a direct weight replacement for cement at the levels of 0, 15, 20, 25, and 35 percent. All mixes containing greater than 3.5 percent air maintained a durability factor greater than 60. Average test results of specimens with 6.0 sacks per cubic yard and air contents between 4.5 and 5.5 percent are shown in Figures 12 and 13. Concrete containing Type IP cement completed the 300 cycle test undamaged with air contents as low as 2.5 percent.

Abrasion resistance

Concrete pavements and industrial floors subjected to heavy traffic may be damaged severely by the cracking and spalling associated with poor abrasion resistance. The abrasion resistance of concrete is related to aggregate type, compressive strength, curing conditions and the finished surface condition.

In this study, specimens were cast containing 0 and 35 percent Class C and F fly ash by weight. The cementitious content was 6.0 sacks per cubic yard and the air content was 5 percent in all mixes. In addition, the aggregate source, curing conditions and finishing techniques were identical for all specimens.

Abrasion resistance of concrete was measured as depth of wear caused by a rotating dressing wheel. Concrete containing Class C fly ash exhibited superior abrasion resistance to that of either plain portland cement concrete or concrete containing Class F fly ash.

ilar depths of wear were observed in plain concrete and concrete containing Class F ash, as shown in Figure 14.

Although the concretes had equal strengths at the time of testing, the reduction in ter demand in the fly ash mixes for equal workability reduces the amount of bleed water ich decreases the permeability of concrete, improving the surface hardness.

Furthermore, during the life of the pavement or floor the concrete containing fly ash continue to gain strength over that of plain concrete. Increased compressive strength add to the wear resistance of the concrete surface.

inkage

Shrinkage in highway pavements and in exposed structural members can lead to cking and the subsequent deterioration of concrete associated with cracked members, h as: corrosion of reinforcing bars, reduced freeze-thaw resistance and increased sceptibility to sulfate attack.

Shrinkage was monitored on concrete subjected to hot-dry conditions (100 F, 32% H after 3 days of moist curing and on concrete exposed to moderate conditions (72 F, % RH), moist cured for 14 days.

Concrete containing Class F fly ash reduced the shrinkage under both hot-dry and derate conditions when compared to concrete without fly ash. Concrete containing % Class F fly ash under hot-dry conditions had 28 percent less shrinkage than portland ment concrete after 120 days, and under moderate conditions an 11 percent reduction s measured after 220 days. Class C fly ash added to concrete did not change the long m shrinkage under hot-dry or moderate conditions when compared to concrete without ash.

eep

The inelastic time-dependent deformation of concrete caused by creep must be nsidered in deflection calculations to properly predict the behavior of both concrete slabs d structures. The primary mechanism of creep is best described as strains induced by e plastic flow of cement gel under load. This gel flow is directly affected by the gnitude of the sustained load with respect to the compressive strength.

Figure 15 shows concrete containing 35 percent Class F fly ash by weight displayed % lower creep deformations than concrete without fly ash, despite being subjected to igher load ratio. Similarly, concrete containing Class C fly ash showed a 5 percent luction in creep over that of portland cement concrete. Class F fly ash displayed a ater reduction because it is more pozzolanic in nature than the Class C fly ash and erefore continues to gain strength over a longer period of time.

It should be noted that the creep strains in concrete containing Class F fly ash were greater at early ages than those in concrete without fly ash. This is also due to the slower compressive strength gain characteristics of fly ash when compared to portland cement.

FIELD TEST RESULTS

The second portion of this study included high strength concrete mixes made with commercially produced concrete containing fly ash. Three Class C fly ashes, including the same Class C fly ash as was used in the laboratory portion of this study were used in this field study. Nine separate batches of ready-mix concrete were cast.

This portion of the study was conducted to show that Class C fly ash can be consistently used with water reducers and superplasticizers to produce good quality commercial concrete. Furthermore, mixes containing Class C fly ash can be designed with equal or less cementitious material by weight than mixes without fly ash and still produce higher compressive and flexural strengths.

Compressive strength

Concretes with high compressive strengths are being used in a broader range of structural applications in recent years. In addition to high rise construction, the prestressed and precast industries have used high strength concrete in many innovative projects. Fly ash is often an integral part of the high strength concrete mix design.

In this test series concrete containing fly ash is compared to a control mix without fly ash. The results are presented in tabular form in Table 2.

The 28-day compressive strength of concrete containing the Class C fly ash exceeded that of concrete without fly ash in 5 of the 7 fly ash mix designs. In addition, the 56 and 91 day compressive strengths of concrete containing fly ash in 3 of 5 fly ash mixes exceeded that of concrete without fly ash, despite 4 mixes containing fly ash having lower cementitious contents than the control mixes.

Flexural strength

The development of flexural strength of high strength concrete is of significant importance in the precast and prestressed industries. Cracking in concrete can lead to accelerated deterioration of prestressing strand as well as bonded reinforcement. The same concrete mixes used in the preceding section were used to cast flexural specimens tested for the data presented here. These series of tests are presented to indicate that the Class C fly ash can be used to obtain flexural strengths in excess of those obtained without fly ash, without increasing the cementitious content.

The flexural strength of high strength concrete containing fly ash is compared to that of the control concrete without fly ash in Table 3. The flexural strength of concrete containing fly ash depends on the source of fly ash and the amount used as a replacement. Class C fly ash in concrete consistently produced flexural strengths greater than that of concrete without fly ash at 7 days in 4 of 7 fly ash mixes. Even where the cementitious content was decreased by over 6 percent, the fly ash concrete provided greater flexural strengths. The flexural strength of concrete containing fly ash at 28 days was also equal to or greater than concrete without fly ash in 6 of 7 fly ash mixes.

CONCLUSIONS AND RECOMMENDATIONS

The use of fly ash in normal strength concrete produces excellent quality concrete which is durable and meets strength requirements. In addition, fly ash improves workability, reduces both heat of hydration and water demand and furnishes the mix with good fresh concrete rheological properties.

Concrete containing fly ash on an equal weight replacement displays:

*exceptional freeze-thaw resistance when a sound air void system is present,
*reduced creep deformation,
*equal of less shrinkage under either hot-dry or ambient conditions,
*equal or greater resistance to abrasion,
*slightly reduced early age compressive and flexural strengths
*long term compressive and flexural strengths which range from 5 percent lower to 20 percent higher than concrete without fly ash.

Also presented are the results of high strength concrete mixes, which show that concretes containing Class C fly ash may be designed to have:

*higher 28 and 91 day compressive strengths at lower cementitious contents,
*higher 7 and 28 day flexural strengths at lower cementitious contents.

Furthermore, the fly ash used consistently generated reproducible results. This may be the most important part of the study, in that it allows the mix design to be altered to obtain desired properties. Concrete mixes containing fly ash should be designed to meet to current specifications, not compared to mixes without fly ash.

In normal strength concrete containing fly ash, higher early strengths can be attained by adjusting the cementitious content or the amount of fly ash. The same effect may be gained by using a higher calcium or finer fly ash. Increased long term strength can be obtained by using a high silica, low calcium fly ash, using a finer fly ash, or again adjusting the amount of fly ash used.

Durable concrete containing fly ash requires an adequate air void system, good quality control and attention to curing condition at early ages. These are the same procedures which are required to ensure durability in concrete without fly ash.

The production of good quality normal and high strength concrete containing fly ash requires an analysis of the interaction between a particular combination of fly ash and portland cement. This is accomplished by trial batching, performance records and the evaluation of fly ash and portland cement physical and chemical characteristics. The final result is a high quality competitive construction material which can be used in all classes of concrete.

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Ramon L. Carrasquillo,
Professor of Civil Engineering,
University of Texas at Austin.

Table 1. Basic Mixtures Proportions per Cubic Yard

	Cement Content (sacks per cubic yard)			
	5.5	6.0	6.5	7.0
Cement*, lbs.	517	564	611	658
Coarse Aggregate, lbs.	2099	2099	2099	2099
Fine Aggregate, lbs.	1065	970	876	781
Water, lbs.	233	254	275	296
Air, percent	4.0	4.0	4.0	4.0

* Fly ash was used as a weight replacement for the cement and the fine aggregate content was adjusted to correct for proper yield in the mixes containing fly ash.

Table 2. Chemical and Physical Properties of Cement and Fly Ash

	Type I	Type IP	Class C	Class F
Chemical Composition				
Silicon dioxide (SiO_2) (%)	22.0	28.9	31.7	48.5
Aluminum dioxide (Al_2O_3) (%)	4.1	7.9	20.2	23.5
Ferric oxide (Fe_2O_3) (%)	3.1	3.8	5.1	6.4
Calcium oxide (CaO) (%)	65.8	53.1	38.1	10.2
Magnesium oxide (MgO) (%)	0.9	0.1	6.8	1.7
Sulfur trioxide (SO_3)	2.7	- -	4.2	0.9
LOI	0.90	- -	0.26	0.43
Insoluble residue	0.9	- -	- -	- -
Tricalcium silicate (C_3S)	59.0	- -	- -	- -
Tricalcium aluminate (C_3A)	6.0	- -	- -	- -
Total Alkalies	0.6	- -	1.6	0.7
Physical Properties				
Fineness (325 sieve, % retained)	------	------	12.8	12.8
Blaine (cm^2/gm)	3310	3026	----	3189
Vicat: initial, min.	86	106	----	----
final, min.	167	226	----	----
Pozzolanic Activity, %	------	------	97.0	106.1
Moisture, %	------	------	0.15	0.10

Table 3. Compressive strength results of concrete containing fly ash

Mix ignation	C+FA Content ft^3	Fly Ash % by vol. of C+FA	Compressive Strength (psi) 28-day	56-day	91-day	W/(C+FA) by weight	Fly Ash Source
trol #1	4.54	0	9871	11417	12133	.28	
A	4.35	31	10196	11685	12429	.31	W
B	4.26	35	9414	9763	10986	.33	W
trol #2	5.16	0	10154	11661	12332	.26	
C	5.16	35	11474	12704	12568	.24	W
D	5.12	25	9875	12194	13654	.26	W
E	5.07	31	11028	11820	12173	.27	X
F	5.07	30	11620	13151	———	.25	Y
G	5.02	30	12430	———	———	.29	Y

Table 4. Flexural strength results of concrete containing fly ash

Mix ignation	C+FA Content (sks/cu.yd.) ft^3	Fly Ash % by Volume of C+FA	Flexural Strength (psi) 7-day	28-day	w/(C+FA) by weight	Fly Ash Source
trol #1	4.54	0	1242	1323	.28	
x A	4.35	31	1098	1321	.31	W
x B	4.26	35	1156	1525	.33	W
trol #2	5.16	0	1013	1305	.26	
x C	5.16	35	1427	1429	.24	W
x D	5.12	25	1137	1388	.26	W
x E	5.07	31	910	1192	.27	X
x F	5.07	30	1200	1588	.25	Y
x G	5.02	30	1663	——	.29	Y

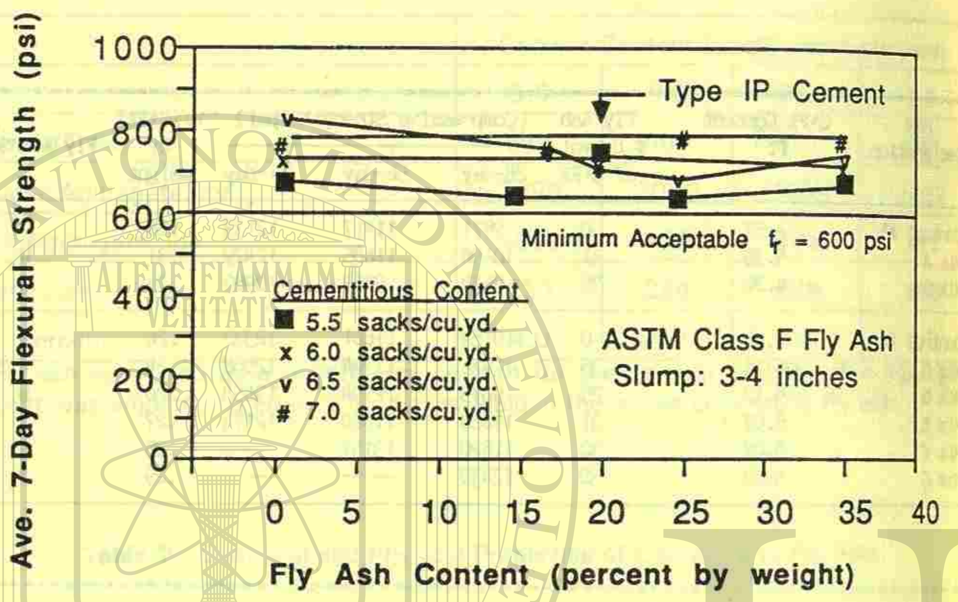


Fig. 1. Flexural strength of concrete containing Class F fly ash

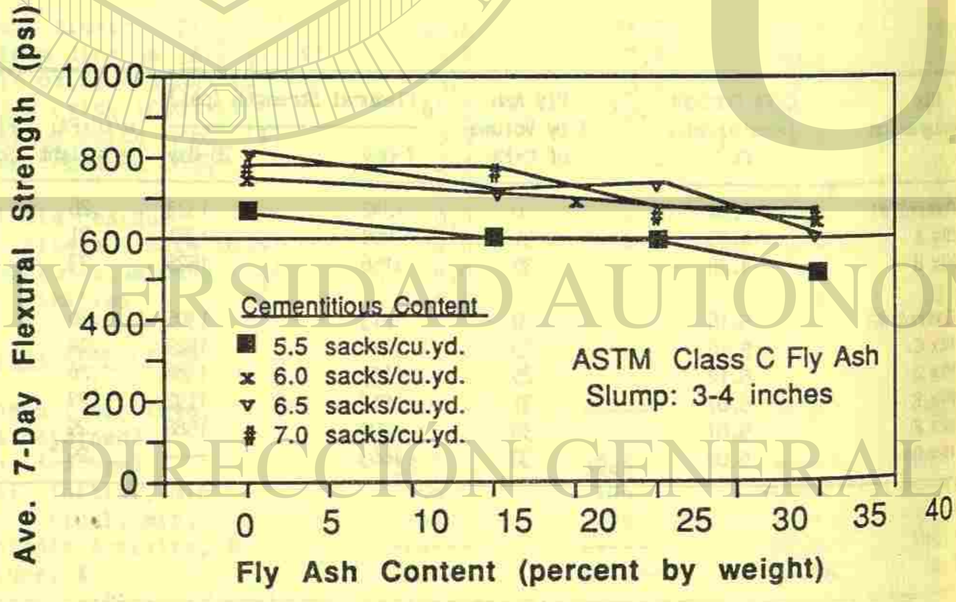


Fig. 2. Flexural strength of concrete containing Class C fly ash.

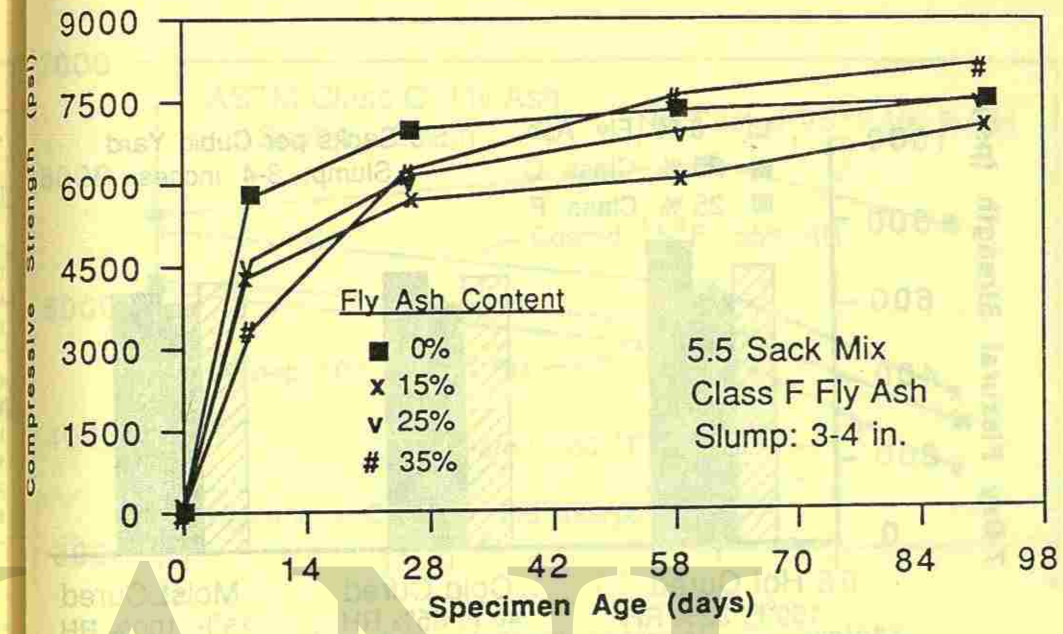


Fig. 3. Compressive strength development of concrete containing Class F fly ash.

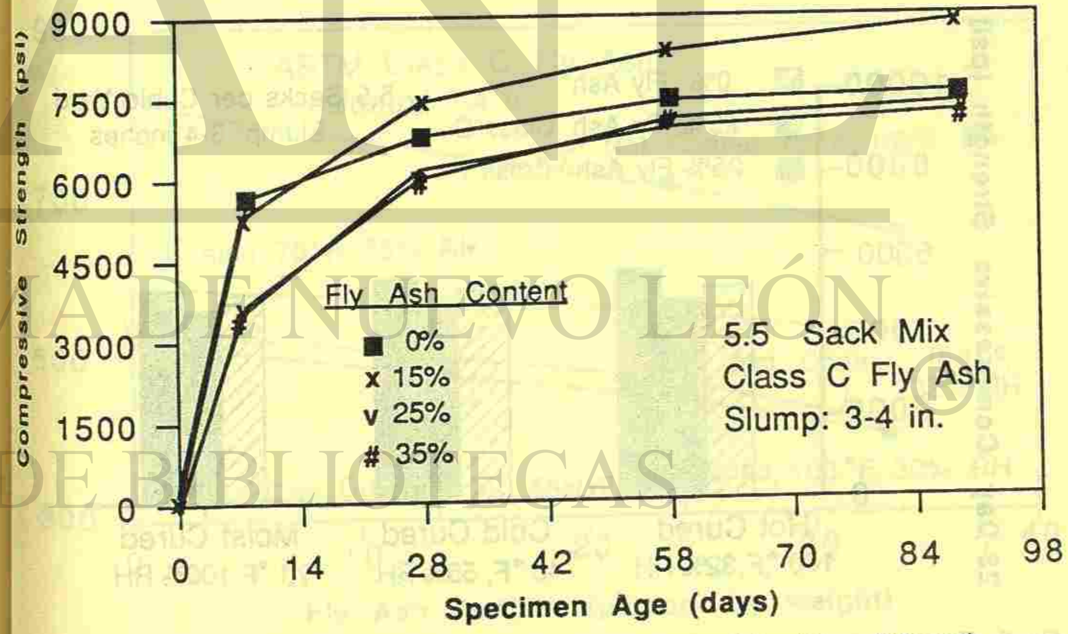


Fig. 4. Compressive strength development of concrete containing Class C fly ash.

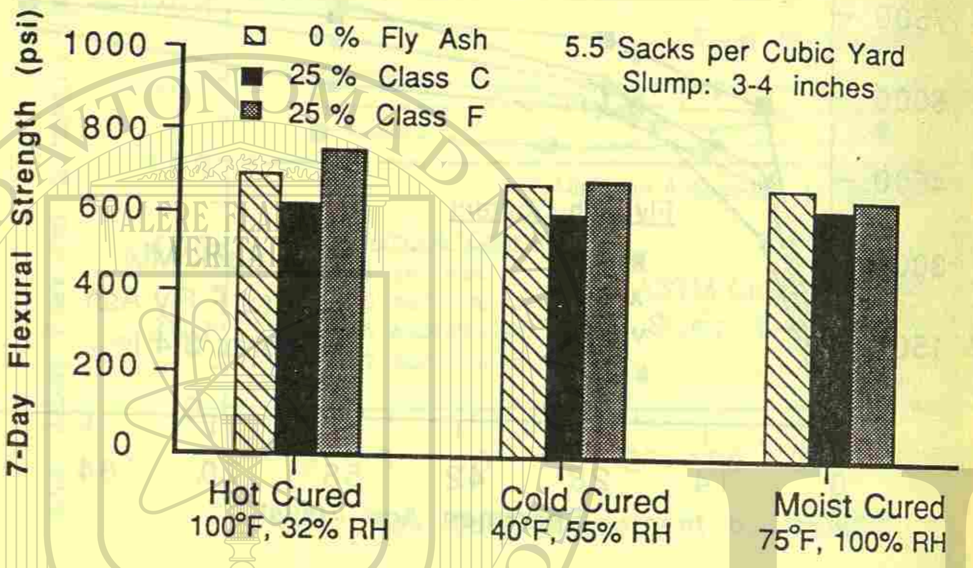


Fig. 5. Flexural strength of concrete containing fly ash under several curing conditions.

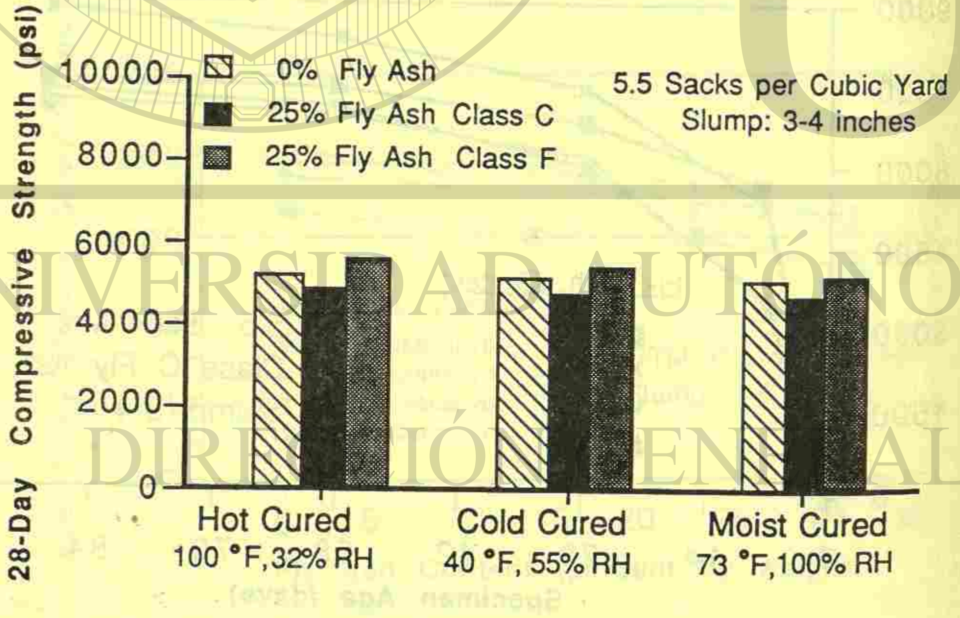
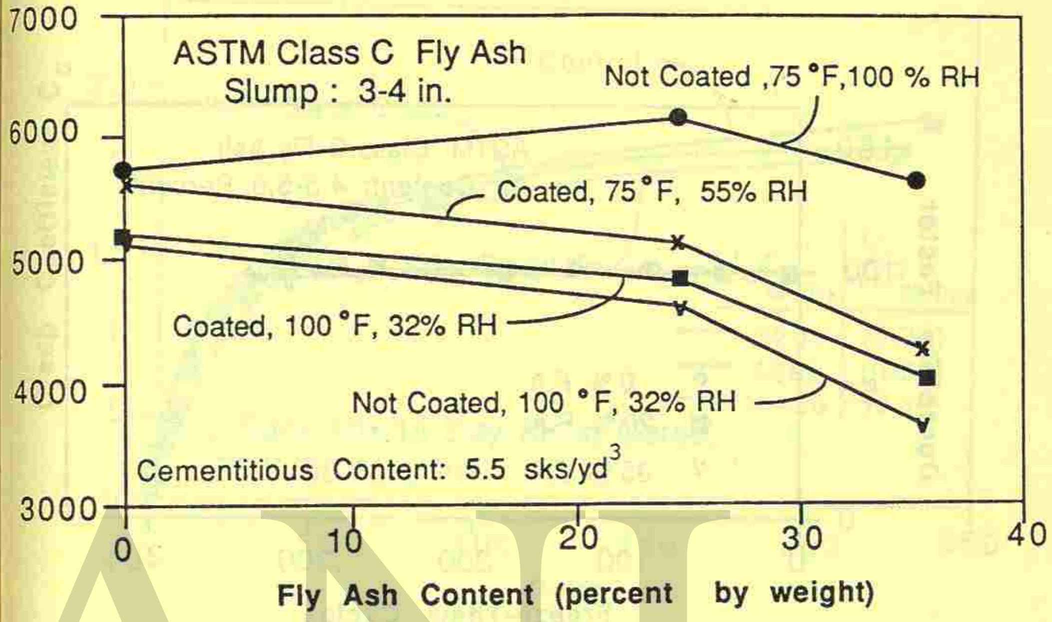
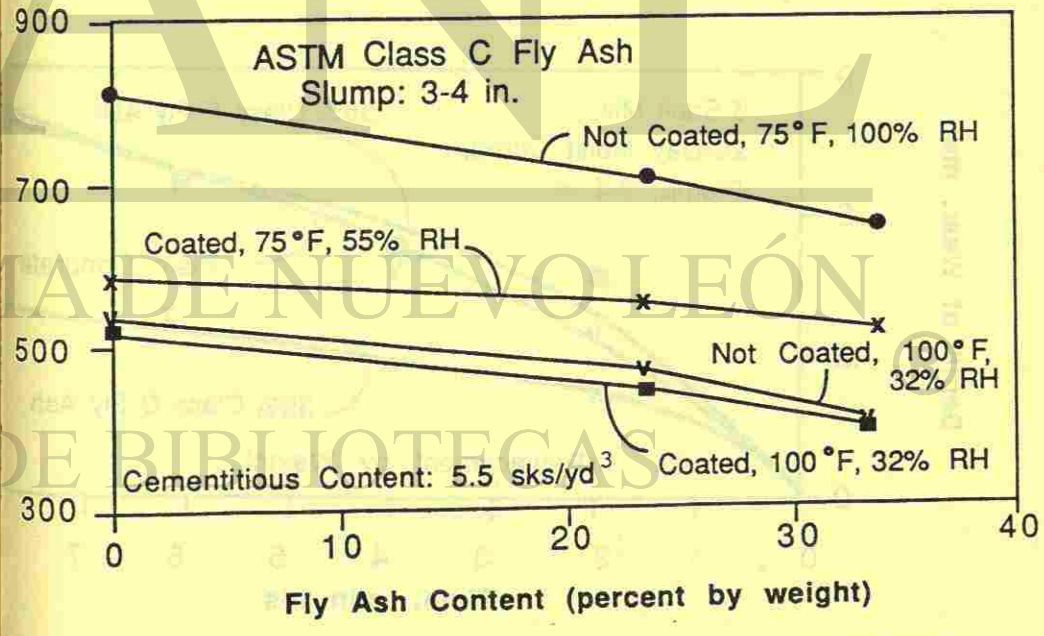


Fig. 6. Compressive strength of concrete containing fly ash under several curing conditions.



g. 7. Compressive strength of concrete with and without membrane curing compound.



g. 8. Flexural strength of concrete with and without membrane curing compound.

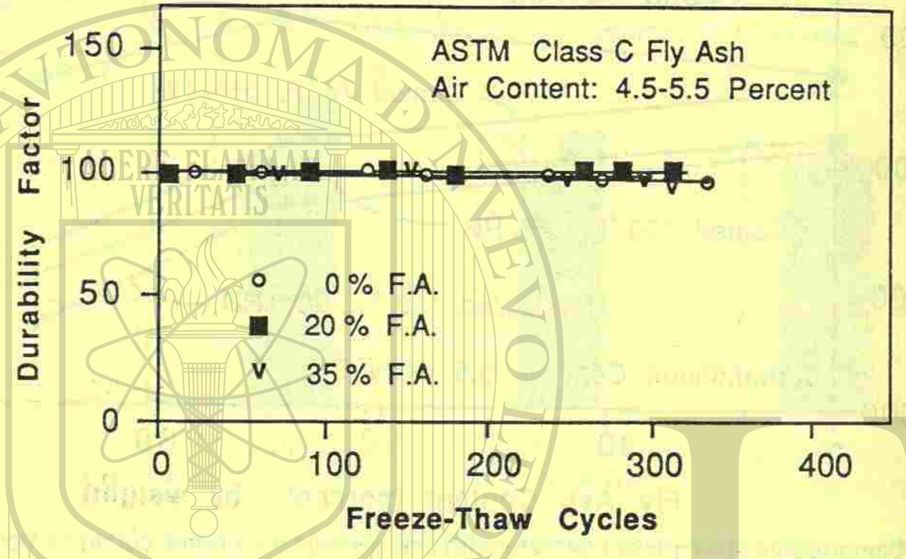


Fig. 9 Freeze-Thaw durability of concrete with fly ash.

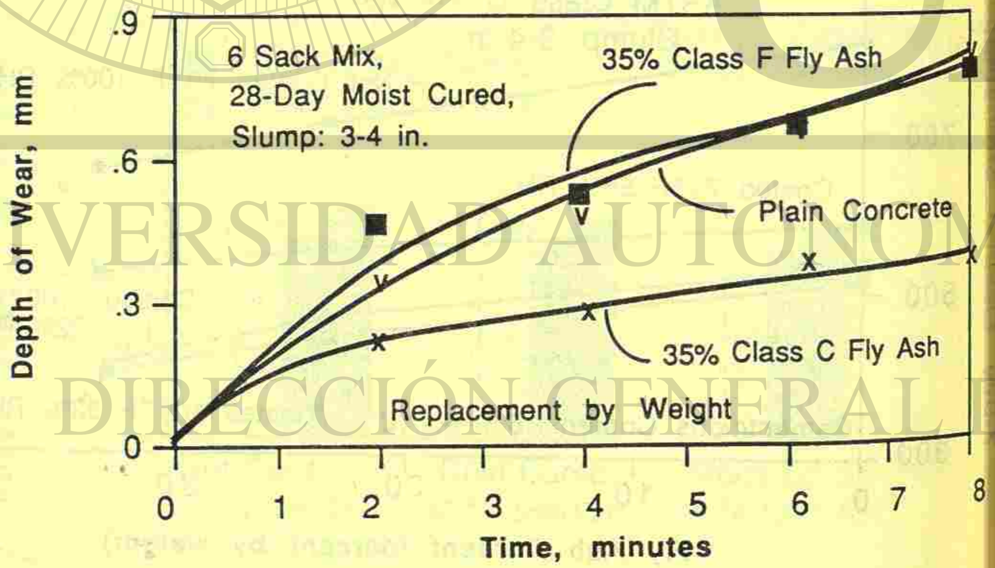


Fig. 10. Abrasion resistance of concrete containing fly as.

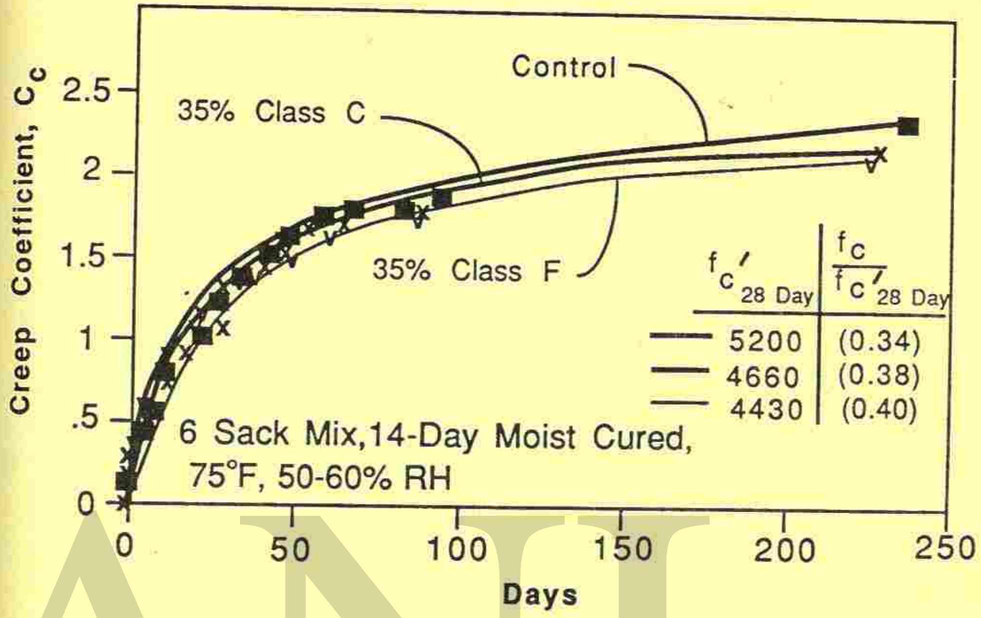


Fig. 11. Creep in concrete containing fly ash.

UNIVERSIDAD AUTONOMA DE NUEVO LEON
FACULTAD E INSTITUTO DE INGENIERIA CIVIL
MEMORIA DEL SEMINARIO INTERNACIONAL SOBRE DURABILIDAD DEL CONCRETO

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